ADDIS ABABA UNIVERSITY

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

DAM BREACH MODELING AND INUNDATION MAPPING

[A CASE STUDY OF DABUS DAM]

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Degree of Masters of Science in Civil Engineering (Hydraulic Engineering)

By:

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Advisor:

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Certification

This to Certify That This Thesis Entitled**Dam Breach Modeling and Inundation Mapping for Dabus Dam** Is Genuine, Done and Submitted

By

AbdulawalUmer

In Partial Fulfillment of the Degree of Masters of Science (M.Sc.)

In

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At

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School of Graduate Studies

Addis Ababa Institute of Technology

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ABSTRACT

Dam breach pre-event analysis is vital to predict the dam breach parameters, outflow magnitude and its downstream nature of propagation. Setting out risk management, emergency action plans or evacuation planning system to protect both lives and materials during sudden dam failure phenomena and resulting flood waves is highly essential. Development of effective emergency action plan demands accurate prediction of inundation levels and the time of flood wave arrival at downstream critical locations.

Dabus dam which will be constructed from rock fill material, within the Dabus Sub-basin of Abay Basin, has been selected for the study of dam breach analysis and flood inundation mapping.

The paper focuses on overtopping and piping as the possible reasons of breach and identifies the inundation properties for bunches of breach parameters suggested by four different scholars. This has resulted in eight computer runs where halves are for piping and the rest halves for overtopping.

HEC-RAS Ver5.0 is used to model the dam failure. HEC-GEORAS Ver 10.0 is used with ARC-Map 10.2.2 to extract basic geometric data for use in HEC-RAS. A 30m DEM is used as a terrain map to be used in ARC-Map.

The four empirical methods of Dam breach study have resulted in peak flow values within a range of 14220m3/s and 8838m3/s during overtopping and 12891m3/s and 12717m3/s during piping. The result shows that the area found around left and right of the river which has been occupied by villages will be inundated partially. These results can be said more close when compared with each other.

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

Key words: HEC-RAS, DEM, HEC-GeoRAS, Dam Failure, Breach, Dam Break Analysis, Scenario, Dabus Dam.

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List of Abbreviations

ADLI Agricultural development led industrialization

BC Boundary Condition

DEM Digital Elevation Model

D/s Downstream

EAP Emergency Action Plan

FEM Finite Element Model

FEMA Federal Emergency Management Agency

GIS Geographic Information System

HEC Hydrologic Engineering Center

HEC-GeoRAS Hydrologic Engineering Center's Geospatial River Analysis System

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

ICOLD International committee on large dams

IC Initial Condition

IDF Inflow design flood

Max WS Maximum Water Surface

MDDL Minimum Draw down Level

MWL Maximum Water Level

NPH No public hazard

NWS National Weather service

PCS Projected Coordinate System

PMF Probable Maximum Flood

PMP Probable maximum precipitation

Q_pPeak Discharge

RS River Station

SDF Spillway design flood

TFH Time of failure

U/s Upstream

USACE US Army Corps of Engineers

WS Water Surface

WSEL Water Surface Elevation

WWDSE Water Works Design and Supervision Enterprise

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

Appendix B: Output Results of HEC-RAS Mod

1. Introduction

1.1 Background

Dams are hydraulic structures used to store, control, divert water which storedbehind the upstream side of dam in a reservoir for different purposes, such as 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. 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.

Hence, while planning and implementing dams, a sufficient care must be taken and reliable Emergency Action Plan (EAP) should be set out to minimize the damages.

“A good example of dam failure and its hazards is the world’s worst dam disaster occurred in Henan province of China, when the Banqiao Dam and the ShimantanDam failed in August 1975catastrophically due to the overtopping caused by torrential rains. Approximately 85,000 people died from flooding, many more lost their lives during subsequent epidemics and starvation and millions of residents get their homes ruined. Modern dam safety issues have been evolving since 1970’s after four dams failed in the United States between 1972 and1977”(Zagonjolli, 2007).

Generally, causes of failure are overtopping due to inadequate spillway capacity, foundation defects, and piping and seepage. Simulation of embankment dam breach events and the resulting floods are crucial to characterizing and reducing threats due to potential dam failures. The increasing use of the risk assessment process as a planning and decision-making tool has highlighted the need for dam breach analysis.

Many organizations and researchers have contributed their findings in the analysis of dam break and its consequences. They have derived regression equations based on data from historical dam failure events that are used in predicting the breach geometry. These include the United States Bureau of Reclamation (USBR) and MacDonald and Langridge equations.

In Ethiopia construction of dams is increasing both in number and type, ranging from small micro earth dams constructed to fulfill community demands such as food and water supply to mega projects as that of the Grand Ethiopian Renaissance Dam (GERD), being implemented to boost the nation’s economy.

Dabusdamand Irrigation Development Project which are located in BenishangulGumz National Regional State at about 55 km east of Asosa town and about 725 km from Addis Ababais one of the rock fill dams of the country intended for impounding water for Irrigation Project, designed by Water Works Design and Supervision Enterprise and financed by the Ministry of Water irrigation and energy. An earthen dam is being constructed across Dabus River to store 202.6 Million cubic meter of water at its FSL. This large stored volume of water makes Dabus dam to be categorized as a large earth-fill dam. Hence, it often poses a higher hazard potential, given the extreme storage volumes and downstream population and property. Thus, implementation of such a large dam needs a complete risk analysis and the identification of pre-event measures. It is, however, quite recommended to risk-assess and to forecast dam break floods for already constructed dams as well dams at design stage so as to identify the risks involved in order to plan for the future and make wise decisions on necessary maintenances and operation of dam and reservoir respectively.

This study, dam break modeling and inundation mapping, “a case study on Dabus dam”, involved estimating the key parameters, dam break outflow hydrograph, time to dam failure, side slope of breach and Manning coefficients. The process of estimating the parameters will be done based on literature reviews and historical data collections. Softwarewill be utilized in the reservoir components of the study and in unsteady flow routing through the downstream reaches. Pre event dam break analysis and investigating the possible consequences is vital to make emergency preparedness. Hence, the intention of this study is to model a breaching process on Dabus Dam under different scenarios of failure mode the consequences that may happen on downstream.

This research is expected to contribute some useful information to Dabus Dam administrating body regarding to dam safety issues that should be considered, precautions to bear in mind while implementing infrastructures on downstream areas and input data in preparing EAP. Moreover, designers, consultants and contractors engaged on building dam may gain an input data for their work.

1.2 Statement of the Problem

In Ethiopia construction of dams has boosted in recent years that many large dams have been built and numerous are under construction. Their purpose includes development of irrigation, hydropower or water supply or the combination of these. Following the construction of dams, the downstream ecosystem is highly changed that huge area is covered with irrigation farms, new settlements and residence areas of inhabitants living on the farms and factories are formed. All these investments and newly settled inhabitants are highly exposed to flooding and are at risk for damage and death respectively.

Though no records are available on dam breaches and the damages they cause in Ethiopia, it is apparent that a pre-event analysis on dam break and assessing the possible risk is vital to:

- to give awareness to government bodies to take a precaution on dam safety plans
- early preparing of flood warning system
- give awareness for downstream inhabitants about the hazardous flood and
- Plan a proper emergency evacuation method on disaster time.

This early dam break analysis event is therefore aimed to investigate the possible breaching process of the Dabus dam and to delineate the area that would be flooded out due to the hazardous wave front.

The problems that might occur are summarized as follows:

1. Removal of dam body and its appurtenant structures which are constructed with high investment.
2. Loss of high volume of water which have been stored over year and could use for irrigate an enormous area of land.
3. Inundation of downstream area causing loss of human life and property

1.3 Research Questions

The following questions can be raised to initiate this study

- ✓ What predominant mode of failure would the dam probably manifest?
- ✓ What are the breach parameters defining the cross section of the breach?
- ✓ What are the different methods that would use to determine the dam breach and which is the most catastrophic?
- ✓ How long would it take for the breach process to complete and for the inflow flood together with the reservoir water to evacuate?
- ✓ How much area would the emerging flood inundate in the downstream until it attenuates?

1.4 Objective of the study

The general objective of this study is to model the dam breach process, apply it to the case of Dabus dam and map the downstream area to be inundated by the flood.

The specific objectives are:

1. To predict dam breach parameters using different breach parameter estimation methods.
2. To simulate the breach process applying different breach parameters that shall be set up based on regression equations which are developed by various scholars using historical dam break data.

3. To determine peak outflows for overtopping and piping failure approaches and then Estimation of the dam-break outflow hydrograph
4. To prepare various types of maps such as area, velocity, and depth.

1.5 Scope of the Thesis

This thesis shall cover determination of dam breach parameters and catastrophic outflow hydrograph. Different scenarios of mode of failure and initial breach size will be analyzed and then the further downstream routing and flooding condition will be done. One dimensional flood simulation model which is based on unsteady flow equations will be used for hydraulic simulation.

From the known modes of failures, seismic type will not be considered in this study due to the rare chance of occurrence during reservoir full condition.

1.6 Significance of the Study

In history of the world lots of lives and great amount of economy have been lost due to dam failure. Therefore, preparing dam safety plans and hazard management strategies are very important one.

Hence conducting pre event analysis of Dabus dam break and its consequences and forwarding the hazard extent to public offices have great significances among which some are listed below:

1. On time maintenance actions can be taken if the dam is susceptible to piping failure mode.
2. Flood early warning system can be developed after the disastrous dam breaching flood has been identified.
3. Downstream settlement of inhabitants can be planned in such a way that evacuation of the people during dam breach can be done rapidly saving as much lives as possible.

4. Protection dikes can be provided at both downstream banks with their top level considerably higher than the propagating flood level, especially on the stretches along residence areas and irrigation command area.
5. The thesis paper can be used as a reference for further detailed researches on the study area.

2. Literature Review

2.1 Background

Different literatures out put such as books, journal papers, master's and doctoral papers and different research publications focusing on this topic are going to be used in this thesis. The term dam breach analysis usually relates to the process of studying a dam failure phenomenon and analyzing the resulting consequences at downstream region. Dam break analysis generally deals with simulation of assumed failure for dams and analyzing the resulting consequences.

It is an evident historical fact that failure of dams have resulted in high casualties. Many dam failures have led to the inception of a new research field that deals with dam failures and its resulting consequences. There are however two basic perspectives to understand this research field that deals with dam failures. The first perspective is to answer the basic question that whether or not, a given dam will fail. This basically refers to the strength parameter of materials of dam section and this perspective basically deals with simulation of breaching process of dam section. Various parameters relating to dam failure like breach characteristics, time of formation etc. are estimated by this approach. The second perspective is to assume a given dam failure and study the resulting consequences in the downstream area. This approach helps in preparation of emergency action planning for a possible failure of dam. The final basic product of such analysis is inundation details of downstream area owing to the flood generated due to the failure of dam.

The techniques of breaching process have been improved with time from simple regression equations to modern physically based numerical models. Since dam failure phenomena is unanticipated and abrupt one, onsite observation and driving comprehensive analysis is rarely possible.

To formulate equations and develop models of embankment breaching process, physical models are usually used (Zagonjolli, 2007).

2.2 Dam Failures

As per guidelines for dam breach analysis (Colorado dam safety office, 2010), breach is defined as the opening formed in the dam body that leads the dam to fail and this phenomenon causes the concentrated water behind the dam to propagate towards downstream regions. Most scholars agree in categorizing the failure mode in one of the following types.

Overtopping is one of the most common failure modes for earth fill dams. It can be triggered by inflows higher than the design inflow, malfunctioning or a mistake in the operation of gates on spillway or outlets, inadequate carrying capacity of spillways, settlement of the dam or as result of landslides into the reservoir.

Piping failure: - is a failure mode caused by water penetrating through the dam's body, carrying with it small particles of dam material, continuously widening the gap. If the initial piping can be detected before it reaches the critical condition, remedy might be possible. Penetration of water in the dam body can cause slope failure. To prevent this type of failure, appropriate instrumentation is needed to estimate the rate of infiltration within an embankment.

Seepage failure or foundation failure: - occurs due to the saturation of the foundation material leading to either washout of the material or a weakening of the rock towards a sliding failure. The flow of water through a pervious foundation produces seepage forces as a result of the friction between the percolating water and the walls of the pores of the soil through which it flows.

Seismic failure of a dam: - is a one mode of dam failure triggered by earthquake which can result in catastrophic effects, even though it rarely happens. It can cause either landslide in the reservoir that induces a wave towards downstream causing overtopping of dam crest or liquefaction of the fill material to fail in shear and thereby breach develops.

2.3 Dam Failure Hazard Classification

A dam hazard classification is the placement of a dam into one of three/or four categories based on the hazard potential derived from an evaluation of the probable incremental adverse consequences due to failure or improper operation of the dam.

According to FEMA, 2013 hazard caused by dam failures is classified as;

- High Hazard Dam
- Significant Hazard Dam
- Low Hazard Dam
- No public Hazard (NPH) Dam

2.3.1 High Hazard Dam

High Hazard Dam is a dam for which loss of human life is expected to result from failure of the dam. Designated settlements located downstream within the bounds of possible inundation should also be evaluated for potential loss of human life. It is important to note that the potential of loss of a single life is sufficient to classify a dam as high hazard.

2.3.2 Significant Hazard Dam

Significant Hazard Dam is a dam for which significant damage is expected to occur, but no loss of human life is expected from failure of the dam. Significant damage is defined as damage to structures where people generally live, work or recreate, or public or private facilities. Significant damage is determined to be damage sufficient to render structures or facilities uninhabitable or inoperable.

2.3.3 Low Hazard Dam

Low Hazard Dam is a dam for which loss of human life is not expected, and significant damage to structures and public facilities as defined for a "significant Hazard" dam is not expected to result from failure of the dam.

2.3.4 No public Hazard Dam (NPH)

No public Hazard dam is a dam for which no loss of human life is expected, and which damage only to the dam owner's property will result from failure of the dam.

2.4 Dam Break Analysis

In either overtopping or piping cases, the erosion process is so fast that it takes minutes to few hours for the whole embankment to get washed away and for the reservoir water to drain out. The primary tasks in dam break analysis are predicting the out flow hydrograph and routing the flood through the downstream river channel and flood plain (Wahl, 1997). To predict the outflow hydrograph the parameters of the breach through which the reservoir water escapes have to be determined initially.

2.5 General Approaches to Dam Break Analysis

Guideline for dam breach analysis (Colorado dam safety office, 2010) suggests to all dam safety engineers to follow the four critical approaches listed below:

1. Breach parameter estimation (breach size (shape) and time of failure)
2. Breach peak discharge and breach hydrograph estimation
3. Breach flood routing, and

4. Estimation of the hydraulic conditions at critical locations.

2.6 Breach Parameter Estimation

Variation of breach parameters can affect peak discharge and inundation levels, as well as warning and evacuation time. The following are definitions of breach parameters.

A. Shape of Breach

(Wahl, 1997) states that for the hypothetical dam failures the shape of the breach is usually approximated as geometric shape such as rectangle, triangle, trapezoidal or parabola. The shape of the breach is greatly dependent on the type of dam. For embankment dams, a trapezoidal shape is common and for concrete arch dams usually the shape of the breach will be the same as the shape of the dam.

B. Breach Size

As the dam breach advances, the dimensions of the breach keep increasing. For an earth fill dam, usually a trapezoidal breach shape, overtopping failure starts as a small breach and progresses at a linear or non-linear rate down the height of the dam as shown in Figure blew.

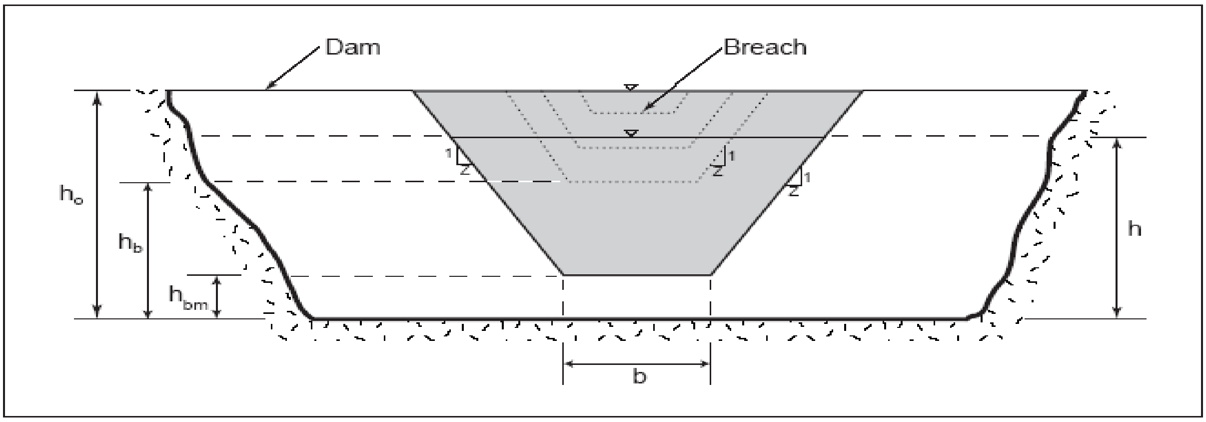


Figure 2-1: Overtopping Breach Dimensions (Fread and Lewis, 1998

For piping failures, the breach starts as a rectangle at some specified elevation from the crest as shown in Figure 2.2. The breach width and height grow until the elevation of the top of the breach is the same as the crest elevation, at which point the breach is identical to an overtopping failure.

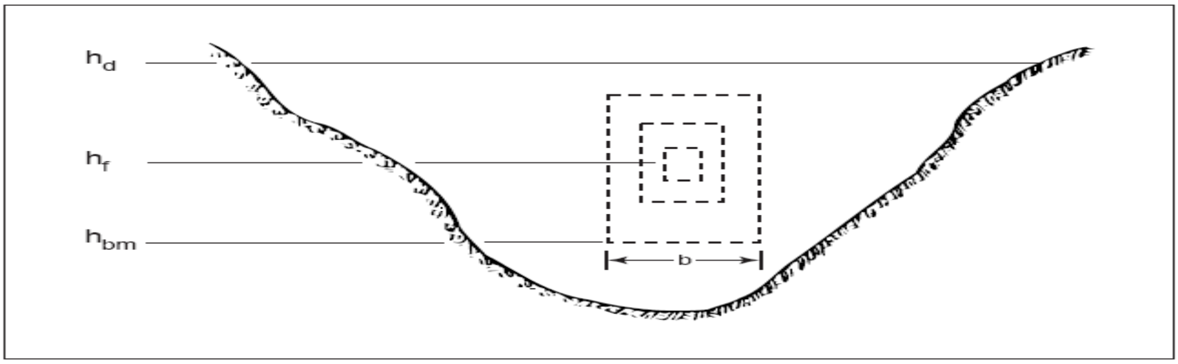


Figure 2-2: Piping Breach Dimensions (Fread, 1991)

Where: -ho= the breach top elevation at time t

hd= elevation at the top of the dam

hb= the breach bottom elevation at time

hbm= lowest breach bottom elevation at time t

hf= specified center-line elevation of the pipe

b = maximum breach bottom width at time t

C. Breaching Time

Breaching time refers to the time elapsed since the initial breach formation until it reaches its terminal size. The failure time for earth dams ranges from six minutes to three hours. Concrete dam failures are much more rapid, and have a time range of six to eighteen minutes. The time of failure most commonly depends on the size of the dam, types of materials used for construction, and the structural strength of the embankment.

D. Rate of Failure

The rate of breach expansion, increase in depth and width, is called rate of failure. This rate can progress at a linear or non-linear rate. Different models have been developed to estimate the breach parameters and magnitude of outflow hydrograph. These models can be classified as non-physical, semi-physical and physical models, as explained below.

I. Non-Physically Based, Empirical Models.

These are regression equations developed as best fit to available data of historically failed dams. Apart from their simplicity they have many disadvantages such as existence of considerable uncertainties within the predictions, their suitability for specific case is usually unknown since users have little knowledge on the background data set, only discrete values are predicted rather than detailed information on a process (i.e., peak discharge, not the whole hydrograph is predicted).

Breach Parameter Relations Based on Dam-Failure Case Studies is summarized below. The relations between breach depth, breach width, reservoir volume, dam height and other breach parameters as developed by various scholars is discussed. Using the known dam height and reservoir storage capacity, one can easily predict the breach parameters such as breach width, depth, formation time and side slopes.

Breach Parameter Relations Based on Dam-Failure Case Studies;

- ✓ **MacDonald et al**

They estimated the quantity of eroded embankment materials Ver (m3) for earth and rock dams as:

$$Ver = 0.0261(Vw*hw)^{0.769}; \textit{ for earth-fill} \qquad \text{Eq. 2.6.1}$$

$$Ver = 0.00348(Vw*hw)^{0.852}; \textit{ for rock-fill} \qquad \text{Eq. 2.6.2}$$

Where, Vw = volume of water discharged through breach (m3), and hw= hydraulic depth of water at dam at failure above breach bottom (m)

$$t_{f(hr)} = 0.0179(Ver)^{0.364} \quad \text{Eq. 2.6.3}$$

$$Wb = \frac{V_{eroded} - h_b^{2}\left(CZ_b + h_b Z_b \cdot \frac{Z_3}{3}\right)}{h_b\left(C + h_b \cdot \frac{Z_3}{3}\right)} \quad \text{Eq. 2.6.4}$$

Where: - W_b = Bottom width of the breach (m)

Veroded = Volume of water eroded from the dam

Vout= Volume of water that passes through the breach

hw= depth of water above the bottom of the breach

hb = Height of the dam to the bottom of the breach

C = Crest width of the top of the dam

$$Z3 = Z1+Z2$$

Z1 = Upstream Slope of the dam

Z2 = Downstream Slope of the dam

Zb = Side Slope of the Breach

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

Shown below is a peak flow equation that is developed from historic dam failures, by Mac Donald and langridge-Monopolis.

$$Q = 3.85(Vwhw)^{0.411} \quad \text{Eq. 2.6.5}$$

- ✓ **Von Thun and Gillette**

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

$B_{avg}(m) = 2.5hw + Cb$ Eq. 2.6.6

$t_f(hr) = 0.015*hw$ *for highly erodible dam*

$t_f(hr) = 0.0209*hw + 0.25$, *for erosion resistant dam*

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

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

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

Table 2-1: Values of Cb based on reservoir size

- ✓ **Froehlich, 2008**

In 2008 Dr. Froehlich updated his breach equations based on the addition of new data. Dr. Froehlich utilized 74 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rock fill data sets to develop as set of equations to predict average breach width, side slopes and failure time. The data that Froehlich used for his regression analysis had the following ranges.

Height of the dams: 3.05 -92.96 meters

Volume of water at breach time: 0.0139 – 660.0 m3 x 106

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

$Bave = 0.27\ Ko\ Vw0.32\ hb^{0.04}$ Eq. 2.6.7

$$T_f = 63.2\sqrt{\frac{V_w}{g h_b^2}} \quad \text{Eq. 2.6.8}$$

Where: Bave= average breach width (meters)

Ko= constant (1.3 for overtopping failures, 1.0 for piping)

Vw= reservoir volume at time of failure (cubic meters)

h_b = height of the final breach (meters)

g = gravitational acceleration (9.80665 m/sec2)

T_f = breach formation time (seconds)

Froehlich's 2008 paper states that the average side slopes should be:

1H: 1V for overtopping failures

0.7H: 1V otherwise (i.e. piping/seepage)

While not clearly stated in Froehlich's paper, the height of the breach is normally calculated by assuming the breach goes from the top of the dam all the way down to the natural ground elevation at the breach location. Frohelich has suggested the peak flow as follows;

$$Q = 0.607 V_w^{0.295} h_w^{1.24} \quad \text{Eq. 2.6.9}$$

- ✓ **Xu and Zhng**

Height of the dams: 3.2 - 92.96 meters

Volume of water at breach time: 0.105 – 660.0 m3 x 106

Xu and Zhang's regression equation for average breach width is:

$$\frac{B_{ave}}{h_b} = 0.787\left(\frac{h_d}{h_r}\right)^{0.133}\left(\frac{V_w^{1/3}}{h_w}\right)^{0.652} e^{B_3} \quad \text{Eq. 2.6.10}$$

Where: Bave = average breach width (meters)

Vw = reservoir volume at time of failure (cubic meters)

hb = height of the final breach (meters)

hd = height of the dam (meters)

hr = fifteen meters, is considered to be a reference height for distinguishing large dams from small dams

hw = height of the water above the breach bottom elevation at time of breach (meters)

B3 = coefficient that is a function of dam properties

The Xu and Zhang paper does not provide estimates for side slopes directly. Instead they provide an equation to estimate the top width of the breach, which can then be used within the average breach width, to compute the corresponding side slopes.

$$\frac{B_t}{h_b} = 1.062\left(\frac{h_d}{h_r}\right)^{0.092}\left(\frac{V_w^{1/3}}{h_w}\right)^{0.508} e^{B_2} \qquad \text{Eq. 2.6.11}$$

Where: Bt = Breach top width (meters)

B2 = coefficient that is a function of dam properties

Breach side slopes can be computed with the following equation

$$Z = (B_t - B_{ave})/h_b \qquad \text{Eq. 2.6.12}$$

The peak out flow discharge is suggested as follows,

$$\frac{Q}{\sqrt{gV_w^{5/3}}} = 0.175\left(\frac{h_d}{h_r}\right)^{0.199}\left(\frac{V_w^{1/3}}{h_w}\right)^{-1.274} e^{B_4} \qquad \text{Eq. 2.6.13}$$

2.6.1 Outflow Hydrograph Prediction

The breach outflow hydrograph is crucially important for the assessment of flooding characteristics in the downstream areas. As in the case of parameter estimation, there is also regression equations developed in order to predict the amount of outflow through the breach using dimensions of the dam and reservoir as discussed in previous section.

