

# ADDIS ABABA UNIVERSITY INSTITUTE OF TECHNOLOGY



**Addis Ababa University**  
**School of Graduate Studies**  
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**“Dam breach modeling and inundation mapping,  
a case study on Gidabo 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.

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## ABSTRACT

Preparing dam safety plans and hazard management strategies are unquestionably vital, since lots of human lives have been lost and tremendous amount of economic crisis have been recorded from dam failure events throughout the world in history. 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.

This thesis addresses pre-event analysis of a dam breach scenario which is currently under construction in the Sidama zone of SNNPRS, the Gidabo dam. The paper focuses on overtopping and piping as the possible reasons of breach and identifies the inundation properties for a bunch of breach parameters suggested by five different scholars. This has resulted in ten computer runs where halves are for piping and the rest halves for overtopping.

The objective is to arrive at possible dam breach parameters when calculated by different methods and to route the resulting flood through this calculated dam openings and to the downstream extents. This routed flow has resulted in almost same inundation extents by all of the methods. In addition the paper compares the breach results when analyzed by physical based model “breach model” and non-physical based models.

HEC-RAS Ver 5.0 is used to model the dam failure. HEC-GEORAS Ver 10.0 is used with ARC-Map 10.0 to extract basic geometric data for use in HEC-RAS. A DEM is used as a terrain map to be used in ARC-Map.

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.

The four non physically based empirical methods have resulted in peak flow values within a range of 24966.21 m<sup>3</sup>/s and 16039.97 m<sup>3</sup>/s during overtopping and 17348.94 m<sup>3</sup>/s and 9843.82 m<sup>3</sup>/s during piping. This results can be said more close when compared with the physically based model results.

In conclusion, In any dam breach study, it would be good to check the breach by different scenarios, to see the range of hazardness when treated with different methods and to point out the optimal catastrophe so that emergency action