Various scenarios triggering dam failure such as variable inflow, different gate operations, seismic loads and sunny day failure are set and the selected model is simulated to predict the outflow hydrograph. The highest flood of the different scenarios is selected as a catastrophic one.

I. Modes of Failure

i. Overtopping failure

If an overtopping failure is simulated, the water level (H) in the reservoir must exceed the top of the dam before any erosion occurs. The flow into the channel is determined by the broad-crested weir relationship:

$$Q_b = (H-H_c)^{1.5} \quad \text{Eq. 2.6.14}$$

Where; H - elevation of the breach bottom

Q_b- flow into the breach channel

Bo – instantaneous width of the initially rectangular-shaped channel formed on

Downstream face of the dam

II. Piping Failure

If a piping breach is simulated, the water level (H) in the reservoir must be greater than the assumed center-line elevation (Hp) of the initially rectangular-shaped piping channel before the size of the pipe starts to increase via erosion. The bottom of the pipe is eroded vertically downward while its top erodes at the same rate vertically upwards. The flow into the pipe is controlled by orifice flow, i.e.,

$$Qb=A\ [\frac{2g(H-HP)}{(1+\frac{fL}{D})}]\ 0.5 \qquad \text{Eq. 2.6.15}$$

Q_b - flow (cfs) through the pipe

g - Gravity acceleration constant

A - Cross-sectional area (ft2) of the pipe channel

(H-Hp) - Hydrostatic head (ft) on the pipe

L - Length (ft) of the pipe channel

D - Diameter or width (ft) of the pipe, and

f - Darcy friction factor.

II. Breach Parameter Determination

i. Breach Width

The method of determining the width of the breach channel is a critical component of any breach model. In this model, the width of the breach is dynamically controlled by two mechanisms. The first assumes the breach has an initial rectangular shape. The width of the breach (Bo) is governed by the following relation:

$$B_o=B_r y$$

In which Br is a factor based on optimum channel hydraulic efficiency and y is the depth of flow in the breach channel.

The parameter Br has a value of 2 for overtopping failures while for piping failures, Br is set to 1.0. The model assumes that y is the critical depth at the entrance to the breach channel, i.e.

$Y=2/3(H-H_e)$

The second mechanism controlling the breach width is derived from the stability of soil slopes. The initial rectangular-shaped channel changes to a trapezoidal channel when the sides of the breach channel collapse, forming an angle (α) with the vertical, see Figure2.1Front View of Dam with Breach. The collapse occurs when the depth of the breach cut (H'c) reaches the critical depth (H') which is a function of the dam's material properties of internal friction (Φ), cohesion (C), and unit weight (γ), i.e.

$$H_k' = \frac{4C\cos\Phi\sin\Theta_{k-1}'}{\gamma[1-\cos(\Theta_{k-1}'-\Phi)]} \ldots K=1,\ 2,\ 3 \qquad \text{Eq. 2.6.16}$$

Where;

K – Denotes three successive collapse conditions

Θ – The angle that the side of the breach makes with the horizontal

The maximum discharge through the breach is dependent on the rate of breach enlargement via erosion and the rate at which the reservoir head decreases as a result of the increasing flow caused by the increasing breach opening.

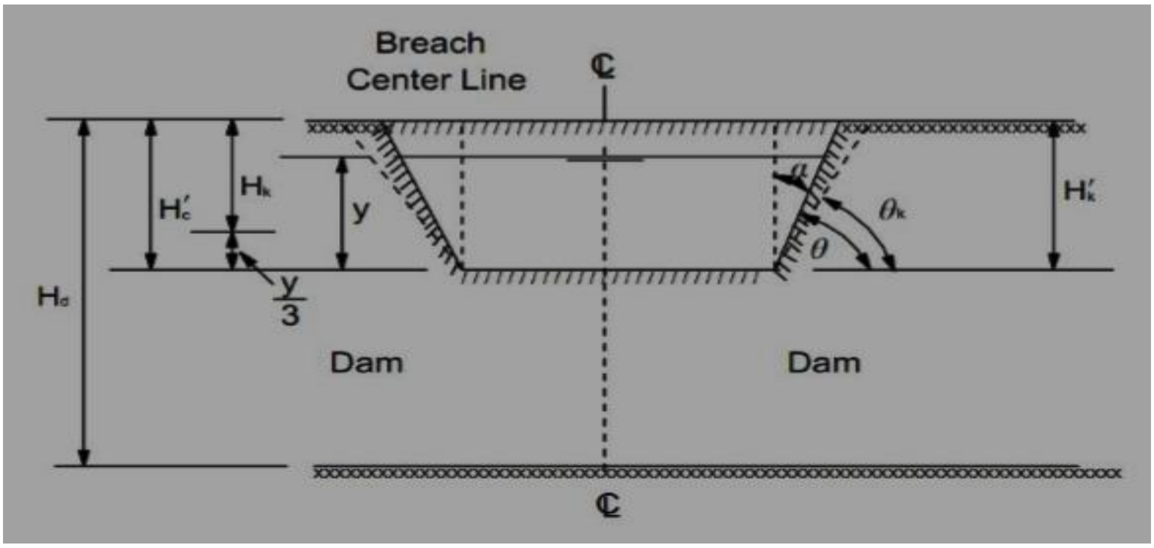


Figure 2-3: Front View of Dam with Breach (Fread D., 1991)

II. Reservoir Level Determination

Conservation of mass is used to compute the change in the reservoir water surface elevation (H) due to the influence of reservoir inflow (Qi), spillway outflow (Qsp), crest overflow (Qo), breach outflow (Qb), and the reservoir storage characteristics. The conservation of mass over a time step (Δt) in hours is represented by the following:

$$Q_i - \left(Q_b + Q_{sp} + Q_o\right) = S_a = \frac{\Delta H43560}{\Delta t3600} \qquad \text{Eq. 2.6.17}$$

In which ΔH is the change in water surface elevation during the time interval Δt, and Sa is the surface area in acres at elevation H. All flows are expressed in units of cfs. The reservoir elevation H, at time t can easily be obtained from the relation,

$$H = H' + \Delta H$$

Where, H ´ is the reservoir elevation at time t-Δt.

III. Breach Channel Hydraulics

The breach is assumed to be adequately described by quasi-steady uniform flow since the extremely short reach of the breach channel and very steep channel slopes (ZD) contribute very less flow variation along the breach length. This also greatly simplifies the hydraulics and computational algorithm compared to the unsteady flow equations as it had been contrasted and confirmed with the unsteady flow study conducted by Ponce and Tsivoglou (Fread D. , 1991). The flow is determined by applying the Manning's open channel flow equation at eachΔt time step, i.e.

$$Q_B = \frac{1.49 s^{0.5} A^{1.67}}{np0.67} \qquad \text{Eq. 2.6.18}$$

in which S = 1/ZD, A is the channel cross-sectional area, P is the wetted perimeter of the channel, and n is the Manning coefficient computed using Strickler relation which is based on the average grain size of the material forming the breach channel, i.e.,

$$n=0.013D_{50}^{0.67}$$

IV. Sediment Transport

The rate at which the breach is eroded depends on the capacity of the flowing water to transport the eroded material. The Meyer-Peter and Muller sediment transport relation as modified by Smart (1984) for steep channels is used, i.e.

$$Q_S=3.64\left(\frac{D_{90}}{D_{30}}\right) P\frac{D}{n}^{2/3} S^{1.1}(D_S-\Omega) \qquad \text{Eq. 2.6.19}$$

In which Q s is the sediment transport rate (cfs);

D30, D50, D90 (mm) are grain sizes at which 30, 50, and 90 percent of total weight is finer;

D is the hydraulic depth of flow (ft), S is the slope of the downstream face of the dam.

v. Breach Enlargement by Sudden Collapse

The primary weakness of BREACH and other similar simulator models is the fact that they do not adequately model the head cut-type erosion process that dominates the breaching of cohesive-soil embankments (Wahl, 2004). The model considers the possibility for the breach to be enlarged by a sudden collapse failure of the upper portions of dam in the vicinity of the breach development. The collapse would be due to the pressure of the water on the upstream face of the dam exceeding the resistive forces due to shear and cohesion which keep the wedge in place. A check for collapse is made at each Δt time step during the simulation. The collapse check consists of assuming an initial value for Yc of 10 and then summing the forces acting on the wedge of height, Yc. The forces are those due to the water pressure (Fw) and the resisting forces which are the shear force (Fsb) acting along the bottom of the wedge, the shear force (Fss) acting along both sides of the wedge, the force (Fcb) due to cohesion along the wedge bottom and (Fcs), the force due to cohesion acting along the sides of the wedge, as shown in Figure 2. Thus, collapse occurs if:

$$FW > Fsb + Fss + Fcb + FCS$$

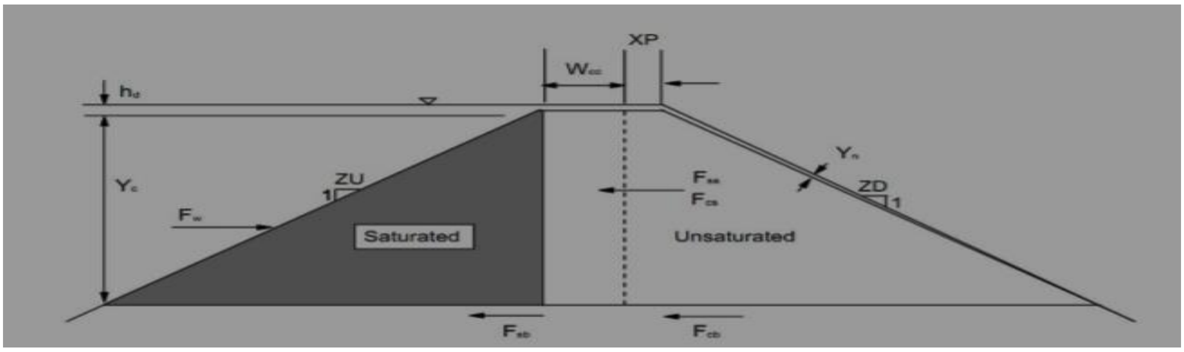


Figure 2-4: Forces Determining the Possible Collapse of the Upper Portion of the Dam (Fread D.1991)

2.6.2 Modeling the Propagation of Flood

Having determined the breach parameters and peak outflow the next task is to route the most catastrophic outflow hydrograph over the downstream channel and flood plain and prepare an inundation map.

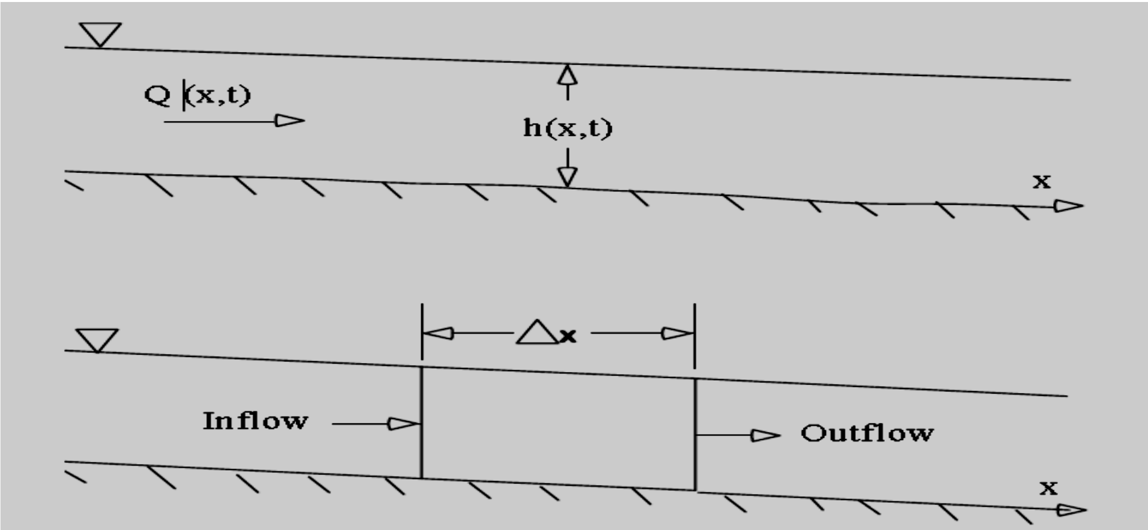
In one-dimensional flow routing, flow through the river channel and the floodplains is treated only in the longitudinal direction parallel to the reach.

Governing Equations

The laws 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

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



Inundation maps show the flood contour for natural floods of certain return periods, in most cases up to PMF, and flood contours of potential dam break floods caused by a “sunny day failure” and a failure superimposed over certain natural “base flood conditions”. (ICOLD, 1998)These inundation maps are used to control building activities, develop necessary warning and evacuation plans and conduct consequence assessments.

Various models have been developed to execute the flood inundation mapping process. HECRAS is an example of one dimensional models analyzing downstream flow using unsteady flow equations. The FESWMS (Finite Element Surface Water Modeling System) is a two dimensional model developed by the Federal Emergency Management Agency (FEMA) to simulate movement of water in rivers, estuaries and coastal waters. While the

1-D models require cross sectional data of flow channels with sufficient extension on both left and right direction, the 2-D models use topographical data in the form of a continuous surface represented by finite element mesh. Case studies comparing 1D and 2D could show that the 2D models represent more accurate flow in the flood plain due to both lateral and longitudinal movement of water (Cook, 2008).

In spite of the timely improvements of flood routing models, the predictive accuracy of such models can be subject to significant error (2 feet or more in the crest profile) due to inaccuracy in the computed reservoir inflow, breach parameter estimation and dynamics, downstream cross section properties, flow resistance coefficients both in channels and flood plains, effect of sediment transport and flow constrictions in channel and the complex flow patterns that are not adequately described by one-dimensional flow equations (Fread D. , 1981). One-dimensional models are best suited to geographic regions with moderate to steep slopes whereby flood waters are contained within a relatively narrow flood plain and generally flow in single stream direction. Whereas two dimensional models can route both channel flow which is one-dimensional and two dimensional overland flows on flat terrain (FEMA, 2013).

To correctly estimate the consequences derived from a structural failure modeling of flood propagation should be highly accurate. Identification of the inundated areas, inundation depth, speed and duration, as well as the impact that flood water characteristics (salt, freshwater, contaminated water, etc.) can have on the inundated areas, are very important for decision making, emergency evacuation and early warning. The De Saint–Venant equations or shallow water equations are used for modeling dam break flood propagation. These equations consist of the mass and momentum conservation equations, and assume that the vertical velocities are much smaller than the horizontal velocities, which leads to hydrostatic pressure distribution in a channel cross section (Zagonjolli, 2007).

2.6.3 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 conditions, over topping and piping inundation, are 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-Geo-RAS Arc-Map 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)

2.7 Review of Selected Model

2.7.1 The HEC-RAS model

The Hydrologic Engineering Center’s River Analysis System (HEC-RAS) is a simulation software developed by the US Army Corps of Engineers and has been developed to manage rivers and other public works under their jurisdiction. The HEC-RAS software has found wide acceptance among hydraulic engineers and researchers due to its robust channel flow analysis capabilities and its ability to simulates unsteady flood wave propagation and identifies flood prone areas as the areas where the ground is lower than the computed water elevation and allows the user to visualize the flood propagation in real time, thus making the software ideal for dam breach modeling.

The HEC-RAS system contains four one-dimensional river analysis components for:

(1) Steady flow water surface profile computations;

(2) unsteady flow simulation;

(3) Movable boundary sediment transport computations; and

(4) Water quality analysis. A key element is that all four components use a common geometric data representation and common geometric and hydraulic computation routines (HEC, 2010).

The HEC-RAS system is comprised of a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, and graphing and reporting facilities.

HEC-RAS is able to take into consideration hydraulic effects of bridges, culverts, weirs, and other structures in the river and floodplain on water surface calculations (HEC, 2010). The HEC-RAS modeling system was developed as part of the Hydrologic Engineering Centers Next Generation software and replaces several existing Corps of Engineers programs, including the HEC-2 water surface profile program.

The dam break tool in HEC-RAS can simulate the breach of an inline structure such as dam, or a lateral structure such as a levee. The latest versions of the HEC-RAS model include algorithms to model both overtopping and piping breaches (HEC, 2010). HEC-RAS uses hydraulic principles through cross sections upstream and downstream of the dam to define how the reservoir drains during the formation of a dam breach.

The dam crest is modeled as an inline weir and either a piping failure or overtopping failure is simulated with enlargement of the breach occurring over time as defined by a specified breach progression. Flow through the piping hole is calculated as orifice flow and flow through the breach is calculated as weir flow. The water surface profile upstream of the dam is back-calculated using unsteady momentum and hydraulic principles for each time step and the resulting drawdown through the hole and/or breach produces an outflow hydrograph. Resulting water levels for each time step downstream of the dam are used to model potential backwater effects and the weir and orifice coefficients are automatically adjusted for submergence, if necessary.

HEC-RAS can also model a piping failure that does not progress to the point of collapsing the crest. In this scenario, the piping hole is simulated as a sluice gate (HEC, 2010). The simplest case that can be used for testing the coupling of breach process models with HEC-RAS is that of a single reach bounded upstream by a dam with a reservoir described by a storage-elevation function, and the next refinement that can be simulated is the same system with dynamic routing being used for flow in the reservoir.

The HEC-RAS computational program has the ability to model extreme flow dynamics in the downstream of the reach due to a dam break flood waves and produce water surface profiles over the length of the modeled area (Asburry and Goodell, 2009). HEC-RAS is the most widely used hydraulic model for dam safety analyses in the United States and can be utilized for steady and unsteady flow analyses (Colorado dam safety office, 2010). Moreover, the HEC-RAS outputs like water surface profiles can easily be converted to flood inundation maps by its companion program called HEC-Geo-RAS which is an Arc-GIS extension.

Because of complexity of solving routines, the output solutions were thoroughly reviewed for stability and correctness. Furthermore, many studies indicate that the solution found by HEC-RAS is stable and trustworthy

(Ackerman and Brunner, 2006). Hence, due to its extensive capabilities and freely availability in USACE website; its' outputs compatibility with Arc-GIS (HEC-GeoRAS) software packages and for the said advantages, HEC-RAS model is selected by the Author to simulate the Dabus dam break.

2.7.2 HEC-Geo-RAS

HEC-GeoRAS is an ArcGIS extension developed by the HEC. This model contains a set of tools specifically designed to process geospatial data to support hydraulic model development and analysis of water surface profile results. It assists in creating data sets in GIS to extract information essential for hydraulic modeling.

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

2.7.3 ArcGIS

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

3. Methodology

3.1 Description of the Study Area

“DabusDam and reservoir is located in BenishangulGumz National Regional State at about 55 km east of Asosa town and about 725 km from Addis Ababa. The project area is located in Abbay River Basin with geographic coordinates of 10^{O} 02’ to 10^{O} 14’ N and 34^{O} 51’ to 35^{O} 02’ E between altitudes of 850 and 1,100 masl.The proposed headwork site is located near Fudindu, a kebele market town in Mengewereda with an all-weather gravel road and public transport connection to Asosa. The topography of the project area is characterized mainly by plains and undulating land. The agro ecology of the area is characterized as wet kola or lowland, which is below 1,500 masl with minimum and maximum temperatures ranging from 20^{o}C - 25^{o}C in May to October and 35^{o}C - 40^{o}C in November to April and the average lies between 20^{o}C -25^{o}C (MoARD 2010). During the hottest months (January - May) it reaches a 28^{o}C - 34^{0}C.Hydrologic ally, Dabus River and its catchment have a number of features. According to a study by Seleshi et al (2011), Dabus River is the biggest tributary of Abay River (Upper Blue Nile) with a mean annual flow of 6246Mm^3 at its confluence with Abay River. The Dabus River catchment is also the biggest in Abay Basin with a total area of 21030km^2. The rainy season spreads through May to October. The rainfall distribution is mono modal and the annual rainfall ranges from 500-1800 mm and the mean is between 680 to 1200 mm. with the total potential evaporation of 1112mm. Contrary to its exceptional hydrologic attributes, it has a runoff coefficient of 0.13, which is the lowest in the whole basin. The reason for that is mainly attributed to the presence of a big swamp in the upper part of the catchment, where highest precipitation falls. The Swamp area strongly controls the flow characteristics of the River”. (WWDSE)

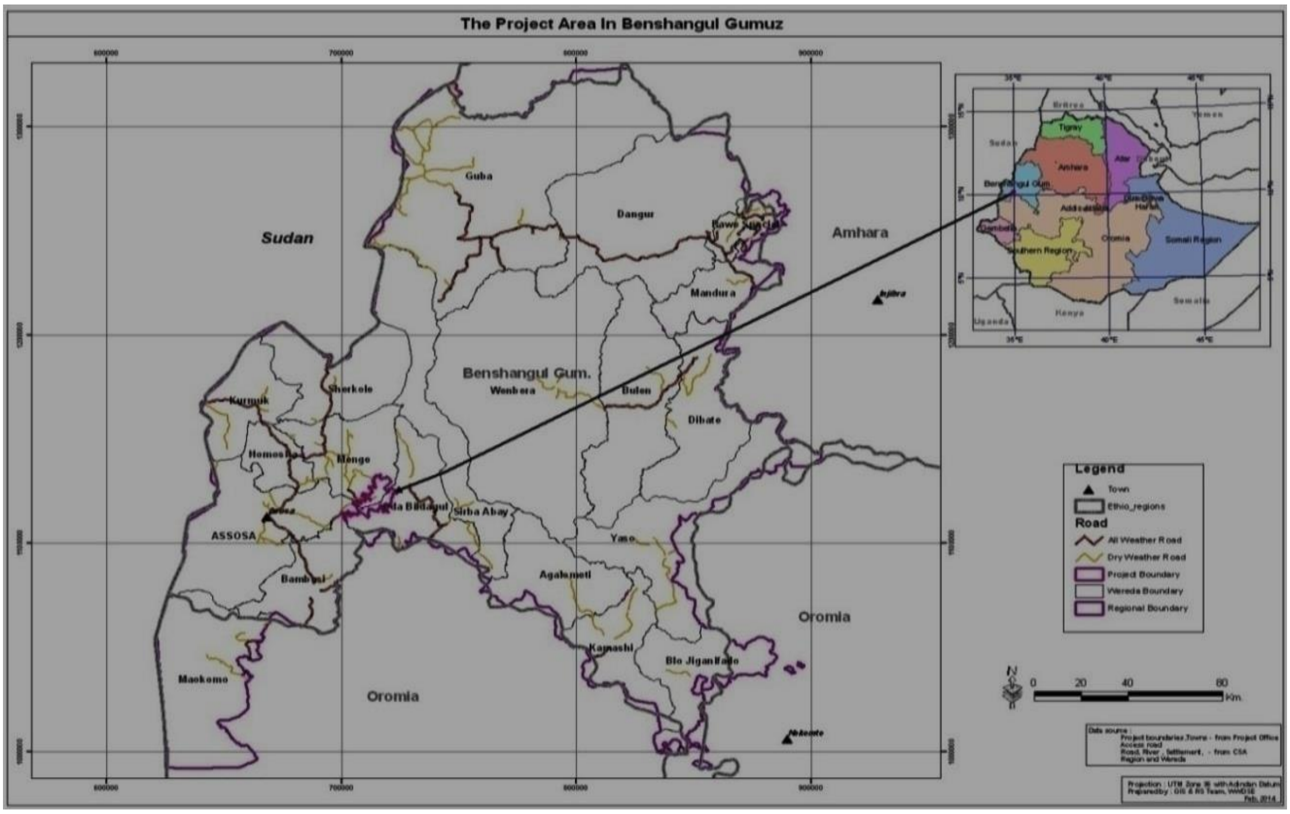


Figure 3-1: Location map of the study area (WWDSE)

3.2 Catchment Morphology

According to water works design and supervision enterprise Dabus River originates from the highlands dividing the Baro-Akobo and southern boundary of Abay basin. At the headwork location the river has a catchment area of about 11723Km2. The catchment has a mean slope of 10.93% and mean catchment elevation of 1536masl that ranges from 3130masl near Begito 976masl at the outlet. At the very upstream of the catchment, Dabus River flows through a valley in the middle of a large plateau and overflows from its main course to form the Dabus Swamp. The swamp has an average surface area of about 370km2and could potentially store about 1.8 Bm3 (Hal crow, 1982).About 60% of the catchment directly outflows in to the swamp, including Dilla River.

The catchment receives an annual rainfall in the order of more than 1500mm throughout the catchment. The rainfall distribution in the Dabus basin is mono-modal, with the length of the wet season decreasing as one goes to the North and Northwest inthe basin. The southwestern part of the basin experiences longer rainy season extending from April/May to October/November.

The irrigation command area falls entirely within the climatic classification of Tropical Climate II. The land in the catchment is largely covered by many small patches of cultivation mainly in the Eastern and Southern part of the catchment, woodlands in the middle and natural forests composed of bamboos, grasses, shrubs and indigenes trees in the Southwest including the command.

The catchment is dominantly covered by a red clay soil called Nitosols with some 20% of the catchment in the east consisting Gleysols. Vertisols (heavy black clay soils) are found near the swamp region. The command is composed of Nitosols in the left bank and Xerosols (semi-desert soils) in the right bank side. They are fertile and well-drained soils suitable for irrigated agriculture.(WWDSE)

No.	Station name	Class	Elevation, m	Latitude	Longitude	Location
1	Assosa	1	1600	10.01	34.31	In the watershed
2	Bambasi	3	1460	9.43	34.43	In the watershed
3	KiltuKarra	4	1850	9.43	35.13	In the watershed
4	Begi	3	1650	9.2	34.32	Out of the watershed
5	Kurmuk	3	800	10.26	34.18	Out of the watershed
6	Mendi	3	1650	9.47	35.06	In the watershed
7	Menge	4	1200	10.2	34.44	Out of the watershed
8	Nedjo	1	1800	9.3	35.27	In the watershed
9	Hoha	2	1550	10.09	34.39	In the watershed
10	Dengoro	1	1900	9.12	35.44	Out of the watershed
11	Jarso	4	1750	9.27	35.19	In the watershed

Table 3-1: Distribution of meteorological stations in Dabus sub-basin (WWDSE)

3.3 Dam and Spill Way Type

The main dam body is a rock fill with central clay core. Downstream and upstream filter, drainage zone and transitions are provided to protect migration of soil, to drain seepage water and smooth transition from fine to coarse textured dam fill materials are also provided. The embankment crest length is 760.0m and the maximum height above river bed level is 37m.

The reinforced concrete spillway is designed to discharge a maximum of 3444.70m3/s, with a nominal crest length of 100m. The crest level of the spill way is 1007.00 m. The structure is 660m long, comprising an approach channel, ogee crest, chute and flip bucket as terminal structure. The length of the approach channel is about 255m. The spilled water leaving the terminal structure will join the natural river channel which is composed of erosion resistant bed rock. A 6.5m wide reinforced concrete bridge provides access across the spillway. (ECDSC)

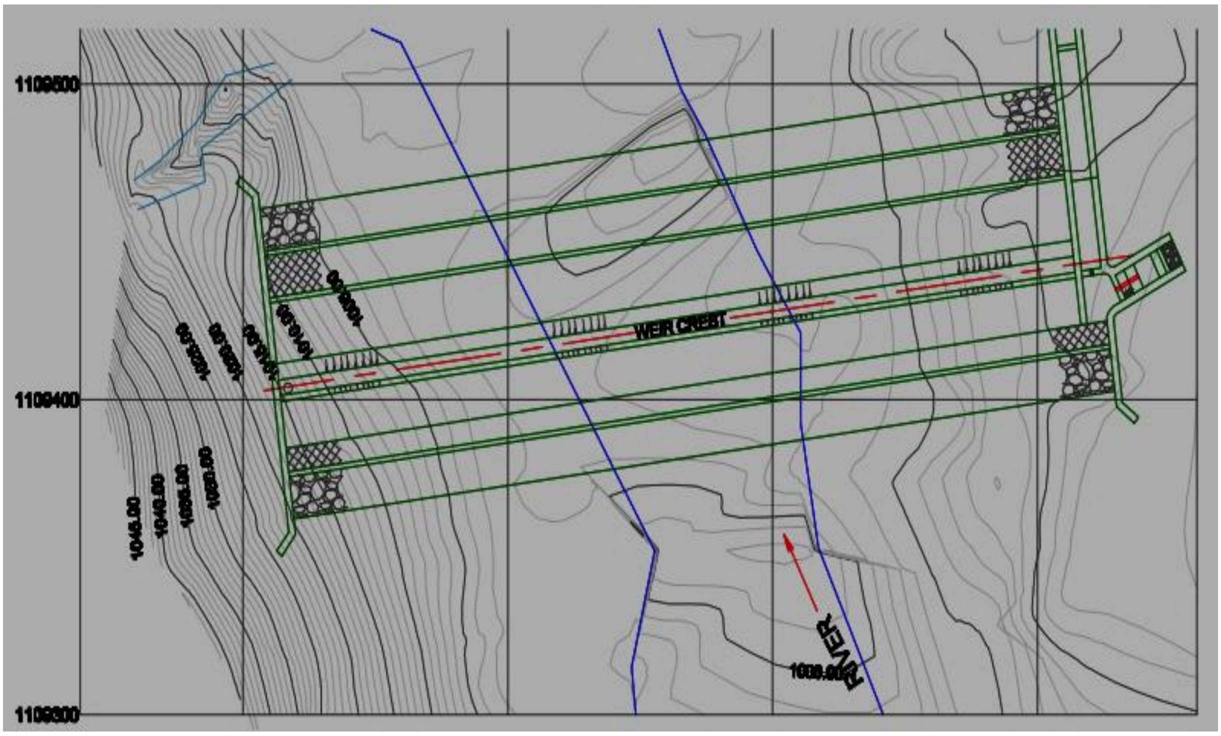


Figure 3-2: General layout of dabus dam (ECDSC)

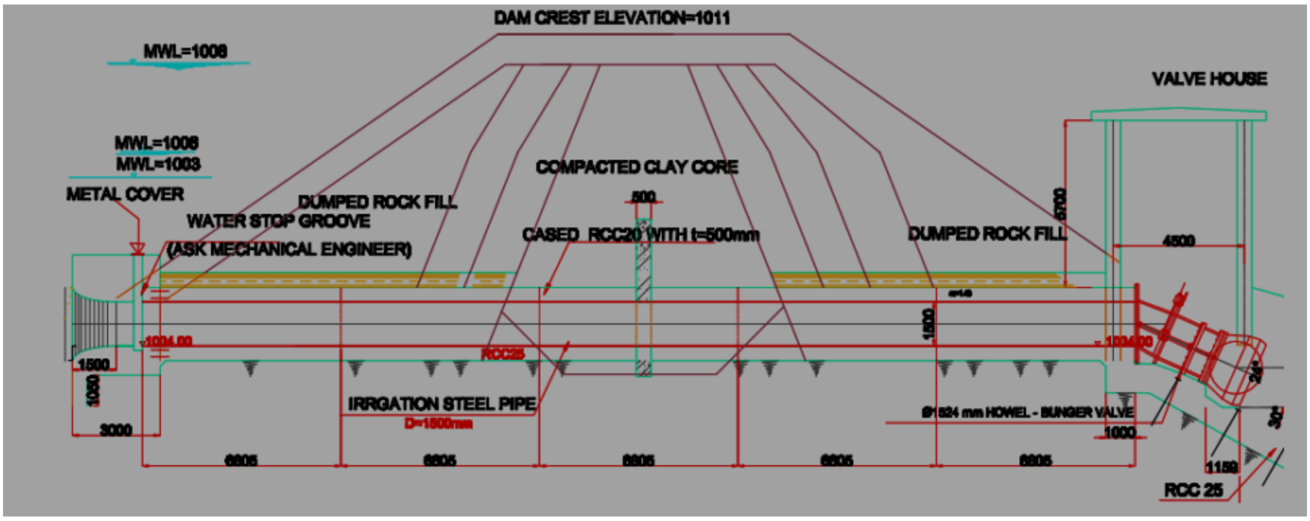


Figure 3-3: Dabus Dam cross-section and zoning (ECDSC)

3.4 Data Collection

Before undertaking and processing of any research data it is imperative to make a tough search for the data. Therefore, the primary assignment of the study was getting relevant information and data of the study area. This section identifies and discusses the types and source of data required for the study, and their analysis.

3.4.1 Topographic Data

Topography is mandatory in order to map the flood inundation area and also to analyze the river geometry and cross section at different reaches. This data can be obtained by either manually surveying the field or by DEM (digital elevation model). Digital elevation model is a digital file consisting of terrain elevations for ground positions at regularly spaced horizontal intervals This model is prepared in the united states and the 30*30 DEM is now available Its uses range from scientific, industrial, engineering and even to military application.

Manual surveying is most of the time done on a specific small area for the purpose of detail investigation or design. Due to its high capital requirement as well considerable time demands, manual surveying is nowadays compromised by satellite image readings.

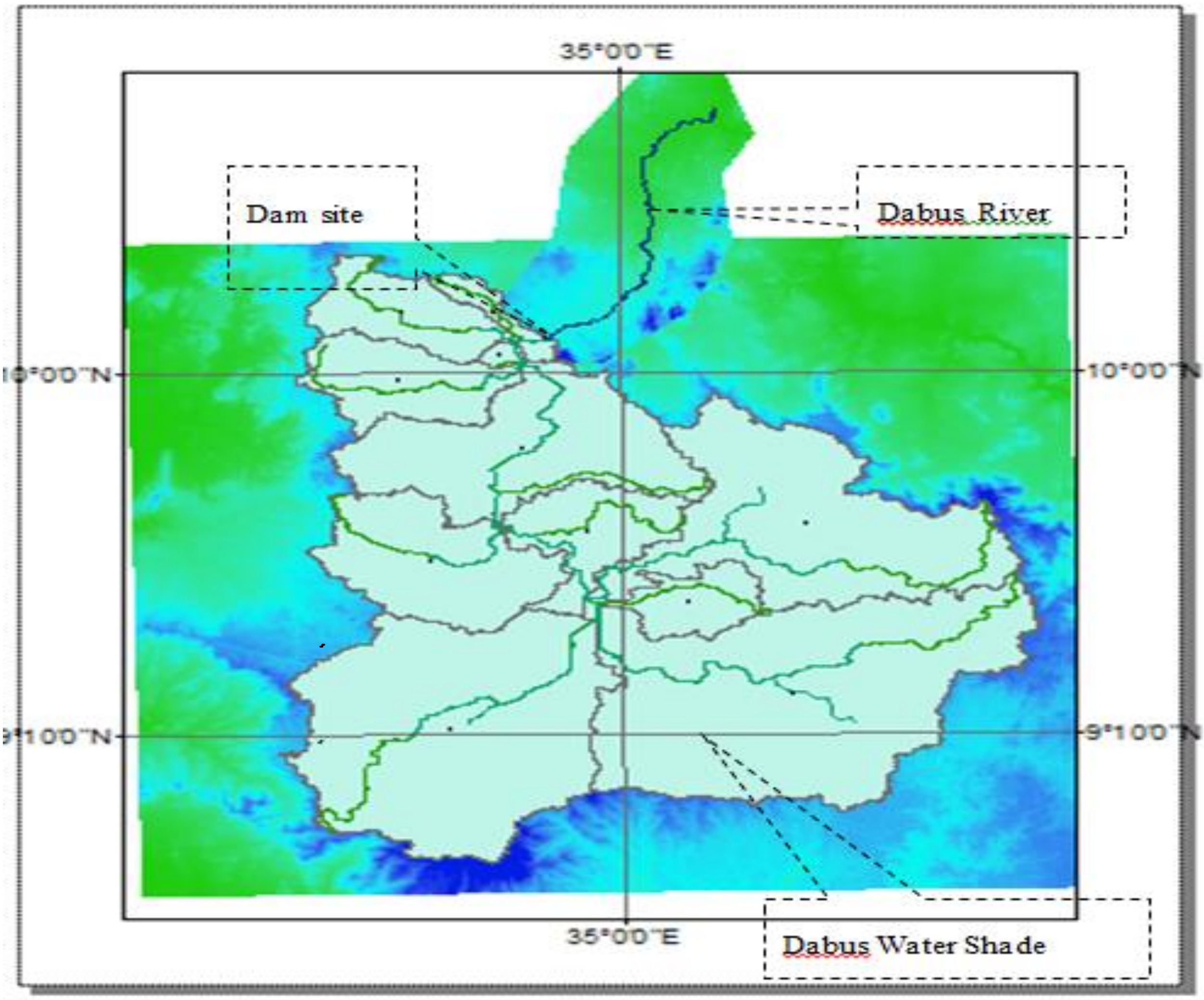


Figure 3-4: DEM representation ofDabus Dam, water shade and downstream catchment area.

3.4.2 Hydrology

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

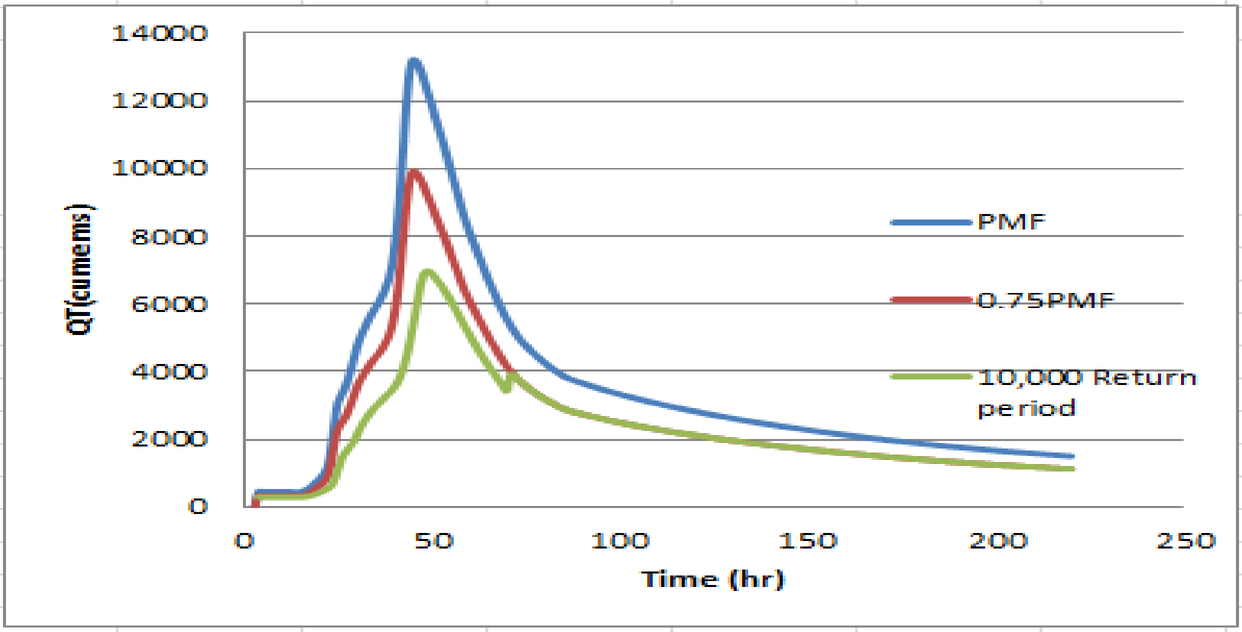


Figure 3-5: the inflow hydrograph to Dabus dam for various return periods (WWDSE)

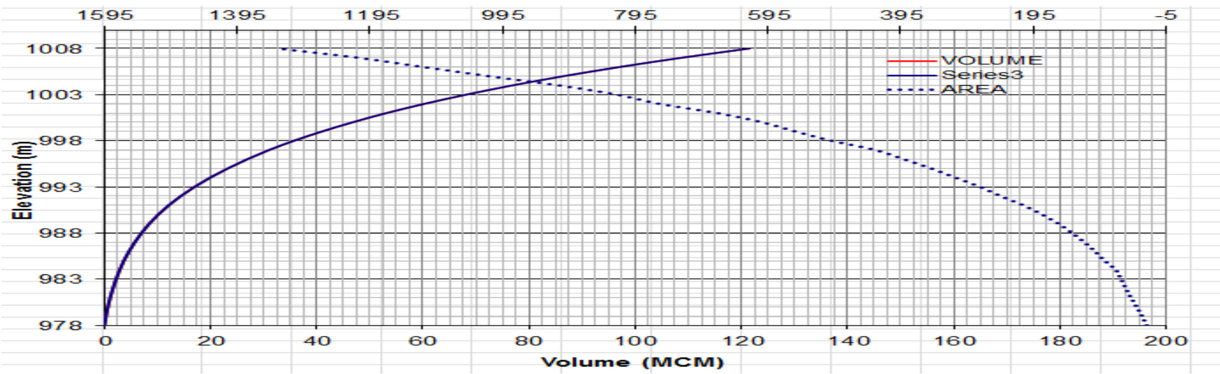


Figure 3-6: Capacity-Area –Elevation Curve of Dabus Reservoir (WWDSE)

3.5 Downstream Command Information

According to the irrigation study of WWDSE, the command area of Dabus lies on the right and left bank of the Dabus River, which is the boundary of the two project weredas; Menge and OdaBildiglu. The project area is part of Abbay River Basin with geographic coordinates of 10^{O} 02’ to 10^{O} 14’ Northing and 34^{O} 51’ to 35^{O} 02’ Easting between altitudes of 850 to 1,100masl.It is expected to develop about 10,000ha of land through Dabus dam construction and the local communities would be the beneficiaries of the project.

There is no upstream user of DabusRiver for any purpose and the distance between the inundation of renaissance dam and the end of this project command area is 22km. almost all suitable area for irrigation is tried to be captured. It is from the above facts and the interest of the client that all possible gravity based command area tried to be accommodated. As the project is planned to be used by small holder farmers maximum effort is made to avoid complexity up on operation and maintenance. The topography of the project area is mainly plain and undulating land. According to the survey data the elevation range between 850 and 1100 masl and the slope range between 0% and 8%.

3.6 Population at Risk

According to the BoFED the population of the Region in 2011 was 938,996, however, the total of which 50.8% were male and 49.2% were female. The population of this regional state has been growing at a rate of 3% per annum during 1994 to 2007 which is higher than the national population growth rate of 2.6% per annum. Population density is low with an average of 12.1 per km2.the population of two woredas i.e. people living in Mengieworeda at the left bank and OdaBildigilu woreda at the right bank are the main which may be affected by this dam breach. The survey done by water works design and supervision enterprise (WWDSE) shows that the size of people living on the left and right bank of the river is 47,076 people for MengeWereda and 1,666.76 people for OdaBilidigluWereda. It touches Fundudu, Shaga and Signor of MengeWoreda and Belfudi, Gambella Dale and Zumbakebeles of Oda Bildigilu woreda. since all the above people are found around left and right bank of the river ,they are considered as population at risk. There is also irrigation command area found along left and right bank of the river which could be affected by this dam breach. This can be displayed in map below.

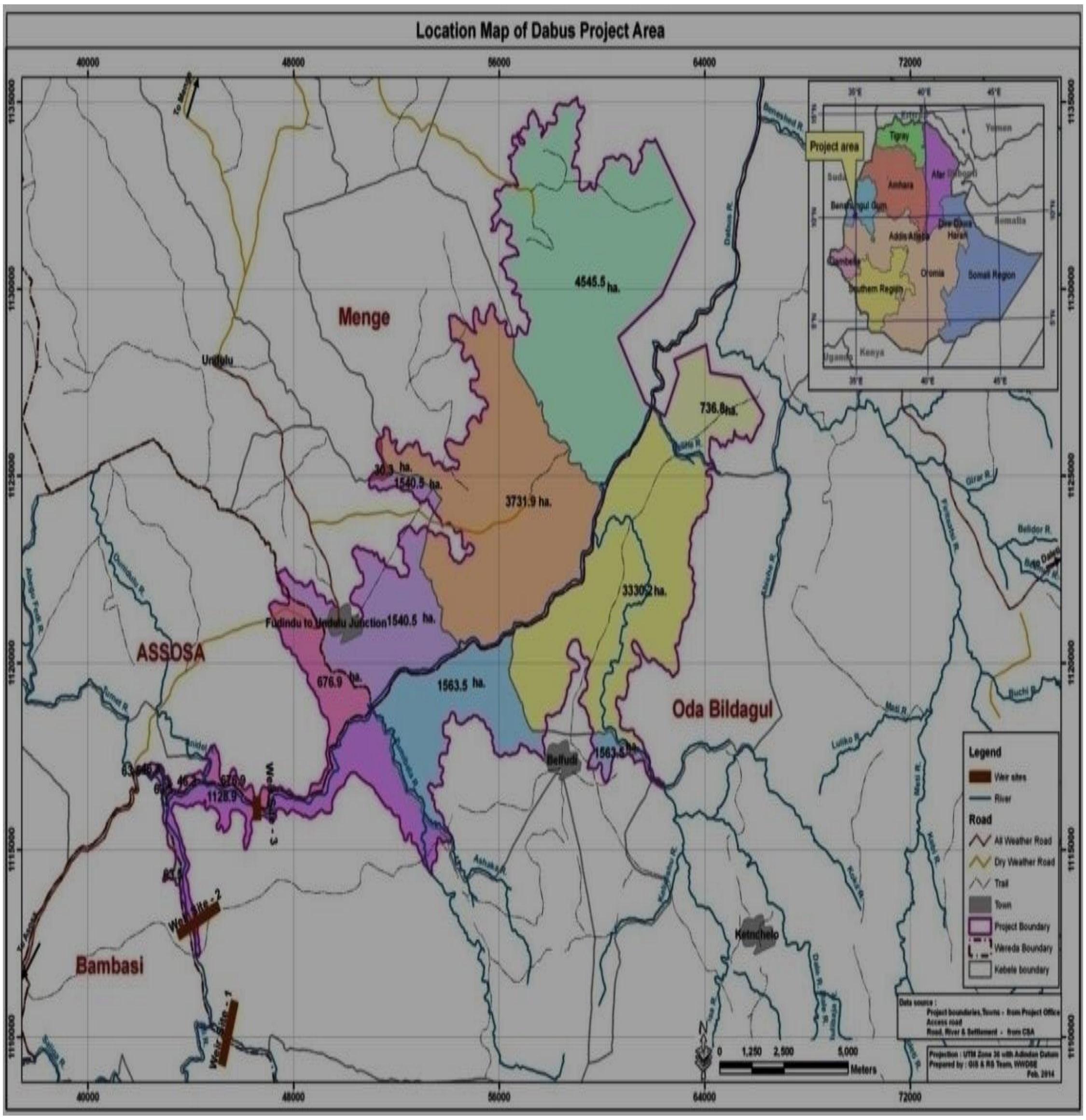


Figure 3-7: Location Map of the downstream (WWDSE)

3.7 General Process of Dam Breach Modeling

The procedures followed in any dam breach modeling and inundation mapping process is as follows. In this thesis, thefour famous and widely accepted empirical equations were selected to determine breach parameters. The breach parameters are computed using the selected empirical equations and comparison among results is done. The stream centerline andriver bank line was traced Using Google earth for use in hec-georas. Develop Hec-Georas project in Arc map to extract downstream channel geometry, to define inline structure, lateral structures (if any), manning's coefficient, to set stream lines and to define reaches. Export the Hec-Geo Ras results to Hec-Ras for further analysis. Import hec-georas results and perform unsteady state analysis by inserting the required hydrologic inputs. The breach outflow is then routed through downstream reaches to calculate hydraulic properties at critical locations. Finally exporting the model results to Arc-Map the flood inundation will be mapped to analyze the affected critical locations.

The watershed of Dabus catchment is delineated using HEC-Geo-Hms, and the unit hydrograph is computed. This dam is modeled for overtopping and piping modes of failure. For overtopping failure mode, the failure location is assumed at centerline of channel riverbed, whereas for piping mode of failure location is assumed to be at an elevation of 1003 where settlement is expected to form seepage line along the periphery of the outlet's external body and the dam.

These procedures are depicted on the conceptual framework given in Figure 3-8 and discussed in subsequent chapters.

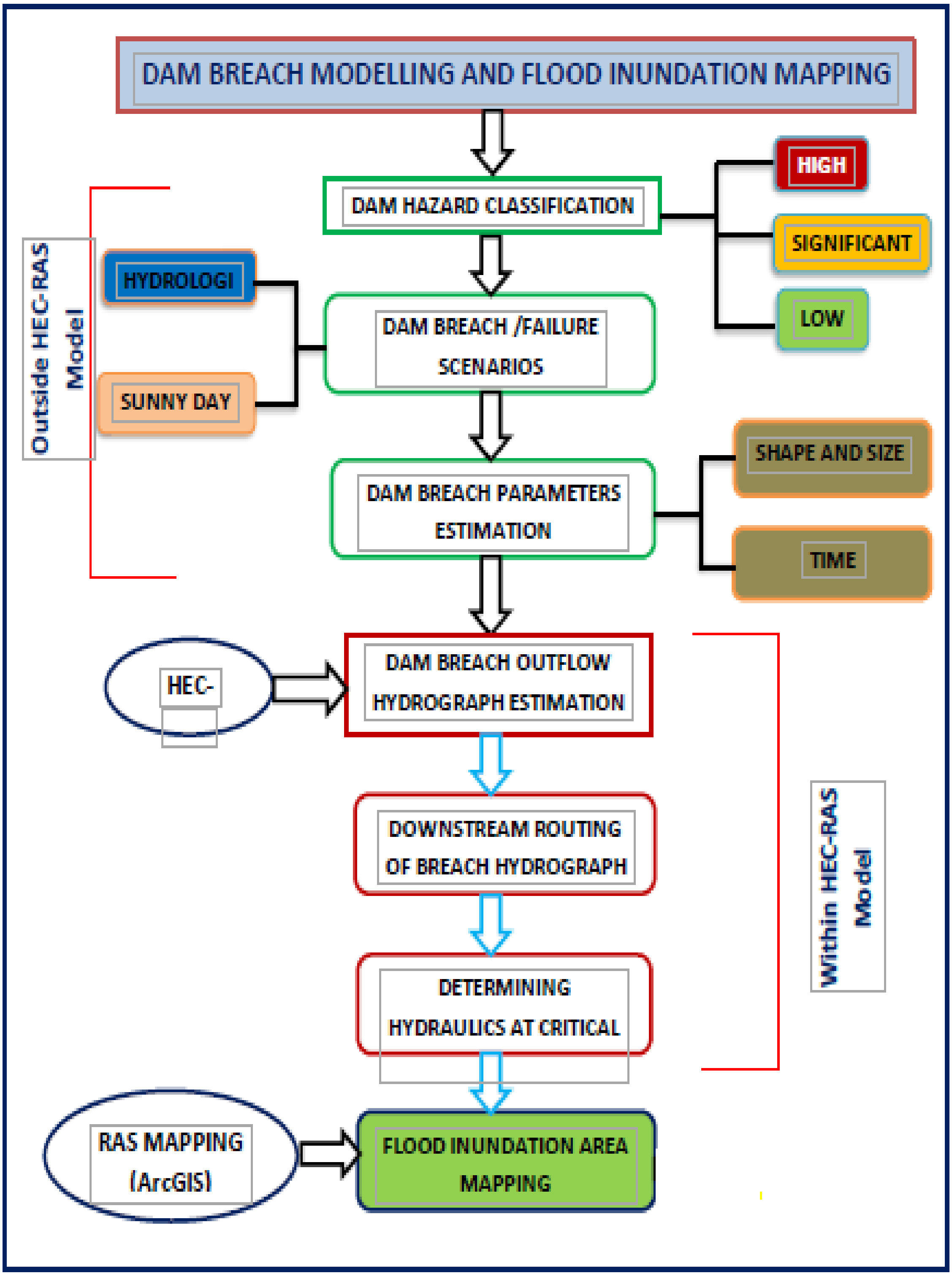


Figure 3-8: Flow diagrams of General Dam Breach Modeling and Flood inundation Area mapping.

3.7.1 Breach Parameters Estimation

These are the parameters pertaining to the breach such as the width, height, and geometry of the breach and the time of failure. HEC-RAS 5.0 is able to estimate the breach parameters given top of dam elevation, pool elevation at failure, breach bottom elevation, pool volume at failure, dam crest width, u/s and downstream slopes and type of dam material.

The salient features of the dam which are used in the process of breach parameters are listed below.

- Dam type……………………….Rock fill
- Dam crest elevation…………….1011m.a.s.l
- Crest length ……………………. 760m
- Crest Width …………………… 9.1m
- Dam height above the river bed level……… 37m
- Normal Water Level……… 1006m.a.s.l
- Upstream face slope……….2.5H:1V
- Downstream face slope……2.5H:1V
- Maximum Water Level …1008m.a.s.l
- Reservoir capacity (NWL) ……. 121.48Mm3

3.7.2 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 5.0) was used. In addition to this hydraulic modeling , Arc Map 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 10.2.2 was used. Arc Map is the main component of ArcGIS which is geospatial processing software. In this paper, most of the ArcGIS tasks were performed using other Arc Map extension tool HEC GeoRAS. This GIS extension tool was used to prepare a reliable model input for hydraulic model, HEC-RAS 5.0 within theArc Map softwareenvironment. In addition to this, HEC -GeoRAS was used to visualize hydraulic modeling results in the form of inundation depth maps.

3.7.3 Hec-Geo Ras Development

The modeling domain for the purpose of this thesis study is composed of Dabus dam and its reservoir at the upstream boundary, the downstream most cross-section of DabusRiver up to joining of the Abbay River.

Hence, the modeling reach is made up of 3kms upstream of the dam, 83Kms downstream of the dam there are several Agricultural Villages, and ruler towns located in the vicinity of the downstream of the dam. The following figure presents the HEC-RAS model setup for case study dam and settlements under flooding risks.

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.10which is compatible with ArcMapv.10.2.2 is used in this study in creating RAS Layers in ArcGIS to extract information essential for hydraulic modeling (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 centerline 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.

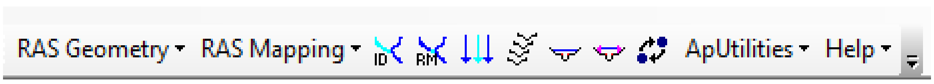


Figure 3-9: HEC-GeoRAS Tool Bar Used in ArGIS v10.2.2

Prior to performing hydraulic computations in HEC-RAS, the geometric data must be imported and completed and flow data must be entered. Once the hydraulic calculations are performed, exported water surface and velocity results from Hec-Ras may be imported back to the GIS using the Geo-Ras for spatial analysis.

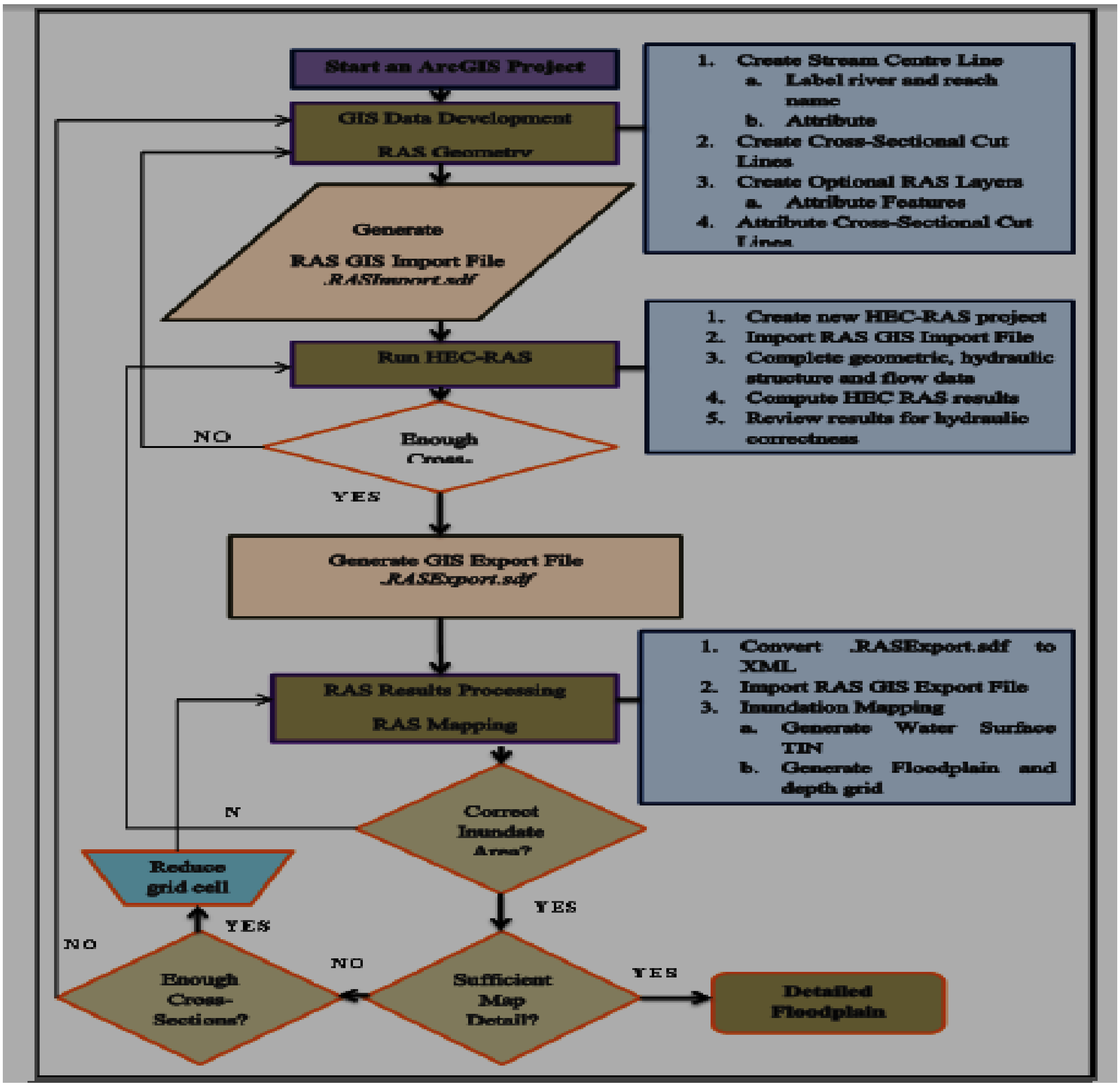


Figure 3-10: Process flow diagram for using HEC -GeoRAS (Adapted from: HEC -2009) (Ackerman, 2009)

3.7.4 Layers Development in HEC-GEORAS

The stream centerlines layer, bank lines layer, flow path layer, cross sectional cut line layer were created at the beginning. The development of all other layers is optional based on the data needs for the river analysis model. The layers which were created in HEC-RAS and their definition is tabulated in the table below.

RAS Layers	Description
Stream centerline	Used to identify the connectivity of the river network and assign river stations to computation points.
Cross-sectional cut lines	Used to extract elevation transects from the DEM at specified locations and other cross-sectional properties.
Bank lines	Used in conjunction with the cut lines to identify the main channel from overbank areas.
Flow path centerlines	Used to identify the center of mass of flow in the main channel and overbanks to compute the downstream reach lengths between cross sections.

Table 3-2: summary of RAS layers definition

Among these four layers the flow path layer need to be identified as being in the “left” overbank, “right” overbank or main “channel” and the stream centerline should be given river code and reach code. Around832 cross sections were taken at an approximate interval of 100m. Elevation data then were extracted from DEM and imported to HEC-RAS. As a rule each cross-section should be perpendicular to flow direction, and can intersect the main channel only once and may not intersect another cross section. The cross section cut lines should be long enough for the flood to attenuate and to include points of interest.

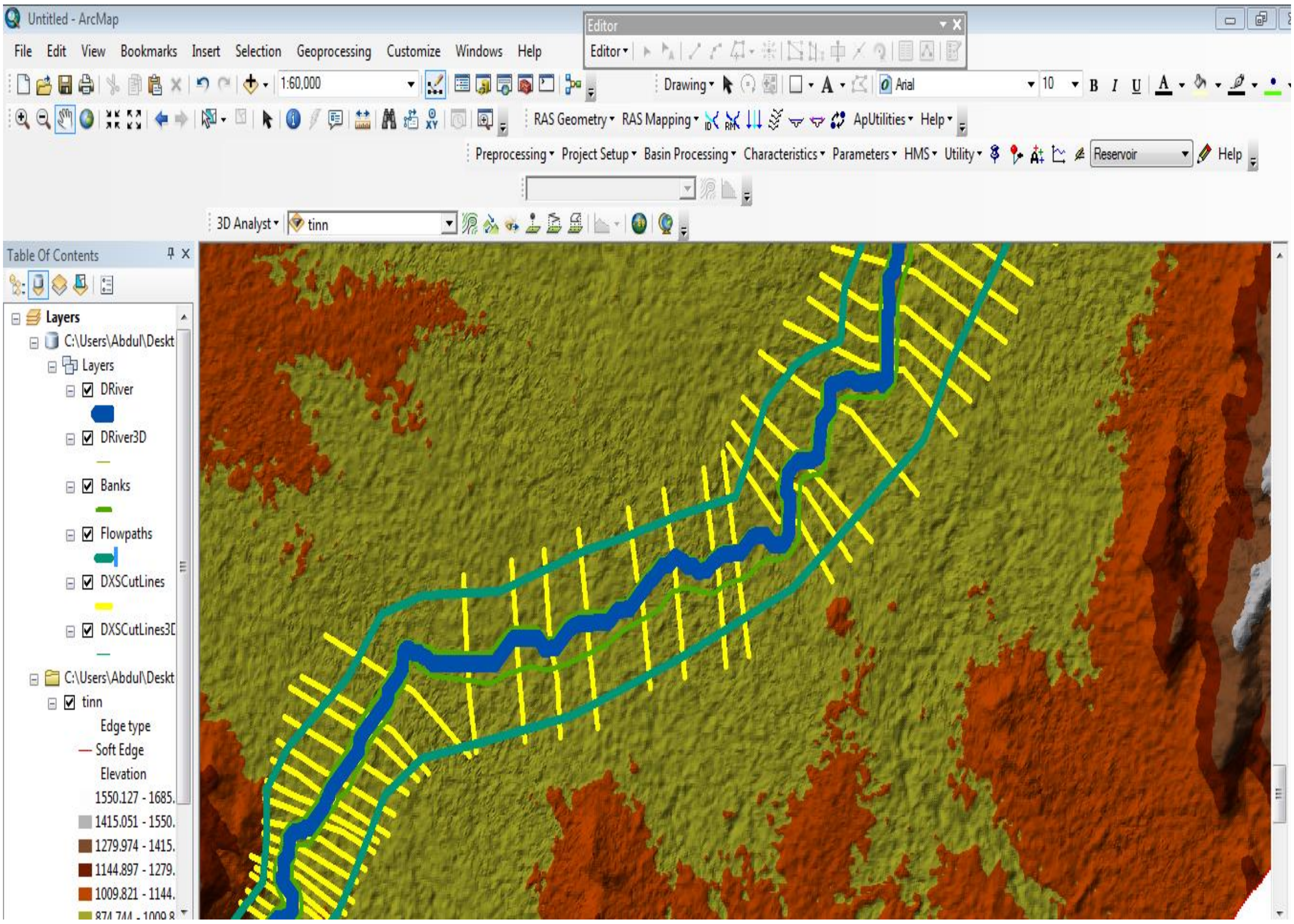


Figure 3-11: Dabus River Geometry developed byHEC-GeoRAS tool in ArcGIS v 10.2.2

After creating the RAS layers and the map, the stream centerline and cross sectional cut line attributes were completed by running Geo-RAS menu items. By running this items the topology, length/Stations and elevations are extracted and saved for stream center lines and River/reach names, stationing, bank stations, downstream reach lengths and elevations are extracted for the cross sectional cut lines.

The HEC-GeoRAS development is completed by generating the RAS GIS import file. This is done by simply exporting the RAS data from the menu item.

3.7.5 Manning's Roughness Coefficient

Manning's coefficient n is used to describe the resistance to flow due to channel roughness caused by sand/gravel bed-forms, bank vegetation and obstructions, bend effects, and circulation-eddy losses and so on. In unsteady state river routing simulation, results were often very sensitive to the Manning n values. Selection of the Manning n is aimed to reflect the influence of bank and bed materials, channel obstructions, irregularity of the river banks and to minimize potential biasness of the results.

Direct determination of the roughness coefficient is almost impossible in studying natural river flows, includingunsteady channel network flows. 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 dense woodland, wooded grassland, open woodland, natural forest cover, riverine forest, settlement, shrub land and open grassland (WWDSE).

In addition to this the Google earth view of the downstream reaches is significantly green indicating considerable bushes and/or trees are found on the flood plain.

Depending on these characteristics of flood plain, the manning's n values for the stream channel downstream of the Dabus 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 0.035 to 0.1 reflecting riverine forests,Savannah Grass land, modera tely cultivated, etc.

Figure 3-12: The main Land cover and land use of Dabus River downstream of the dam

3.8 HEC-RAS Development

3.8.1 General

For this thesis, HEC-RAS ver5.0 is used where the software is able to perform;

- ✓ Steady flow water surface profile computations
- ✓ One dimensional and/or two dimensional unsteady flow analysis
- ✓ Quasi unsteady or fully unsteady flow movable boundary sediment transport computations
- ✓ Water quality analysis

All of the above uses common geometric data representation, common geometric and hydraulic data requirements. This version of HEC-RAS also has the capability of modeling dam breach events under a wide range of scenarios. Cross sections, stream center line and other geometric features of the stream were extracted from GIS using HEC-GeoRAS and Arc-GIS. Dam failure scenarios were analyzed for piping and overtopping (hydrologic and non-hydrologic events). The objective of this modeling effort is to evaluate different catastrophic events, predict the outflow hydrograph just at the outlet of breach for the selected breach parameter result and to route the peak outflow hydrograph on downstream channel and map the area that shall be flooded. In this study the one dimensional unsteady flow of HEC-RAS model is used for downstream dam breach outflow hydrograph routing along the river.

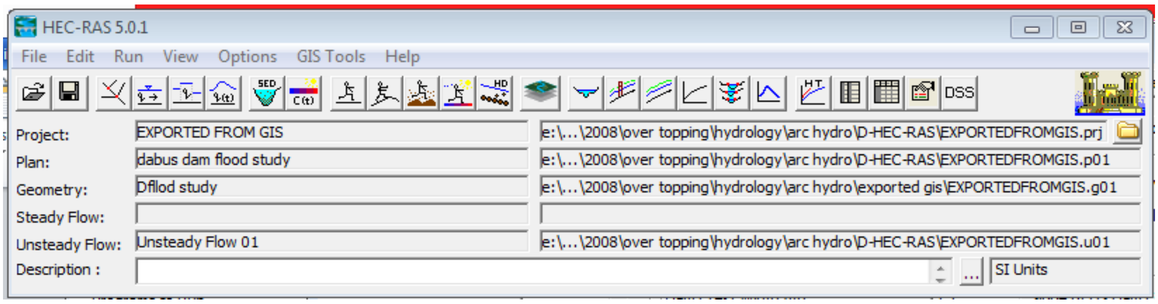


Figure 3-13: HEC-GRAS software v15.0.1

3.8.2 Failure Initiation Criteria

There are two modes of failure, i.e. sunny day (piping or non-hydrologic) and overtopping (hydrologic) failure. The objective of this thesis paper is to set scenarios for both of the cases and to identify the most catastrophic flood event among the scenarios and to map this flood in the downstream reaches.

In piping failure cases, the failure initiation criteria is selected as a critical WSEL of 1003 m, which is equal to Dabus dam's out let level. Where as in the over topping case, the failure initiation criteria was selected as an elevation of 1016m which is over top the dam.

3.8.3 Boundary Conditions

For the solving of the unsteady flow equations in HEC-RAS, boundary conditions need to be defined. There are two external boundary conditions for the Dabus 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 hydrograph. This hydrograph is the regular flood event for piping and the PMF. Downstream boundary condition for Dabus is set to normal depth with a slope of 0.001 for both overtopping and piping scenarios.

3.8.4 Initial Conditions

In addition to boundary conditions, a modeler must establish initial conditions of the system at the beginning of unsteady flow simulation. As per HEC-RAS user's manual (HEC, 2016), the initial conditions consist of flow and stage information at each of the x-section, as well as elevations for any storage areas defined in the system.

HEC-RAS user's manual (HEC, 2016) suggests selecting arbitrary values of initial water surface elevation in such a way that its value will be between the possible maximum and minimum reservoir operation level and to use an initial flow value equivalent to base flow hydrograph of the modeling reach at each cross-sections.

Accordingly, the Author has used an initial water surface elevation value of 500m (which is between 1015m/FRL and 396m/MDDL) for Dabus dam reservoir; and initial flows of 249m^3/s during both piping and overtopping. Initial flows of all the remaining water surface elevations and flows at each x-section of modeling reaches were then calculated by the model.

3.8.5 Cross Sections

HEC-RAS requires cross sections for water surface computation. Approximately 832 cross sections were created for the model using HEC-GeoRAS at about 100m 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 3-9

3.8.6 Downstream Flow Routing and Flood Mapping

The outflow hydrograph resulted from the breaching dam is routed through the downstream river channel and flood plain and different maps of the resulting water surface extent, water depth and velocity are produced. The models HEC-RAS and HEC-GeoRAS are used integrally to extract topographic data, analyze the flow hydraulics and finally to generate the maps.

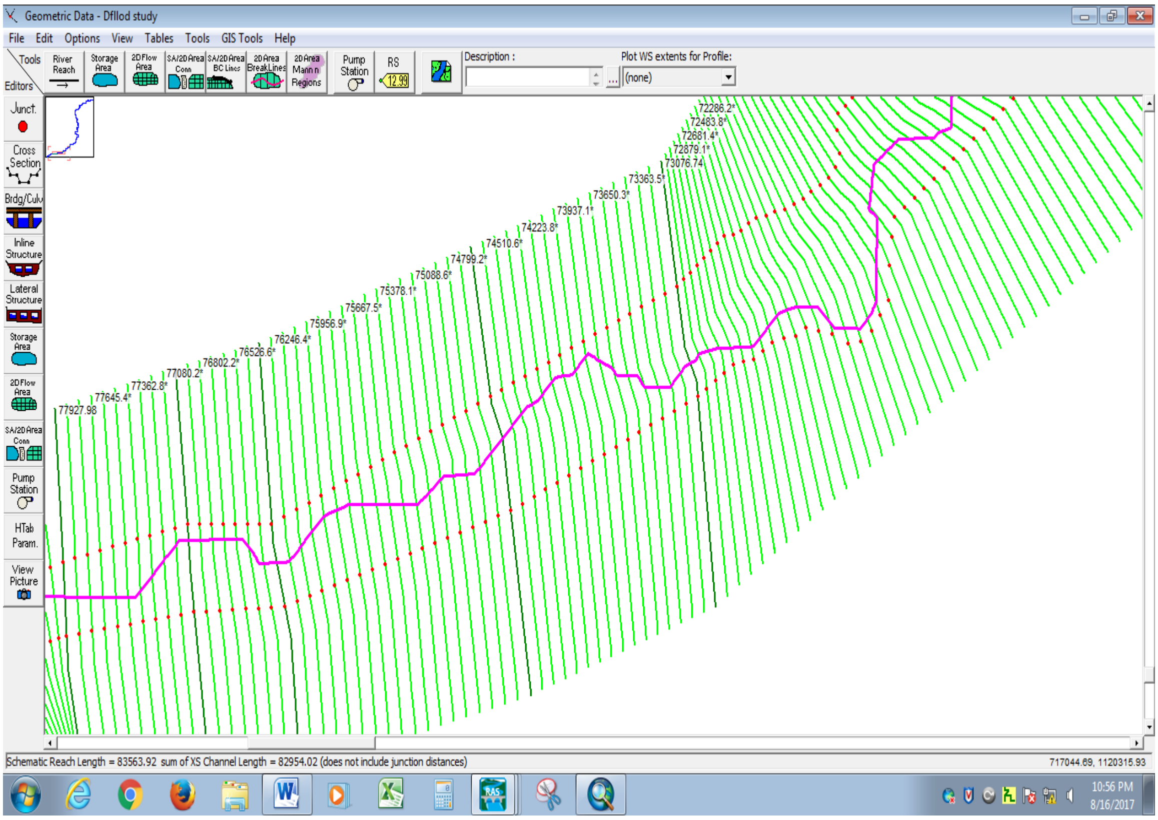


Figure 3-14: exported map from HEC-GeoRAS to analyze the flow hydraulics

3.8.7 Overtopping Failure

Dabus Dam has been tested for overtopping failure applying a 10,000 year return period inflow while the reservoir is at full condition i.e. when upstream water level is at spillway crest level. During simulation process the reservoir is observed to rise almost up to dam crest level and immediately after the peak flow of the10, 000 years return period flood passes it recedes back to its initial level. From this it can be concluded that the spillway can safely evacuate the incoming 10,000 years return period for which it has been designed.

Then the dam is checked for PMF is found to overtop the dam and reason for consequent failure.

3.8.8 Piping Failure

Piping through embankment body may be formed due to excessive seepage that may be triggered when the impervious core material used is poor in quality or when the construction procedure does not follow engineering standards.

4. Result and Discussion

4.1 General

To investigate a potential risk ofDabusdam-failure induced flood to the settlements in the downstream reaches, the modeling domain was tested for two scenarios under overtopping and piping failures. These scenarios are simulated using breach parameter results from four types of methods namely; MacDonald et al, Frohelich (2008), Von thun and Gillete and Xu and Zhang. The breach parameter results used to come up with over topping and piping failure results.

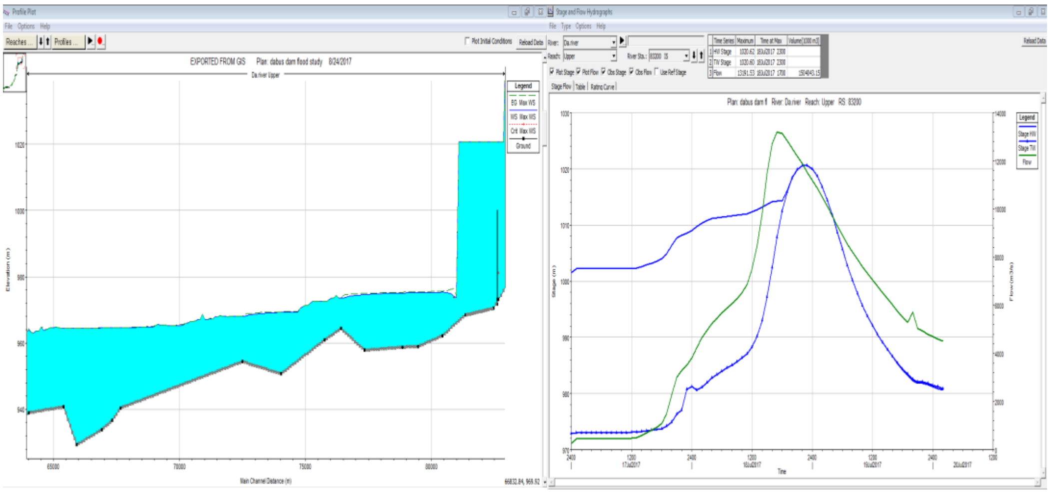
4.2 Dam Breach Parameters Result

The dam breach parameters were estimated using four most currently used regression equations, i.e. MacDonald et al, xu&zhang, VonThun and Gillette, and Froehlich (2008). For the case of overtopping failure, the bottom breach width for four methods were 272m, 171m, 118 and 192m respectively. Their breach development times were 2.86hrs, 2.54hrs, 0.91hrs and 5.41hrs.Similarly, the breach width and breach development times for the above four methods in the case of piping failure were: 272m, 134m, 118m and 112m and 2.86hrs, 2.54hrs, and 0.91hrs and 5.23hrsrespectively.

4.3 Dam Breach Simulation Result

Both overtopping and piping failure modes were checked. When overtopping failure mode is simulated, the 10,000 years return period could be safely evacuated through the spillway and no breaching was observed. Hence, the dam is safe against failuredue to 10,000 return period of flood. The Dam checked again for over topping failuremodeusing PFM and resulted in failure. The maximum breach outflows and time to peak flow of the two failure scenarios for Dabus Dam were computed using the above breach parameter results.

The dam could potentially fail due to piping flow emerged the outletpoint. The piping elevation at which the dam fails giving the most catastrophic flood is 1003m a.m.s.l.



(a) (b)

Figure 4-1: Dabus Dam Breach Results, (a) Water surface profile plot(b) Breach Outflow Hydrograph at Dam

4.4 Overtopping Mode of Failure Simulation Results

The maximum breach outflow and arrival time to peak breach outflow results obtained from HECRAS model simulation at dam for four methods in case of hydrologic (PMF) breach scenario are given in tables below.

method	Breach Peak Flow (m3/s)	time to peak hr(Starting at 17/7/2017, 0000)
MacDonald et al	9702	48
xu&zhang	13016.61	42
Von thun&Gillete	14220	82
Froehlich	8838.36	78

Table 4-1: Maximum pool breach peak outflows for over topping mode of failure

station	peak flow (m3/s)	velocity (m/s	flood heigth (m)	time to peak hr Starting at 17/7/2017, 00:00
at dam	9702	2.1	47	18 July 2017,18:00
1km DS	9597	0.6	30	18 July 2017,20:00
10KM DS	9248	1.61	15	18july 2017,22:00
15KM DS	8722	4.46	13	18 July 2017,22:00
35KM DS	249.75	3.26	3	17 July 2017,00:00
50KM DS	249.75	2.82	2	17 July 2017,00:00

Table 4-2: maximum flow, time to peak, rate of flow and flood height for some Dabus river stations below Dabus dam during overtopping (MacDonald et al), starting time at 17/7/2017, 0000

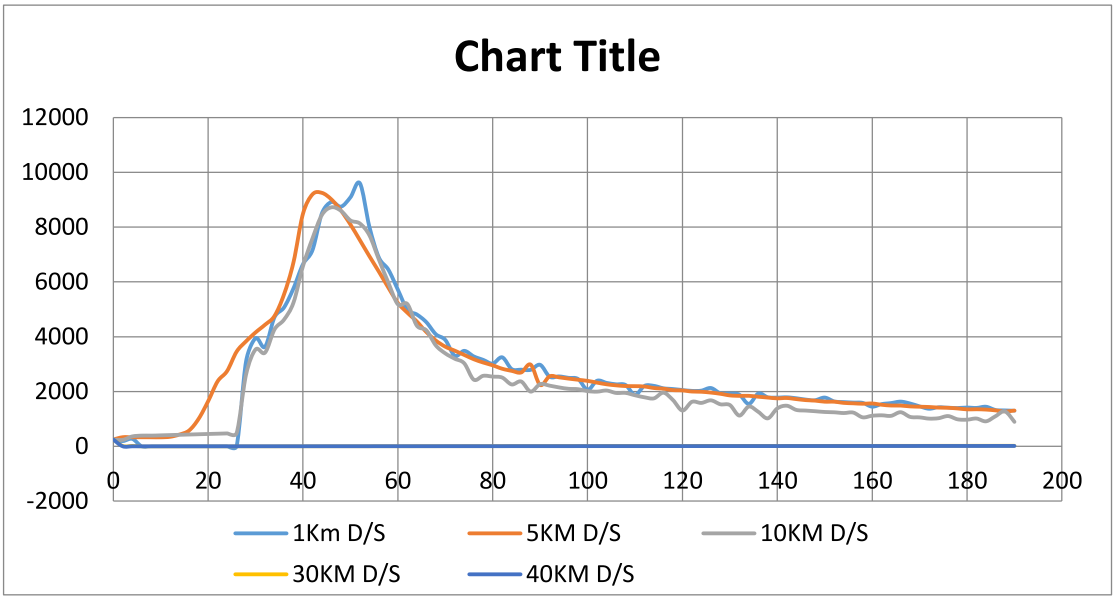


Figure 4-1: Maximum breach outflow hydrographs at selected points, overtopping by Mac Donald et al

station	peak flow (m3/s)	velocity (m/s	flood height (m)	time to peak hr 17/7/2017, 0000
at dam	13196.25	2.41	23	18 July 2017,17:00
1km DS	8468.3	5.61	26	18 July 2017,20:00
10KM DS	8783.31	4.1	15	18 July 2017,22:00
15KM DS	6692.1	3.88	13	19 July 2017,03:00
35KM DS	258.77	3.24	4	20 July 2017,16:00
50KM DS	249.75	2.44	3	17 July 2017,00:00

Table 4-3: maximum flow, time to peak, rate of flow and flood height for some Dabus river stations below Dabus dam during overtopping (Frohelich 2008), starting time at Starting at 17/7/2017, 0000

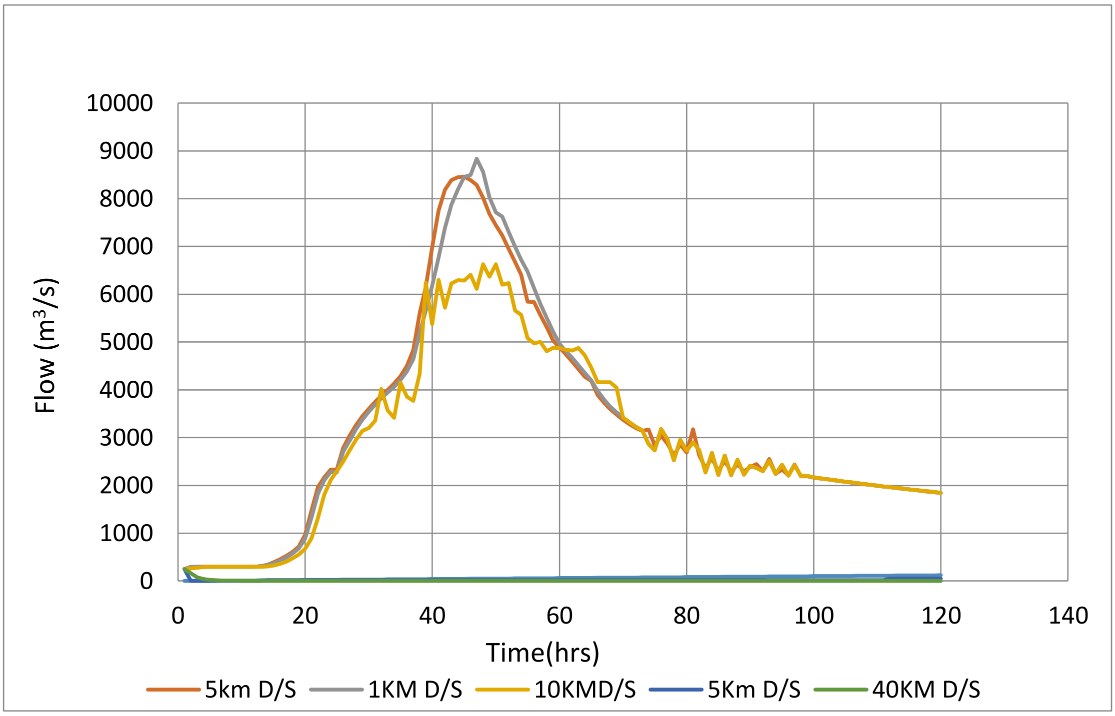


Figure 4-2: Maximum flow hydrographs at selected points, overtopping by Frohelich 2008

Station	Peak Flow (M3/S)	Velocity (M/S	Flood Height (M)	Time To Peak Hr Starting Time 17/7/2017, 0000
At Dam	14220	4.69	19	18 July 2017,15:00
1km DS	13196	0.89	24	18 July 2017,22:00
10km Ds	11941	0.4	18	20 July 2017,09:00
15km Ds	11867	0.51	17	19 July 2017,100
35km Ds	249.75	2.89	3	17 July 2017,00:00
50km Ds	249.75	1.53	2	17 July 2017,00:00

Table 4-4: maximum flow, time to peak, rate of flow and flood height for some Dabus river stations below Dabus dam during overtopping (Von thun and Gillete),startingtime 17/7/2017, 0000

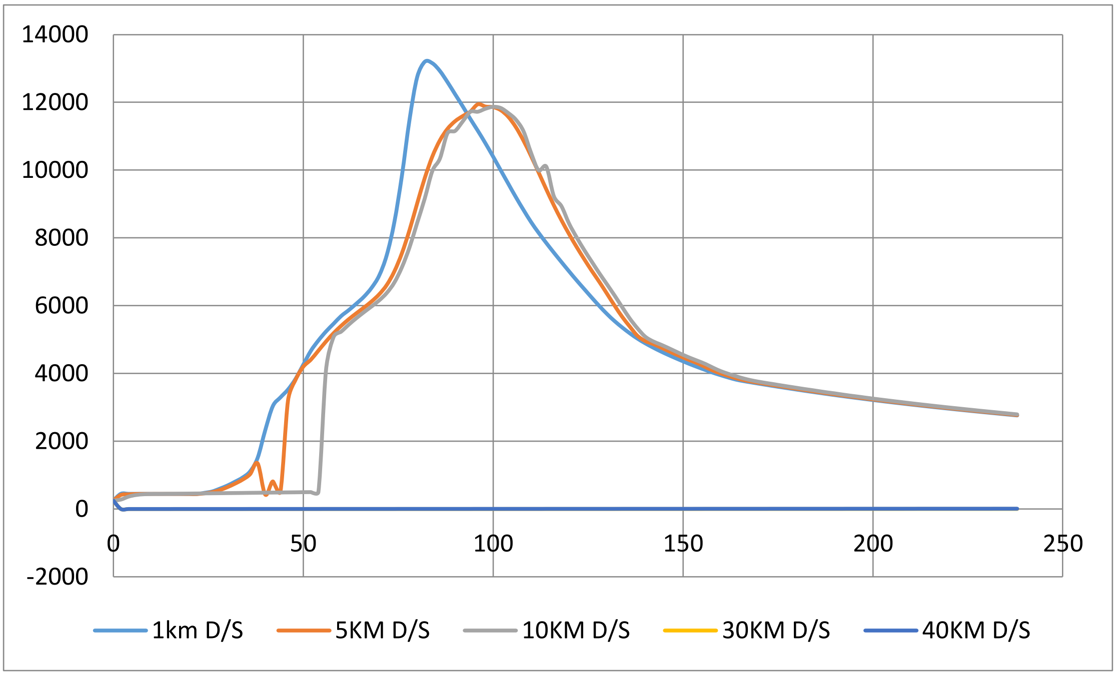


Figure 4-3: Maximum flow hydrographs at selected points, overtopping by Von thun and Gillete

Station	Peak Flow (M3/S)	Velocity (M/S	Flood Heigth (M)	Time To Peak Hr At Starting Time 17/7/2017, 0000
At Dam	13016	1.9	47	18 July 2017,170:00
1km DS	11942	0.6	51	18 July 2017,180:00
10km Ds	11835	3.29	16	19 July 2017,1:00
15km Ds	7928	4.56	19	19 July 2017,1:00
35km Ds	249.75	2.93	3	17 July 2017,00:00
50km Ds	249.75	1.55	2	17 July 2017,00:00

Table 4-5: maximum flow, time to peak, rate of flow and flood height for some Dabus river stations below Dabus dam during overtopping (Xu and Zhang),starting time at starting time 17/7/2017, 0000

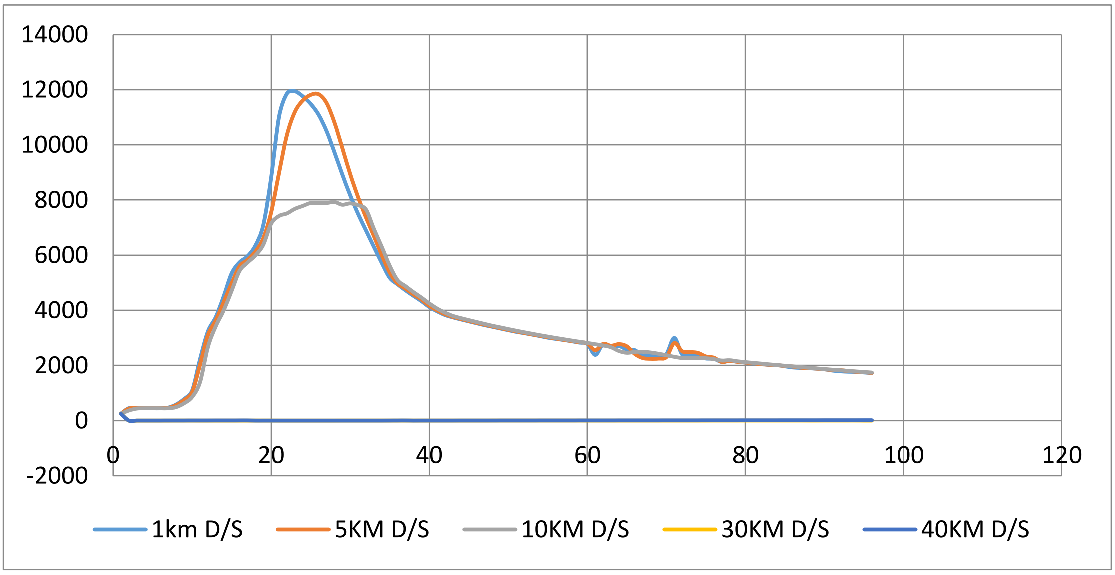


Figure 4-4: Maximum outflow hydrograph at selected points, overtopping by Xu and Zhang

4.5 Piping Mode of Failure Simulation Results

Method	Breach Peak Flow (M3/S)	Time To Peak Hr(Starting At 17/7/2017, 0000)
Macdonald Et Al	12891.53	18 July 2017,17:00
Xu&Zhang	12616.04	18 July 2017,17:00
Von Thun&Gillete	12514.57	18 July 2017,17:00
Froehlich	12717.02	18 July 2017,17:00

Table 4-6: Maximum pool breach peak outflows for piping mode of failure

Station	Peak Flow (M3/S)	Velocity (M/S	Flood Heigth (M)	Time To Peak (Hr) (Starting At 17/7/2017, 0000)
0.335km US	13154.67	4.9	80	18 July 2017,18:00
At Dam	13014.43	0.7	47	18 July 2017,18:00
1km DS	12008.48	0.62	51	18 July 2017,2:00
10km Ds	11831.67	3.3	15	19july 2017,02:00
15km Ds	11833.52	4.5	15	19july 2017,02:00
35km Ds	249.75	3.2	3	17 July 2017,00:00
50km Ds	249.75	1.53	2	17 July 2017,00:00

Table 4-7: maximum flow, time to peak, rate of flow and flood height for some Dabus river stations below Dabus dam; breach by piping (MacDonald et al),starting time at17/7/2017, 0000

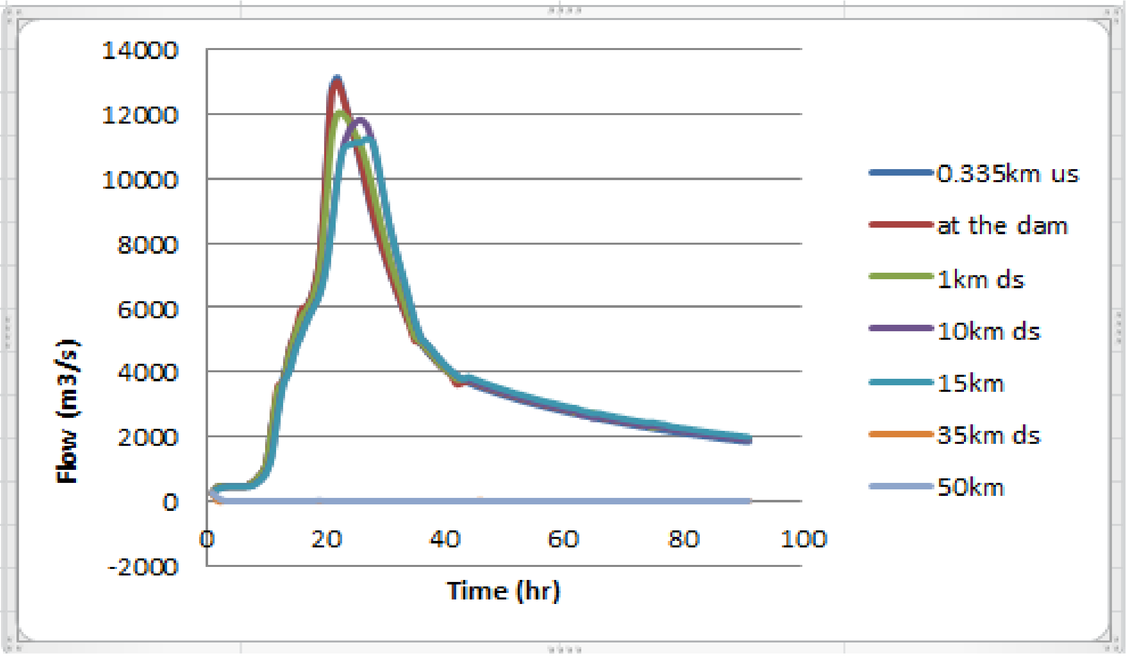


Figure 4-5: Maximum outflow hydrograph at selected points, piping by Macdonald et al

station	peak flow (m3/s)	velocity (m/s	flood heigth (m)	time to peak hr starting time 17/7/2017, 0000
0.335km US	13154.67	4.87	24	18 July 2017,18:00
at dam	12616.04	1.7	47	18 July 2017,17:00
1km DS	12010.28	1.7	51	18 July 2017,20:00
10KM DS	11832.16	3.29	16	19 July 2017,2:00
15KM DS	11447.82	4.58	19	19 July 2017,2:00
35KM DS	249.75	3.26	3	17 July 2017,00:00
50KM DS	249.75	1.53	2	17 July 2017,00:00

Table 4-8: maximum flow, time to peak, rate of flow and flood height for some Dabus river stations below Dabus dam; breach by piping (Frohelich 2008),starting time at starting time 17/7/2017, 0000

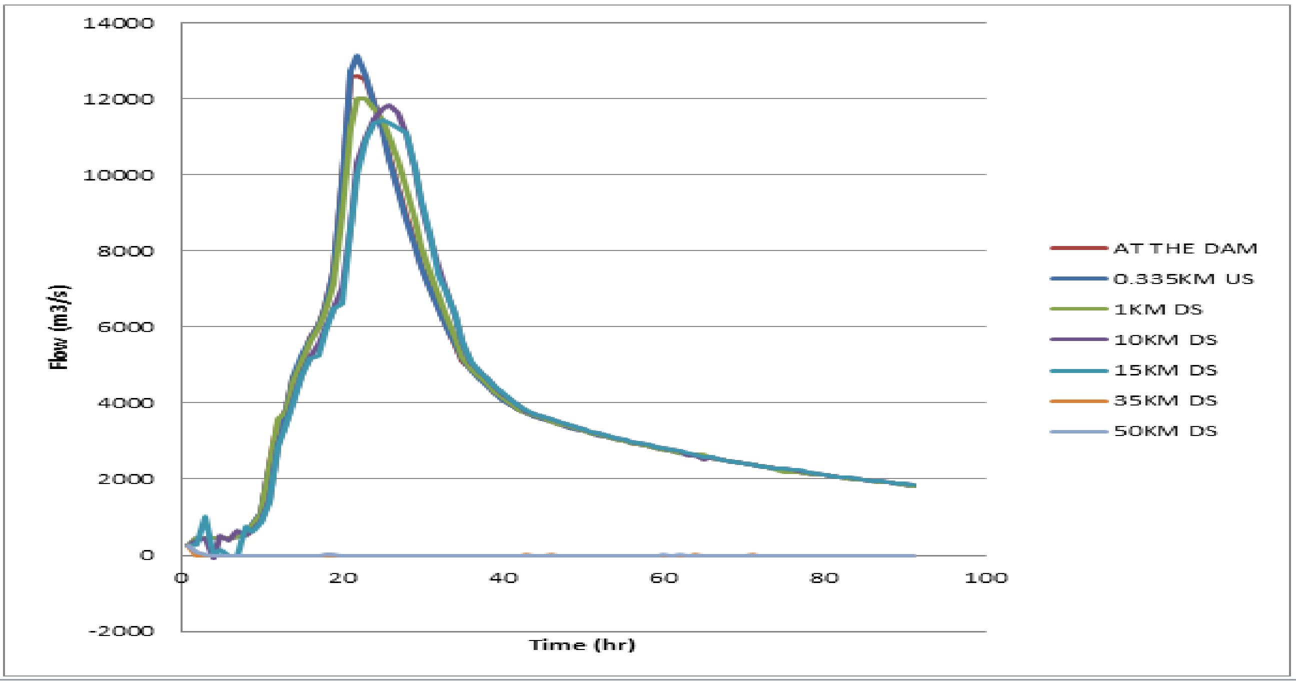


Figure 4-6: Maximum outflow hydrograph at selected points, piping by Frohelich 2008

Station	peak flow (m3/s)	velocity (m/s	flood heigth (m)	time to peak hr starting time 17/7/2017, 0000
0.335km US	13154.67	4.0	42	18 July 2017,18:00
at dam	13016.83	0.54	47	18 July 2017,18:00
1km DS	12008.77	0.62	33	18 July 2017,18:00
10KM DS	12490.34	3.1	15	18july 2017,22:00
15KM DS	11647.97	2.7	18	18july 2017,22:00
35KM DS	249.75	3.2	2	17 July 2017,00:00
50KM DS	249.75	1.55	3	17 July 2017,00:00

Table 4-9: maximum flow, time to peak, rate of flow and flood height for some Dabus river stations below Dabus dam; breach by piping (Von thun and Gillete), at starting time 17/7/2017, 0000

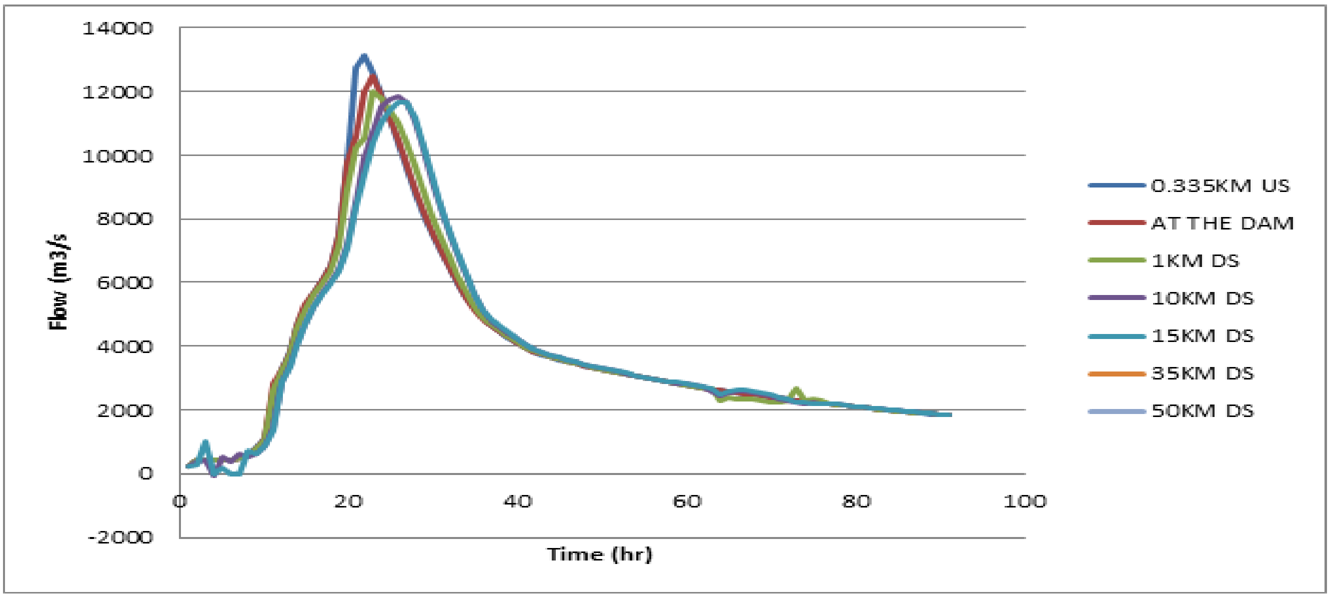


Figure 4-7: Maximum outflow hydrograph at selected points, piping by Von thun and Gillete2008

Station	Peak Flow (M3/S)	Velocity (M/S	Flood Heigth (M)	Time To Peak Hr Starting Time 17/7/2017, 0000
0.335km US	13154.67	2.76	69	18 July 2017,18:00
At Dam	12717.02	0.55	47	18 July 2017,18:00
1km DS	12009.07	0.45	51	18 July 2017,18:00
10km Ds	11829.05	3.3	15	19july 2017,02:00
15km Ds	11530.55	3.3	16	19july 2017,00:00
35km Ds	249.75	1.53	3	17 July 2017,00:00
50km Ds	249.75	3.26	2	17 July 2017,00:00

Table 4-10: maximum flow, time to peak, rate of flow and flood height for someDabusriver stations below Dabusdam; breach by piping (Xu and Zhang), starting time 17/7/2017, 0000

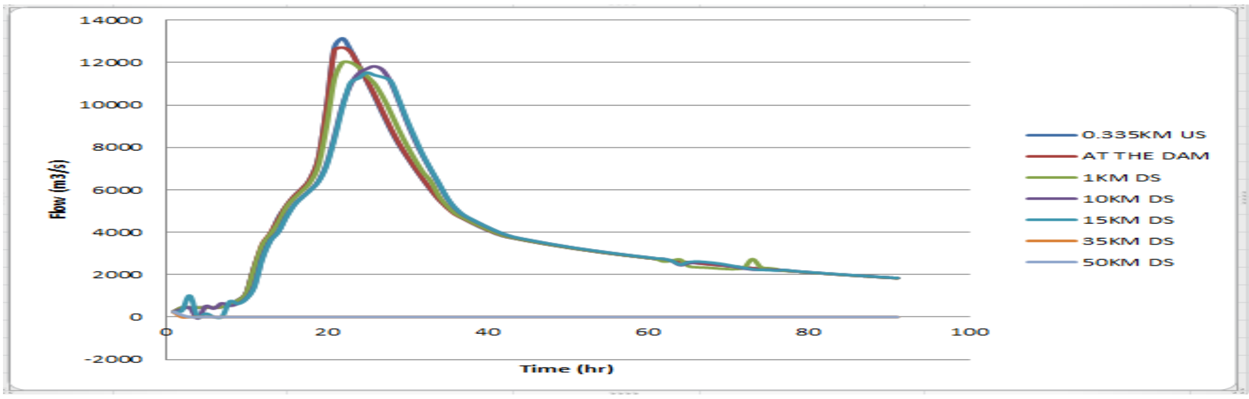


Figure 4-8: Maximum outflow hydrograph at selected points, piping by Xu and Zhang

4.6 Comparative Analysis

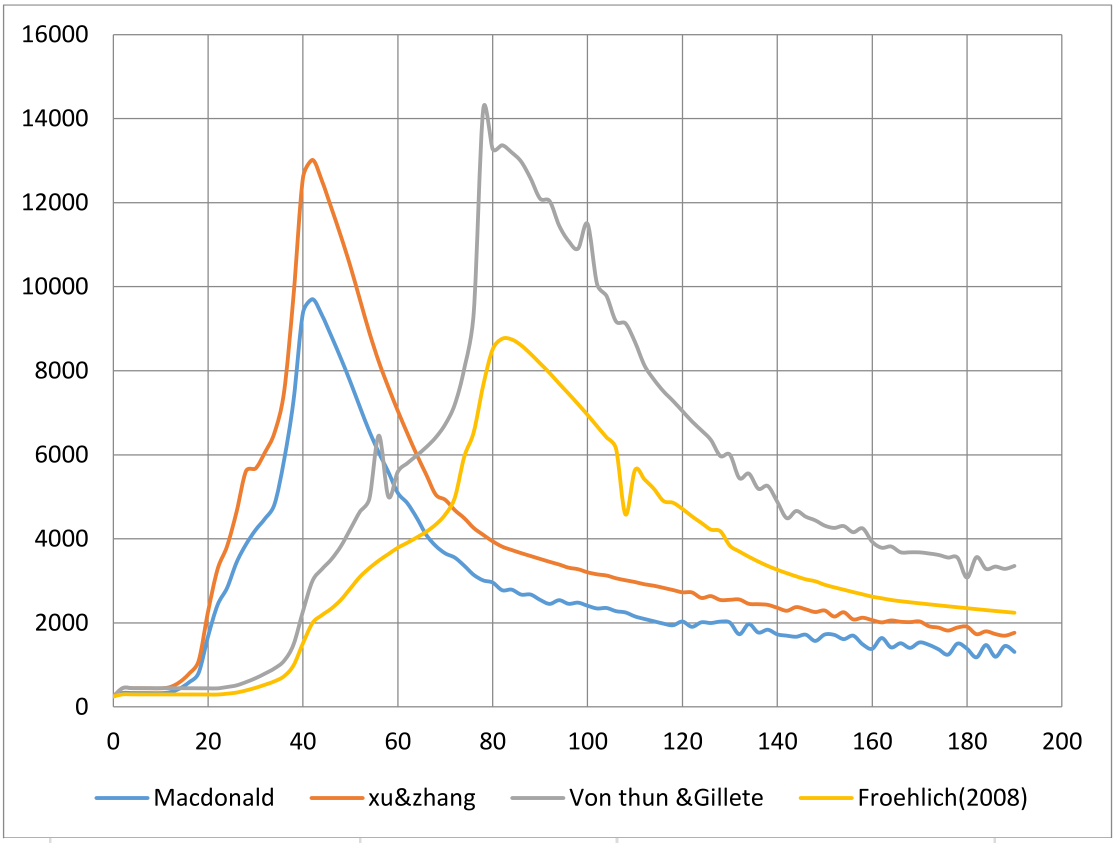
4.6.1 Comparison among Results

The breach Parameters results calculated with four different methods has significant meaning in dam breach modeling. Here below obtained values is tabulated for further use in emergency action plan preparation.

Method	Peak flow (m3/s)	Breach bottom width (m)	Side slopes (H:V)	Breach development time (hrs)
Over topping case				
MacDonald et al	9702	272	0.5	2.86
Froehlich (2008)	13016.61	171	1	2.54
Von thun&Gillete	14220	118	0.5	0.91
xu&zhang	8838.36	192	1.06	5.41
Piping case				
MacDonald et al	12891.53	272	0.5	2.86
Froehlich (2008)	12616.04	134	0.7	2.54
Von thun&Gillete	12514.57	118	0.5	0.91
xu&zhang	12717.02	112	0.62	5.23

Table 4-11: summary of breach parameters and peak flow results

While simulating the breach by the selected four methods the following breach outflow hydrographs are resulted.



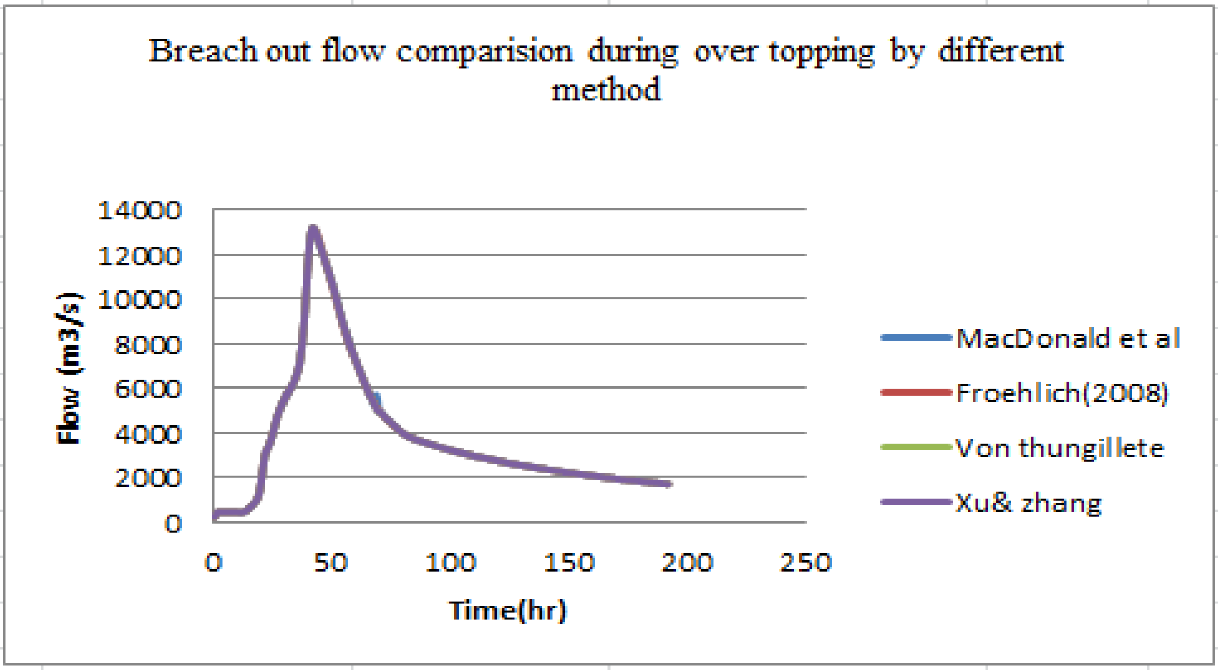


Figure 4-9: *Breach outflow hydrograph comparison during overtopping by different methods*

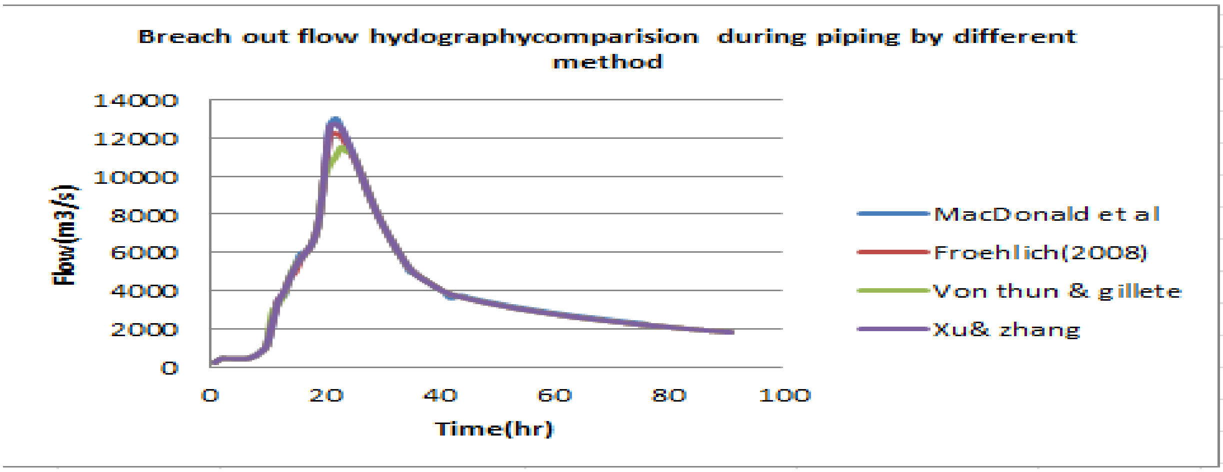


Figure 4-10: Breach outflow hydrograph comparison during overtopping by different methods.

4.6.2 Breach Outflow Comparison with Pre Developed Curve

Many researchers have analyzed historical dam failure data and developed different regression Equations that relate the peak breach outflow with dimensions of height or head (e.g., dam\Height, breach height or depth of water above breach bottom), storage or outflow volume, or a Product of height and volume (Wahl, 1998).

The following Figure is developed based on comparison between peak discharge and height parameters and it comprises of various curves prepared by different researchers and institutes. In the case of Dabus Dam, for Depth of Water 47m at Dam Failurea corresponding peak out flow rate is 14220m3/s in over toping case and 12891.53m3/s in piping cases. The value which corresponds to this height can be read from the graph using Singh and Snorrason's curve. Hence, the peak Discharge from the graph is nearly 15,000m3/s which shows the above results can be reasonably accepted.

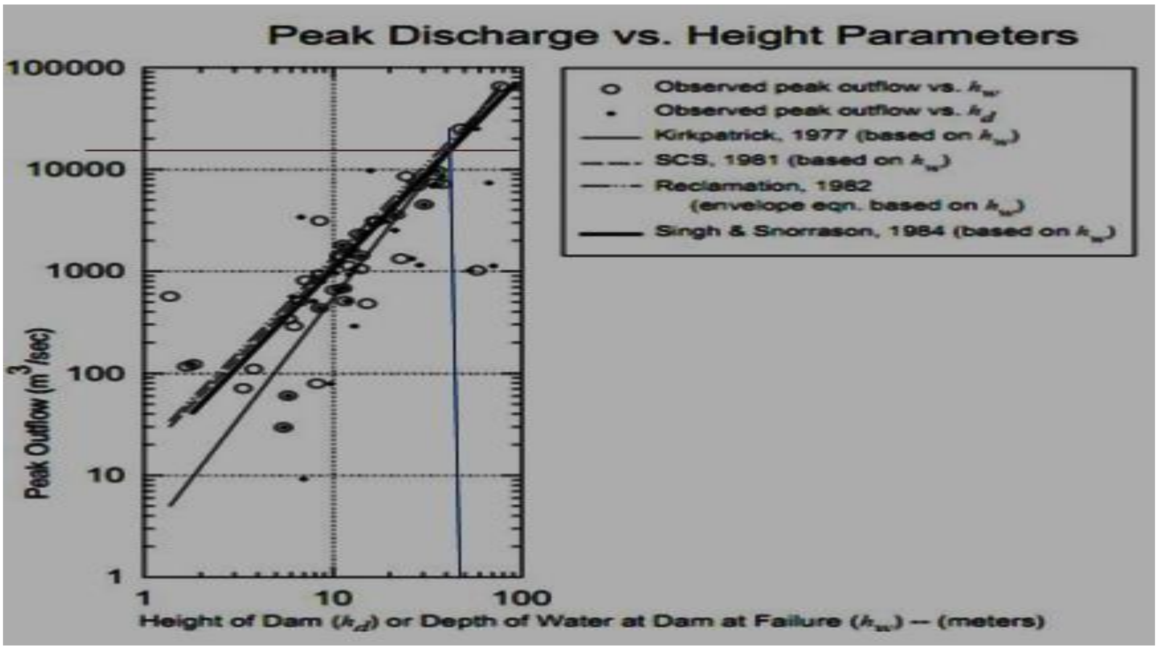


Figure Comparisons of Case Study Data and Proposed Relations for Peak Outflow (Wahl, 1998).

4.7 Sensitivity Analysis of Breach out Flow

The dam breach out flow analysis was done by varying breach parameter, i.e. time to failure, breach width and breach slope. In this study HEC-RAS have been used to perform the sensitivity analysis at the dam site.

In this sensitivity analysis, three different time of failure,breach width and breach slope were used for HEC-RAS dam-break computer runs and resulted in three dam breaching outflow discharge. Based on results, the peak discharge decreased when time of failure increased, this is logical because it is well known that discharge is inversely proportional to time. And the maximum discharge increased as the side slope of breach increased at the dam site.in addition to this sensitivity of breach out flow were tested for breach width and the result showed that as breach width increases the peach discharge also increases.

The increments of percent change in peak out flow of changing breach width were higher compared to the corresponding percent changes in time of failure and breach side slope.

Based the analysis done for time of failure, A 15 % increment (1 Hrs to 1.15Hrs) resulted in 2 %decrease in peak discharge at the dam site.Whereas, a15% reduction of time (1 Hrs to 0.625Hr) resulted in 28 % increase in peak discharge at the same site. The analysis for side slope also was done, and a15 % increment (1 Hrs to 1.15 Hrs) resulted in 17 % decrease in peak discharge at the dam site. Whereas,an15% reduction (1 Hrs to 0.75Hr) resulted in 8 % increase in peak discharge at the same site. Finally dam breach out flow sensitivity was done for breach width and the following results werefound. A 15 % increment (171 Hrs to 196 Hrs) resulted in 34% increase in peak discharge at the dam site. Whereas,an15% reduction (171 Hrs to 145Hr) resulted in 33.6 % increase in peak discharge.

4.8 Flood Inundation Mapping

The downstream area of the dam would be affected as a result of Dabus dam failure. The flood inundation map for the entire study area for Dabus dam failure is presented for overtopping type failure and for piping type failure. Flood inundation area in both failure modes is slightly different when treated with different breach parameter estimation methods. The general flood modes in low lying areas have inundated with different depth and velocity magnitudes.

Comparison of the results tabulated in the above section reveals breach model results ofxu&zhangandMacDonald et al method has resulted in the highest discharge values in over topping and piping modes of failure respectively.

Selection of appropriate and reasonable method for emergency action plan preparation depends on the decision of the EAP preparing body but the results have a direct influence on the economy of flood hazard minimization plan.

Important areas of flood depth and velocity for case of over topping failure mode are shown in the figure below for Froehlich (2008) method. The flood Hazard map according to FEMA (2014) classification is also prepared for overtopping failure mode as shown in Figure 4-12.

Flood hazard map

After running the model, the in-built RAS Mapper tool in HEC-RAS can be used for flood mapping, and other computations related with the flood such as flood velocity map and depth. Alternatively, analysis results can be exported from HEC-RAS in GIS format to ArcGIS for post processing. When exporting, any output interval can be selected; usually the maximum water surface would be of interest for flood inundation mapping. The profiles can be imported to ArcGIS platform for post-processing and with HEC-Geo-RAS post-processing tools, inundation map for the water surface profile of interest can then be plotted. Based on these maps, hazard classification is made in order to implement effective flood forecasting and warning system.

Different studies and guidelines have different flood hazard classification limits and justifications. The FEMA (2014) guidelines used for this study as shown in the figure below

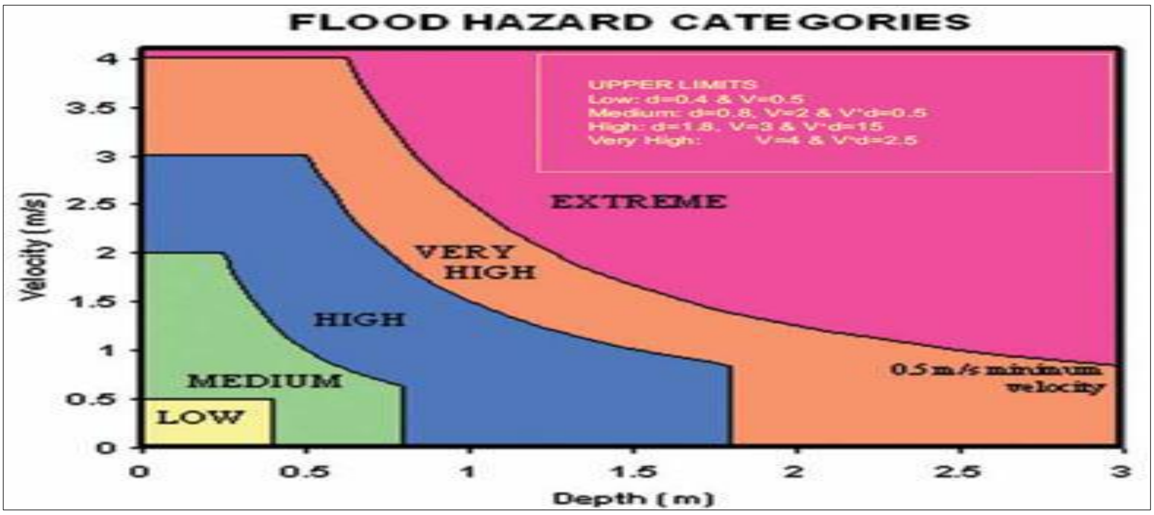


Figure 4-11: Flood hazard categories (source FEMA document)

Representing the information in the figure above in a simplified table, the FEMA document states:

"To produce a flood severity grid that exactly matches the categorization shown in the figure above, additional rules would need to be applied when calculating the depth * velocity product, to take into account the depth and velocity upper limits of each category. Additionally, the flood severity thresholds are different depending on whether they are being considered related to the impact on humans, vehicles, or buildings. As a simplified approach, the following depth * velocity categories can be applied when symbolizing the results of the dataset. However, other categorizations of this data may be used where desired."

Flood Severity Category	Depth * Velocity Range (m2/sec)
Low hazard	< 0.2
Medium hazard	0.2 – 0.5
High hazard	0.5 – 1.5

Very High hazard	1.5 – 2.5
Extreme hazard	> 2.5

Table 4-12: Simplified Flood Depth and Velocity Severity Grid Symbolization Categories (Source FEMA (2014

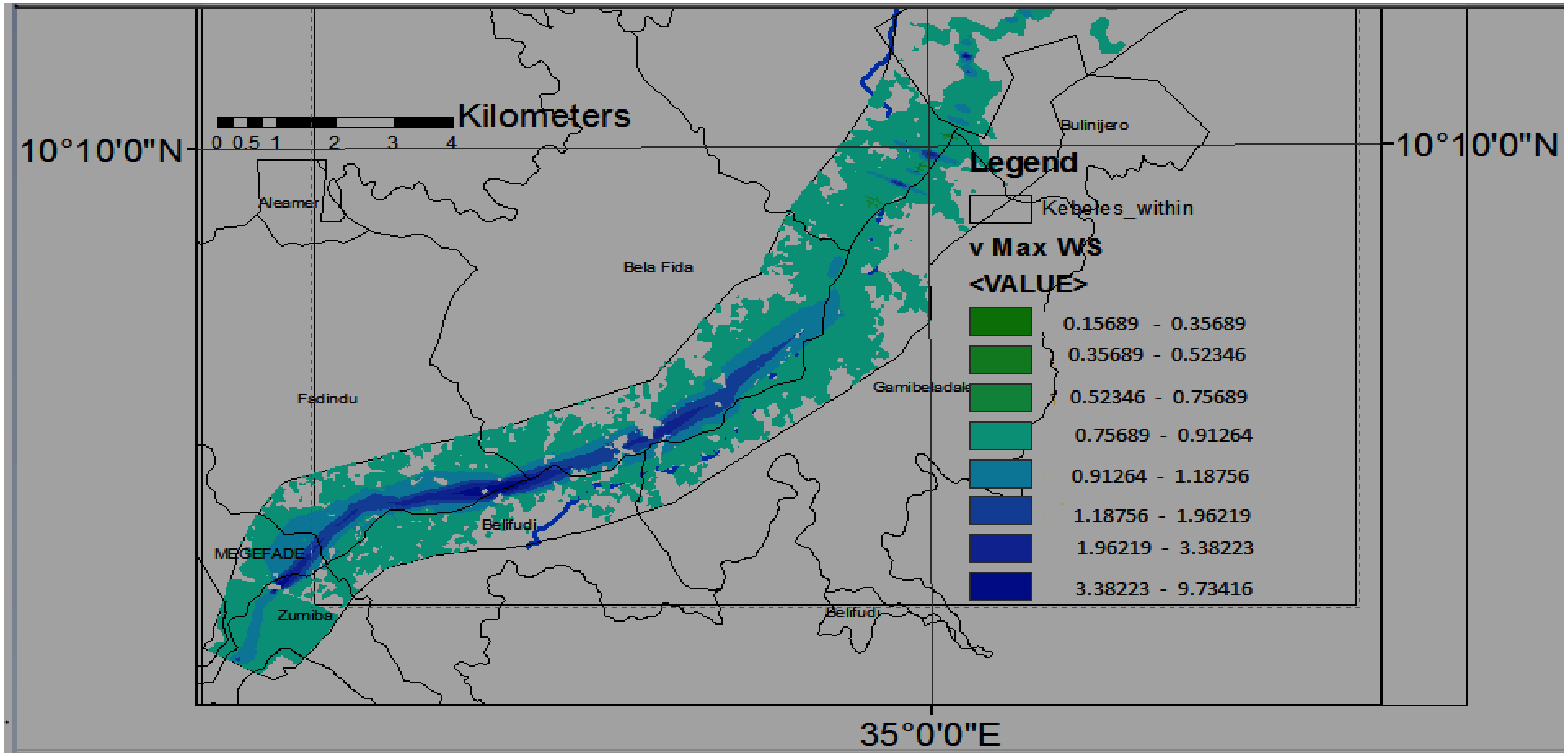


Figure 4-12: over topping, breach by Froehlich (2008) velocity map

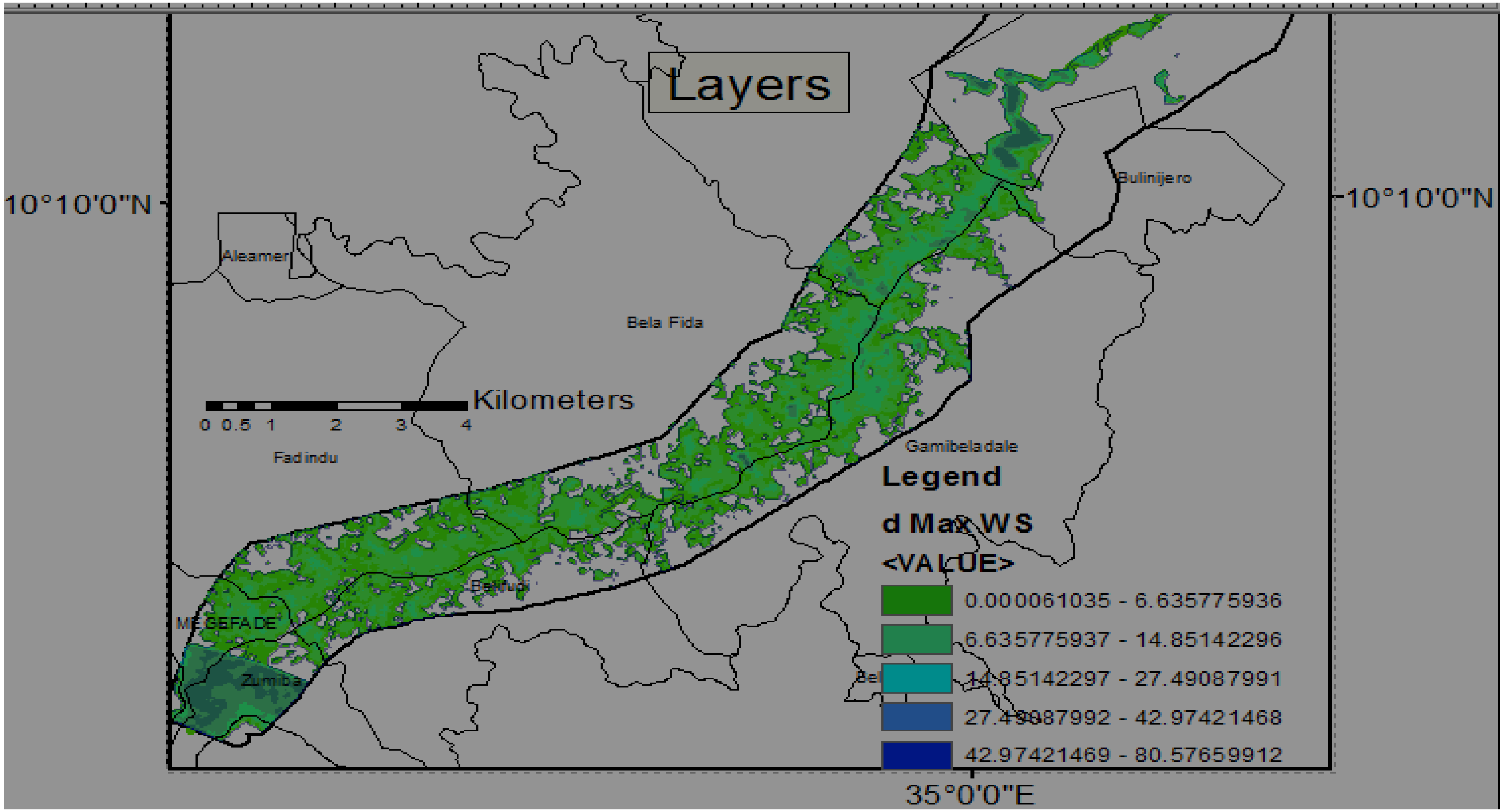


Figure 4-13: over topping, breach by Froehlich (2008) depth map

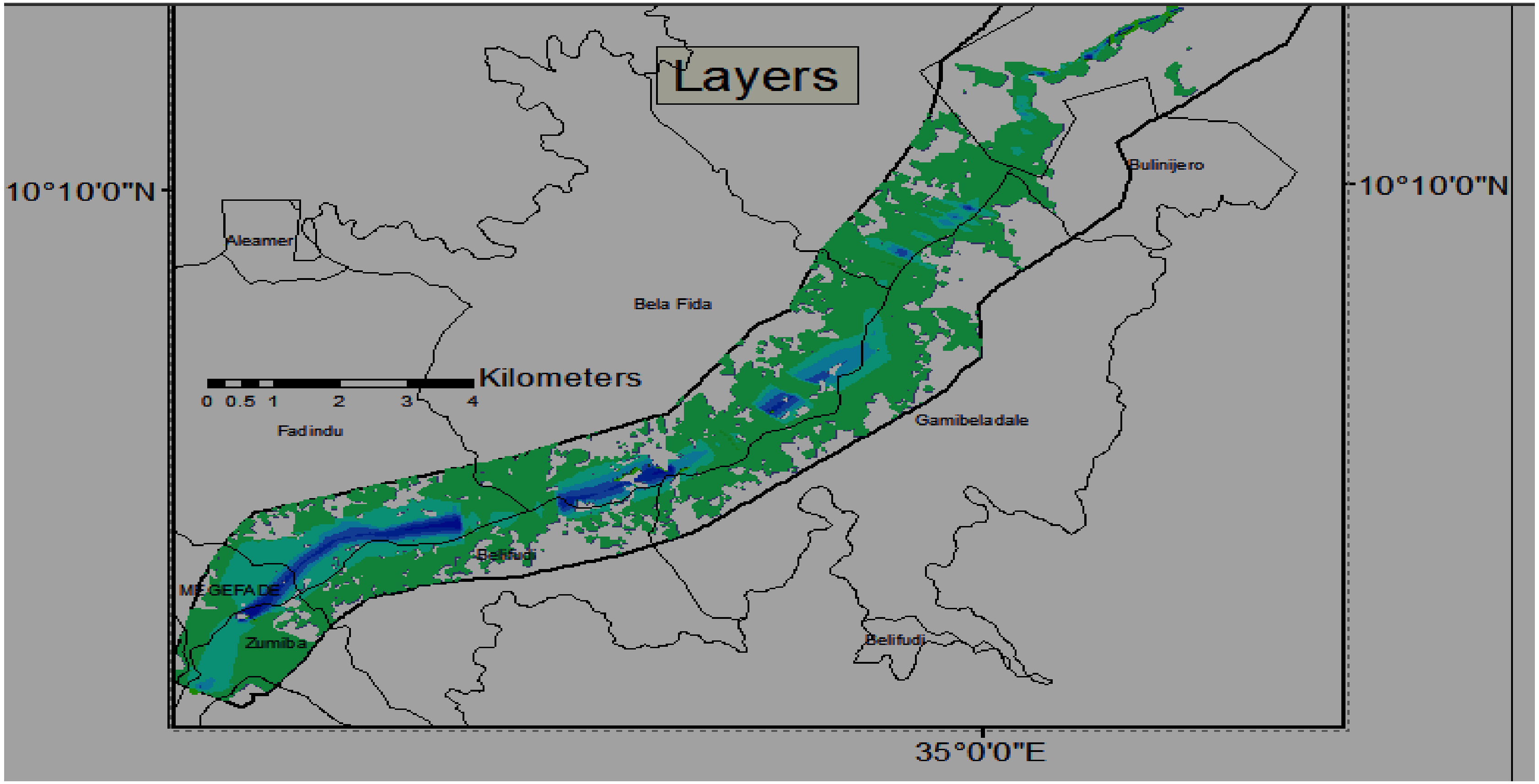


Figure 4-14: Over topping, breach by Froehlich (2008) hazard map

4.9 Discussion

The breach parameters estimated using four most currently used regression equations, i.e. MacDonald et al, Froehlich (2008) ,xu&zhang and VonThun and Gillette are different for the two mode of failure discussed in this paper.

In the case of hydrologic (overtopping) mode of failure, the MacDonald et al equation was over-predicting the breach bottom width,whileVon thun&Gillete equation under estimate it. The result from the equationof Froehlich-2008 and xu&zhangwas between the two results and can be approximated to each other. 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 2.86 hours, 2.54 hour, 0.91 hours and 5.41hours respectively.

When these four results are compared, the result of MacDonald et al breach width is larger and the breach development time is between the other results. Both breach bottom width and breach development time for Von thun&Gilletemethod 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.

Dabus dam is high dam and the volume of the water in the reservoir is about 121.48Mm3 at NWL. 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. In addition to this Dabus 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 have the trends.

While considering result of model simulation in the case of hydrologic or overtopping mode of failure using Probable Maximum Flood event, the maximum breach outflows and time to peak were computed for all cases. Using MacDonald et al breach parameter as model input in HEC-RAS , the maximum breach outflow and time to peak which occurred at the dam were 9702 m^3/s and 2.86 hours respectively. From Von Thun and Gillette also14220m^3/s peak outflows and 0.91 hours' time to peak were calculated at the dam location. Again from Froehlich (2008) andxu&zhangthe maximum breach outflow which occurred at the dam were 8838.36 m^3/s and 1316.61m^3/s respectively and the time to peak were 2.54 and 5.41 hours respectively.

For fair weather or sunny day (Piping) mode of failure at normal pool level, the simulation result of maximum outflows for the four methods at dam i.e. MacDonald et al, Froehlich (2008), xu&zhang and VonThun and Gillette was12891.53m3/s 12616.04m3/s, 12514.57m3/s and 12717.02 m3/srespectively.And the time to peakfor all simulation was 2.86, 2.54, 0.91 and 5.23hours respectively.

The simulated results for overtopping case for MacDonald et al showed a maximum flow velocity of 2.41m/s with a discharge of 9702m^3/s occurred at the dam. For Froehlich (2008) the maximum flow velocity of 2.1m/s with a discharge of 8838.36m^3/s occurred also at the dam. Finally, for xu&zhang and VonThun and Gillette the maximum flow velocity of 4.69m/s and 1.9m/s with a discharge of 1316.63m^3/s and 14220m^3/s occurred at the dam.

Considering downstream of the dam at a distance of 10km from the dam, the peak discharge, flow velocity and flow depth for MacDonald et al in over topping case was 9248m3/s,1.61m/s and 15m respectively.at the same distance from the dam the peak discharge, flow velocity and flow depth for Froehlich (2008) was 8783.31m3/s, 4.1m/s and 15m respectively.

Further analyzing downstream of the dam to the distance of 35km from the dam, the peak discharge, flow velocity and flow depth xu&zhangin piping case was 249.75m3/s,1.53m/s and 3m respectively.at the same distance these values for VonThun and Gillette method in the piping case was 249.75m3/s,2.68m/s and 2.5m respectively.

The results of the flood inundation mapping from dabus Dam breach analysis indicates that much of the severe flood inundation occurs before 40 km downstream of the dam near zumbia, belifud, gemibeledale,fadindu, belafida and egefade village kebeles and large Irrigation field is found in the left and right side of the river, and after 40km downstream the inundation area is limited to the river banks as Shawn in the figure in annexes.

5. Conclusion and Recommendation

5.1 Conclusion

Through this thesis work, eight computer runs were made to see the hazardness of breach when treated with different methods and different scenarios using the Hydrologic Engineering Center River Analysis System (HEC-RAS v5.0.1) Software developed by the US Army Corps of Engineers (USACE, 2016).The methods implied basically are for overtopping and piping modes of failure. Four of the runs are for overtopping and the rest four are for piping mode of failure.

The methods use statistics of historic dam failures and generate results by approximating the dam to similar other failed dams based on given data. These methods proposed by Frohelich 2008, Mac Donald et al, Von thun and Gillette and Xu and Zhangand were selected to determine breach parameters to compare and examine associated results.

The breach bottom width, the peak discharge, time of failure in hr and side slope were selected as major breach parameters and comparison, results and conclusion is pointed as follows.

A change in dam breach characteristics parameters resulted in significant changes in peak flows at the dam site. The rates of changes in peak flows were not uniform in the specified sensitivity analyses. This observation indicated that the peak flows were sensitive to changes in time to dam failure, side slope of breach and breach width at various degrees.

- The dam is safe from breaching out due to overtopping failure mode during 10,000yrs return period flood, but it not safe when checked by PMF, since it over tops the dam.
- While running the model with four different methods (, Mac Donald et al, Xu and Zhang ,Von thun and Gillette and Frohelich 2008) it

resulted in peak out flow values of 9702 m^3/s, 1316.61 m^3/s,14220m^3/s and 8838.36 m^3/s during overtopping and 12891.53m^3/s, 12616.04 m^3/s, 12514.57 m^3/s and 12717.02 m^3/s during piping respectively.

- In the overtopping case the peak flow result which is equal to 14220m^3/s has obtained by xu&zhang breach estimation method and peak flow obtained by Froehlich (2008) breach estimation method was equal to 8838.36 m^3/s.
- In the piping case MacDonald et al method has resulted in a peak flow of 12891.53 $m^3/s$and Mac Von thun&Gilletehas resulted in a minimum flow of 12514.57m^3/s .
- Side slopes are found to be in a range of 0.5H: V to 1.40H: V in overtopping case and 0.5H: V to 1H: V in piping case for both sides, all trapezoidal.
- Time of failure in hours is found to be in a range of 1.15hrs to 3.65hrs in overtopping case and 1.15hrs to 3.58hrs in piping case.
- All cases of inundation were mapped and in all cases similar villages (zumbia, belifud, gemibeledale,fadindu, belafidaandegefade) were inundated and irrigation is on the right and left site of the Dabus River was also inundated.
- The results of the flood inundation mapping from Dabus Dam breach analysis indicates that much of the severe flood inundation occurs before 40 km downstream of the dam near at the left and right of the river where large Irrigation field and settlement occurs.
- A 15 % reduction in time of failure resulted in 28 % increase in peak discharge at the dam site. But, a15 % increase in time of failure resulted in 2 % increase in peak discharge at the same site. This implied that when the dam collapses rapidly, the peak discharge became high at the dam site. This concluded that the maximum

discharge at the dam site was slightly sensitive to change in time of failure.

- A 15% decrease in side slope resulted in 8% reduction in maximum discharge at the dam site. Whereas, a 15% increase in side slope value produced a 17% increase in maximum discharge at the dam site. This concluded that maximum discharge at the dam site was slightly sensitive to change in side slope.
- A 15% decrease in breach width resulted in 33.6% reduction in maximum discharge at the dam site. Whereas, a 15% increase in breach width value produced a 34% increase in maximum discharge at the dam site. This concluded that maximum discharge at the dam site was highly sensitive to change in breach width.

5.2 Recommendation

After analyzing the results obtained in this research, the following recommendations are suggested.

- While doing this thesis there is unavailability of actual survey data to collect cross-section measurement, to fill this data gap cross-section geometry were generated from low resolution DEM (30mx30m details) by using HEC-Geo-RAS software. Therefore This research can be refined further by collecting actual surveying data on the ground or using high resolution DEM data to get more accurate results.
- Based on results of this study, a research focusing on flood damage analysis for the command area of irrigation and resettlement villages can be done.
- The area below the dam is relatively flat and people living on this area, especially near the river banks, shall be resettled at significantly far place.
- Emergency Action Plans (EPA) should be set in order to evacuate people potentially at risk in case the dam fails.
- Finally, even if this dam was designed to pass the inflow design flood safely, which is 10,000 return period flood, the dam will overtop in the case of PMF. 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 and 10,000yr return period Inflow Hydrograph of Dabus River

Time	**PMF**	**0.75*PMF**	**10,000Return period**
1	449.55	337.1625	166.5
2	449.01	336.7575	166.3
3	448.47	336.3525	166.1
4	447.93	335.9475	165.9
5	447.39	335.5425	165.7
6	446.85	335.1375	165.5
7	446.31	334.7325	165.3
8	445.77	334.3275	165.1
9	445.23	333.9225	164.9
10	444.69	333.5175	164.7
11	443.61	332.7075	164.3
12	442.8	332.1	164
13	477.36	358.02	168.9
14	516.78	387.585	176.1
15	598.32	448.74	190.5
16	690.93	518.1975	209
17	802.71	602.0325	232.4
18	925.83	694.3725	259.3
19	1108.35	831.2625	292.2
20	1508.76	1131.57	334.5
21	2324.7	1743.525	414.5
22	3036.15	2277.1125	586.4

23	3292.65	2469.4875	780.1
24	3527.01	2645.2575	888.7
25	3841.83	2881.3725	963.8
26	4254.93	3191.1975	1,046.60
27	4665.6	3499.2	1,157.10
28	4972.32	3729.24	1,280.80
29	5238.54	3928.905	1,396.60
30	5467.5	4100.625	1,488.30
31	5695.11	4271.3325	1,570.30
32	5869.8	4402.35	1,640.30
33	6060.42	4545.315	1,704.70
34	6265.35	4699.0125	1,764.80
35	6526.98	4895.235	1,825.00
36	6886.89	5165.1675	1,887.20
37	7493.58	5620.185	1,957.30
38	8462.34	6346.755	2,053.00
39	9843.66	7382.745	2,198.50
40	11492.28	8619.21	2,400.80
41	12739.14	9554.355	2,646.70
42	13196.25	9897.1875	2,934.60
43	13154.67	9866.0025	3,280.40
44	12924.36	9693.27	3,621.60
45	12594.96	9446.22	3,828.90
46	12238.56	9178.92	3,873.50
47	11882.43	8911.8225	3,832.10
48	11510.64	8632.98	3,762.90
49	11158.29	8368.7175	3,685.50
50	10787.85	8090.8875	3,598.50
51	10399.86	7799.895	3,501.80

52	9997.29	7497.9675	3,396.50
53	9597.96	7198.47	3,286.40
54	9202.14	6901.605	3,173.00
55	8825.49	6619.1175	3,059.60
56	8467.74	6350.805	2,947.30
57	8148.33	6111.2475	2,836.10
58	7850.79	5888.0925	2,726.60
59	7565.13	5673.8475	2,619.60
60	7287.57	5465.6775	2,518.30
61	7013.79	5260.3425	2,426.50
62	6747.57	5060.6775	2,335.70
63	6489.99	4867.4925	2,246.60
64	6235.65	4676.7375	2,158.80
65	5986.98	4490.235	2,072.60
66	5753.97	4315.4775	1,988.60
67	5544.45	4158.3375	1,907.40
3780	5357.34	4018.005	1,829.90
69	5185.35	3889.0125	1,755.90
70	5029.56	3772.17	1,685.70
71	4893.48	3670.11	1,621.40
72	4771.98	3578.985	1,564.40
73	4658.58	3493.935	1,512.20
74	4553.28	3414.96	1,463.90
75	4454.46	3340.845	1,419.40
76	4358.88	3269.16	1,377.30
77	4267.35	3200.5125	1,337.60
78	4181.49	3136.1175	1,301.00
79	4097.79	3073.3425	1,266.80
80	4018.14	3013.605	1,234.90

81	3946.05	2959.5375	1,205.60
82	3878.28	2908.71	1,178.50
83	3817.26	2862.945	1,152.80
84	3773.25	2829.9375	1,128.00
85	3735.18	2801.385	1,103.90
86	3698.19	2773.6425	1,081.10
87	3662.01	2746.5075	1,064.00
88	3626.37	2719.7775	1,050.40
89	3591.27	2693.4525	1,037.60
90	3556.98	2667.735	1,025.10
91	3522.96	2642.22	1,012.70
92	3489.48	2617.11	1,000.60
93	3456.81	2592.6075	988.7
94	3424.68	2568.51	977
95	3393.09	2544.8175	965.4
96	3362.31	2521.7325	954.1
97	3331.8	2498.85	942.9
98	3302.37	2476.7775	931.9
99	3273.21	2454.9075	921.1
100	3244.59	2433.4425	910.5
101	3216.51	2412.3825	900.1
102	3188.97	2391.7275	889.8
103	3161.97	2371.4775	879.8
104	3135.51	2351.6325	869.9
105	3109.32	2331.99	860.2
106	3083.67	2312.7525	850.7
107	3058.29	2293.7175	841.3
108	3033.45	2275.0875	832.1
109	3009.15	2256.8625	823

110	2985.12	2238.84	814
111	2961.36	2221.02	805.2
112	2938.14	2203.605	796.5
113	2915.19	2186.3925	788
114	2892.51	2169.3825	779.6
115	2870.1	2152.575	771.2
116	2848.23	2136.1725	763
117	2826.36	2119.77	754.9
118	2804.76	2103.57	746.9
119	2783.16	2087.37	739
120	2761.83	2071.3725	731.3
121	2740.77	2055.5775	723.6
122	2719.98	2039.985	715.8
123	2699.46	2024.595	708.04
124	2679.21	2009.4075	700.28
125	2659.23	1994.4225	692.52
126	2639.79	1979.8425	684.76
127	2620.35	1965.2625	677
128	2601.18	1950.885	669.24
129	2582.28	1936.71	661.48
130	2563.65	1922.7375	653.72
131	2545.29	1908.9675	645.96
132	2527.2	1895.4	638.2
133	2509.11	1881.8325	630.44
134	2491.29	1868.4675	622.68
135	2474.01	1855.5075	614.92
136	2456.73	1842.5475	607.16
137	2439.45	1829.5875	599.4
138	2422.71	1817.0325	591.64

139	2405.97	1804.4775	583.88
140	2389.23	1791.9225	576.12
141	2372.76	1779.57	568.36
142	2356.56	1767.42	560.6
143	2340.63	1755.4725	552.84
144	2324.7	1743.525	545.08
145	2308.77	1731.5775	537.32
146	2293.11	1719.8325	529.56
147	2277.45	1708.0875	521.8
148	2262.06	1696.545	514.04
149	2246.67	1685.0025	506.28
150	2231.55	1673.6625	498.52
151	2216.7	1662.525	490.76
152	2201.85	1651.3875	483
153	2187	1640.25	475.24
154	2172.69	1629.5175	467.48
155	2158.38	1618.785	459.72
156	2144.07	1608.0525	451.96
157	2129.76	1597.32	444.2
158	2115.99	1586.9925	436.44
159	2102.22	1576.665	428.68
160	2088.45	1566.3375	420.92
161	2074.95	1556.2125	413.16
162	2061.72	1546.29	405.4
163	2048.49	1536.3675	397.64
164	2035.53	1526.6475	389.88
165	2022.57	1516.9275	382.12
166	2009.61	1507.2075	374.36
167	1996.92	1497.69	366.6

168	1984.5	1488.375	358.84
169	1972.08	1479.06	351.08
170	1959.66	1469.745	343.32
171	1947.51	1460.6325	335.56
172	1935.63	1451.7225	327.8
173	1923.48	1442.61	320.04
174	1911.87	1433.9025	312.28
175	1899.99	1424.9925	304.52
176	1888.38	1416.285	296.76
177	1877.04	1407.78	289
178	1865.7	1399.275	281.24
179	1854.36	1390.77	273.48
180	1843.29	1382.4675	265.72
181	1832.22	1374.165	257.96
182	1821.15	1365.8625	250.2
183	1810.35	1357.7625	242.44
184	1799.55	1349.6625	234.68
185	1789.02	1341.765	226.92
186	1778.49	1333.8675	219.16
187	1768.23	1326.1725	211.4
188	1757.97	1318.4775	203.64
189	1747.71	1310.7825	195.88
190	1737.72	1303.29	188.12
191	1727.73	1295.7975	180.36
192	1718.01	1288.5075	172.6
193	1708.29	1281.2175	164.84
194	1698.84	1274.13	157.08
195	1689.39	1267.0425	149.32
196	1679.94	1259.955	141.56

197	1670.76	1253.07	133.8
198	1661.58	1246.185	126.04
199	1652.4	1239.3	118.28
200	1643.49	1232.6175	110.52
201	1634.58	1225.935	102.76
202	1625.67	1219.2525	102.76
203	1617.03	1212.7725	102.76
204	1608.39	1206.2925	102.76
205	1599.75	1199.8125	102.76
206	1591.38	1193.535	102.76
207	1583.01	1187.2575	102.76
208	1574.64	1180.98	102.76
209	1566.27	1174.7025	102.76
210	1558.17	1168.6275	102.76
211	1550.07	1162.5525	102.76
212	1542.24	1156.68	102.76
213	1534.14	1150.605	102.76
214	1526.31	1144.7325	102.76
215	1518.48	1138.86	102.76
216	1510.92	1133.19	102.76
217	1503.36	1127.52	102.76

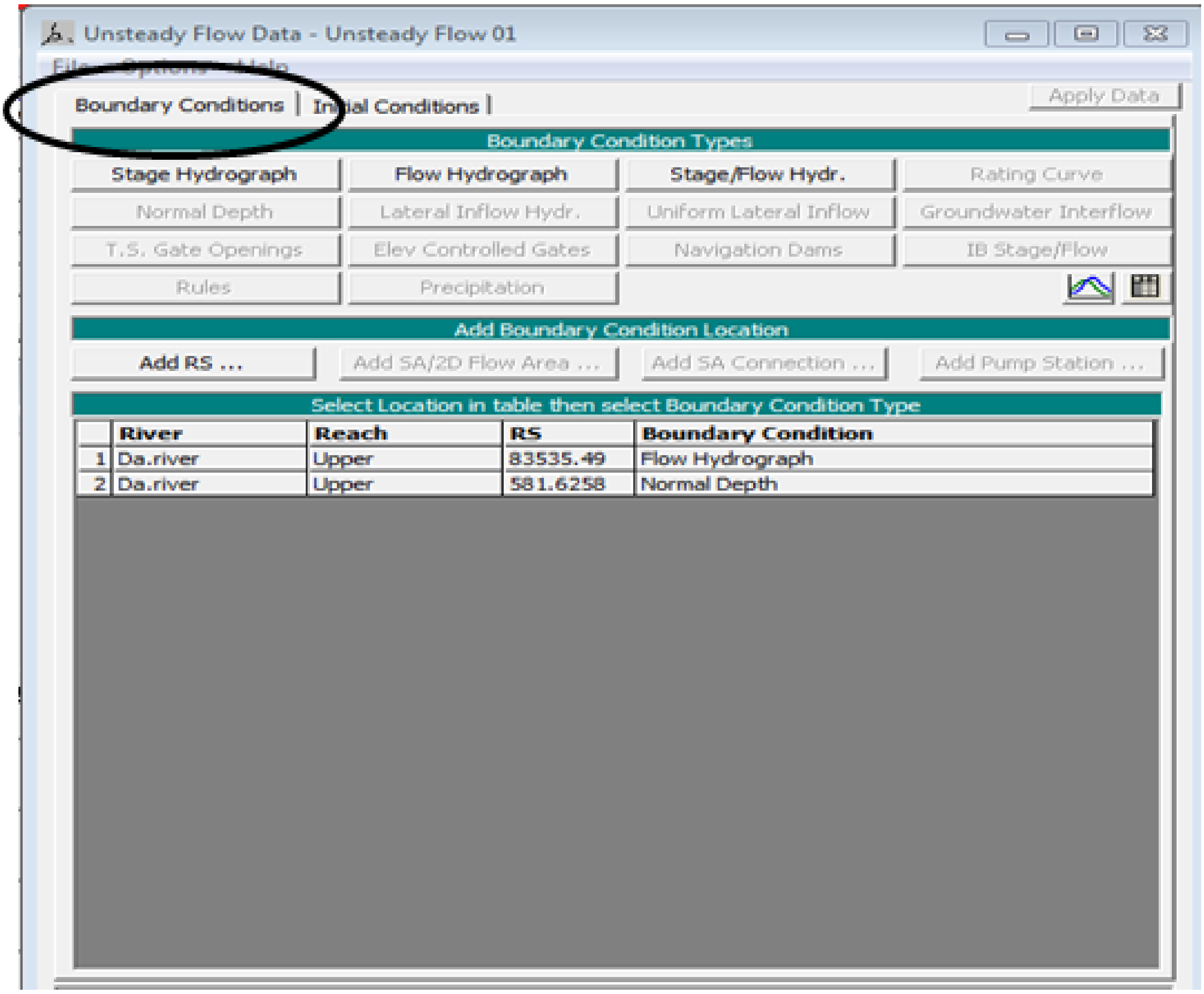

Figure 1: Unsteady Flow Boundary Condition

Appendix B: Output Results of HEC-RAS Model

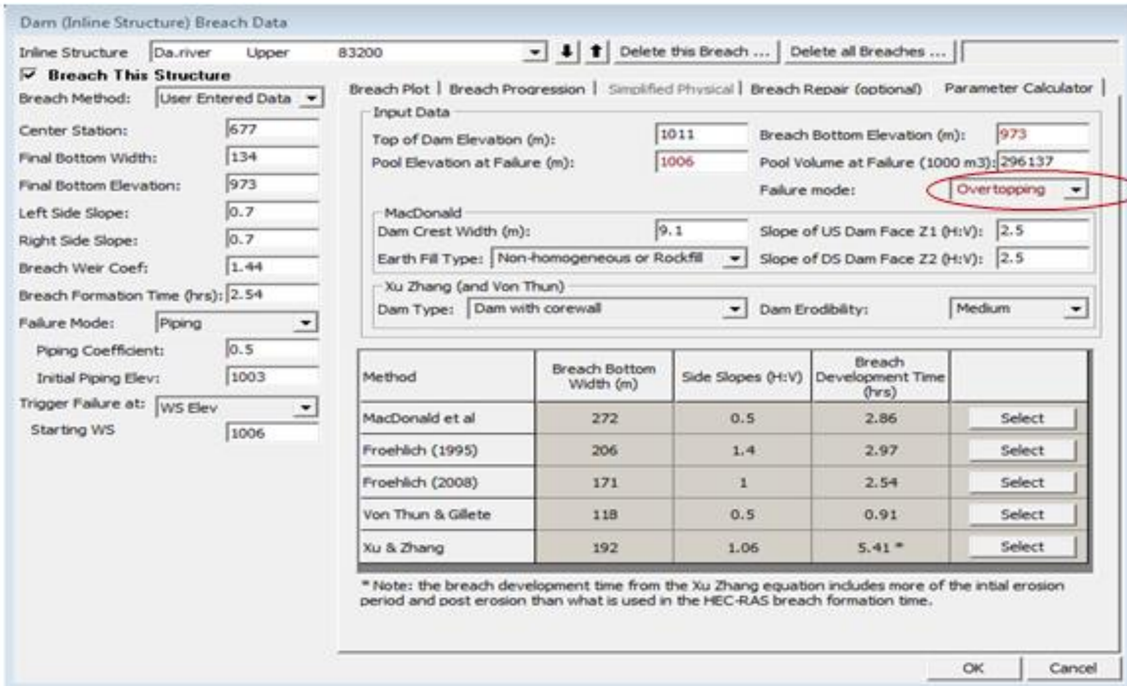

Figure 2: Breach parameter output for over topping case

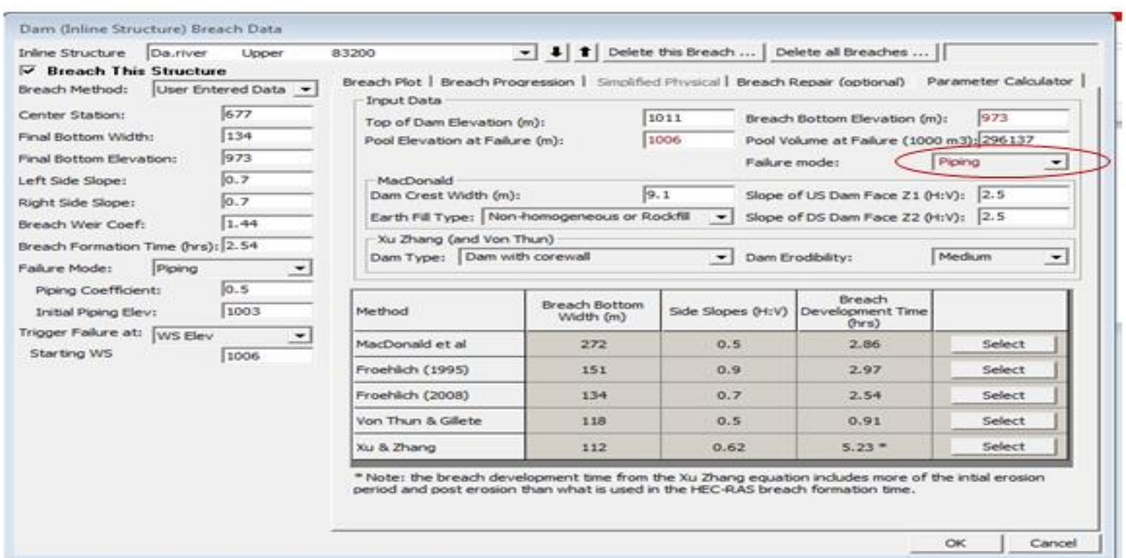

Figure 3: Breach parameter output for piping case

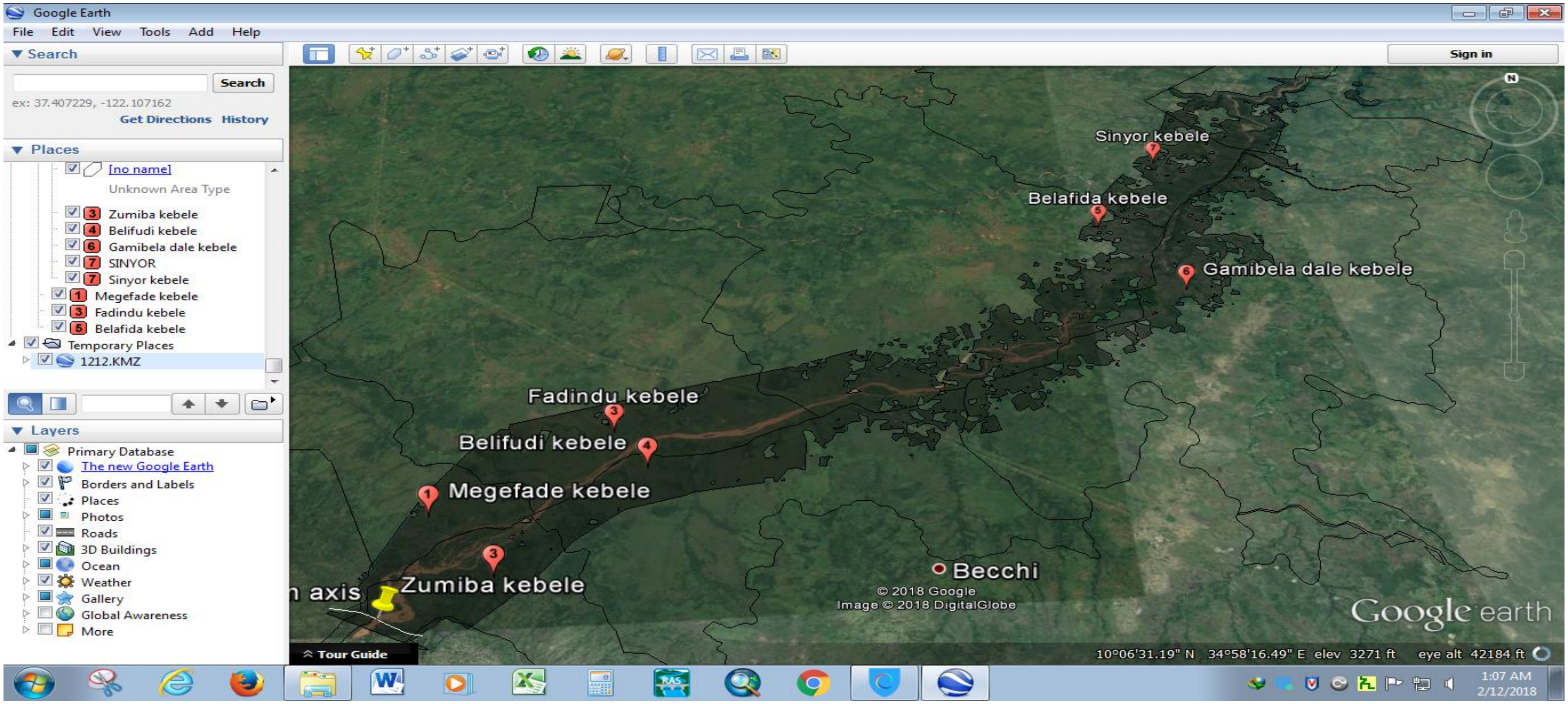

Figure 4: Inundation area map of over topping failure by Macdonald et al method

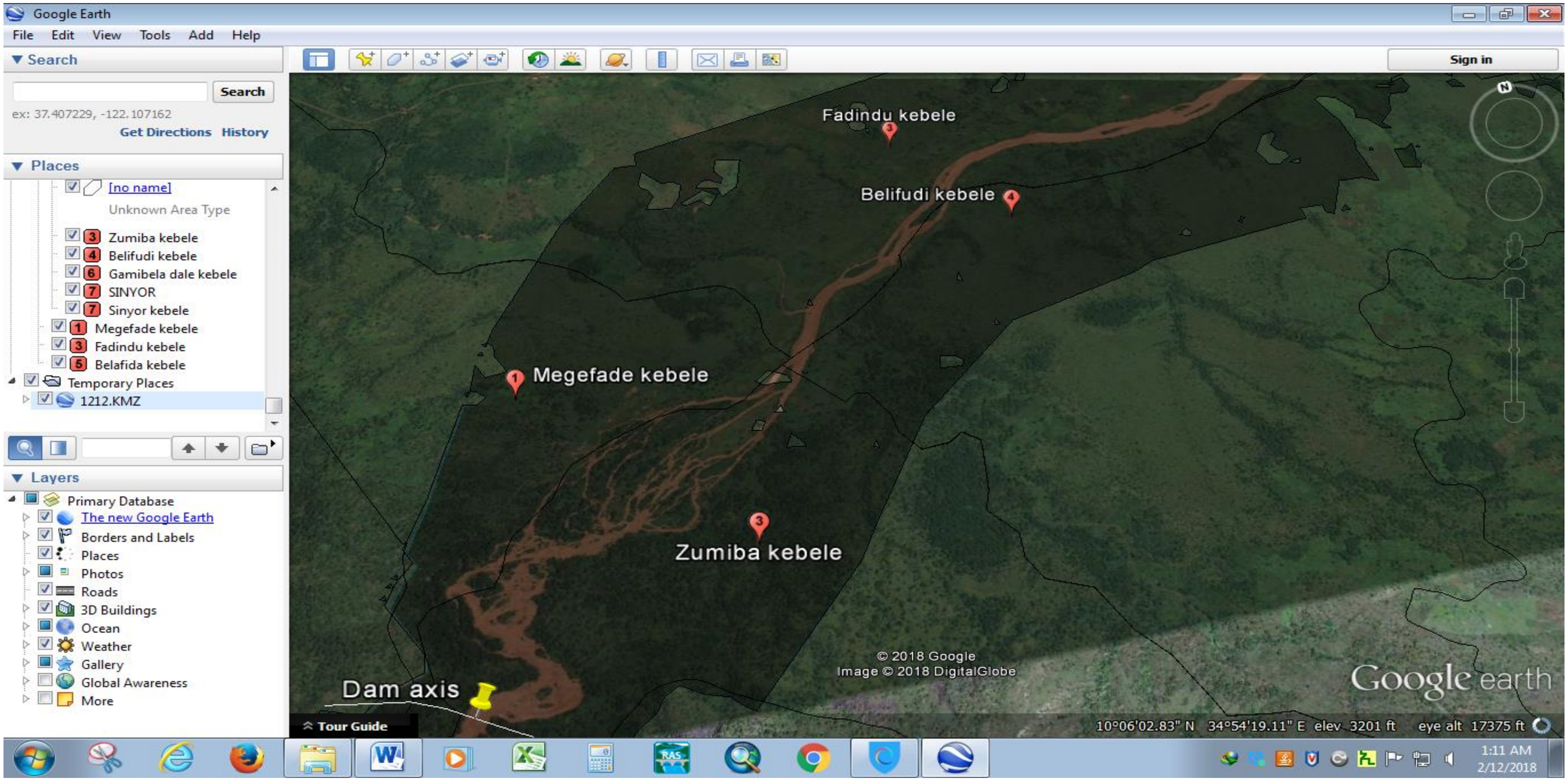

Figure 5: Inundation area map of over topping failure by Macdonald et al method

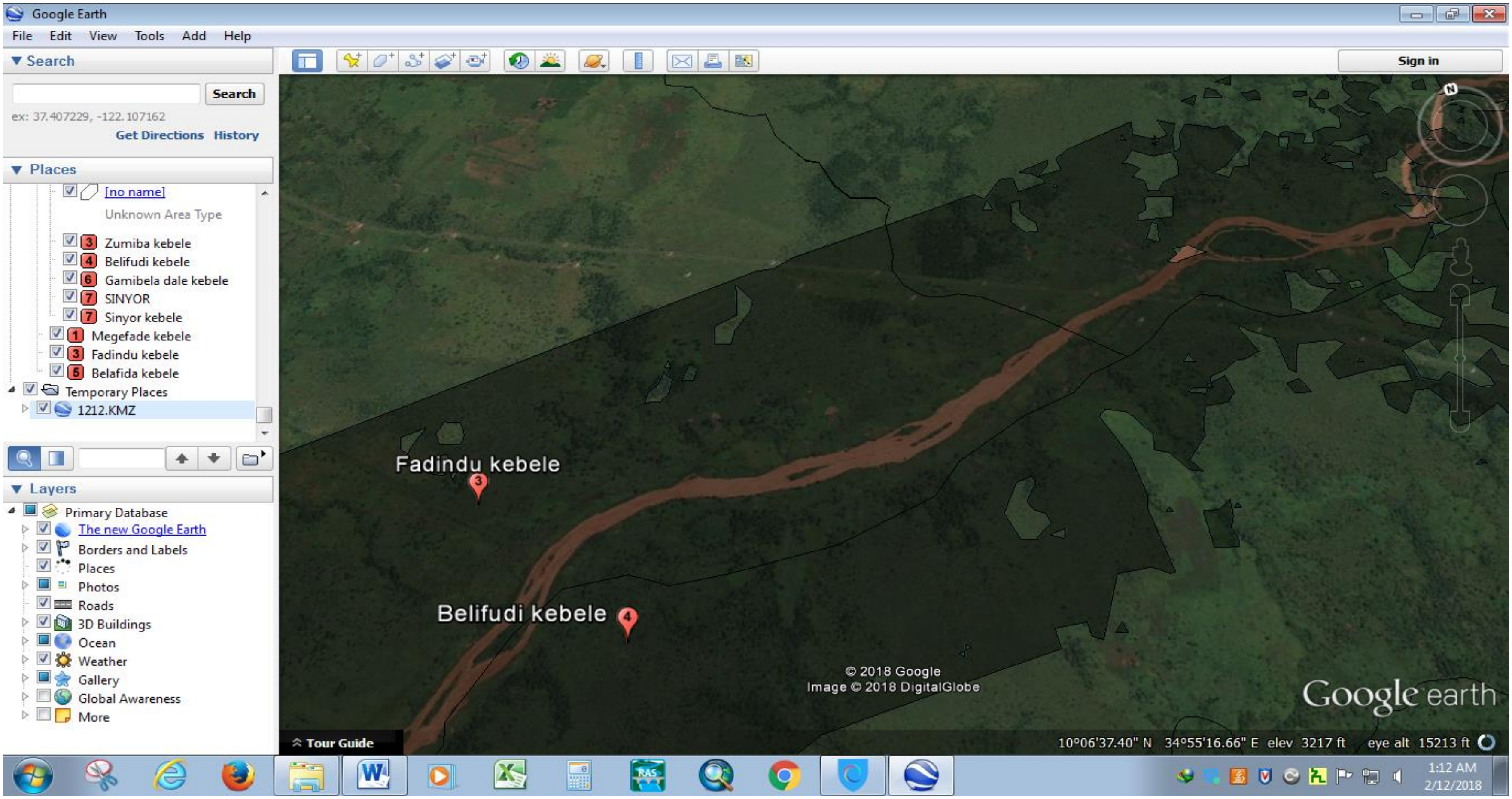

Figure 6: Inundation area map of over topping failure by Macdonald et al method

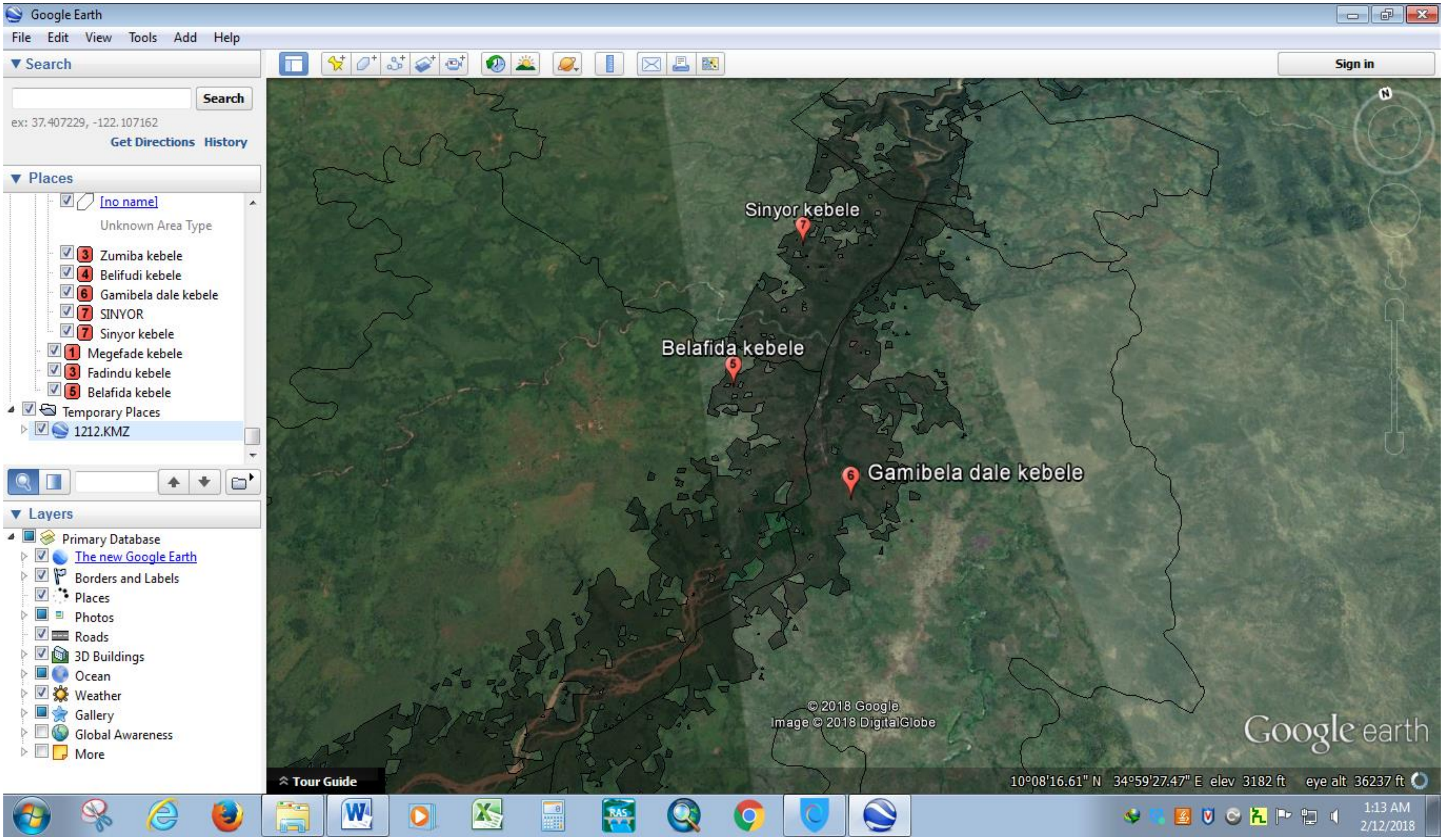

Figure 7: Inundation area map of over topping failure by Macdonald et al method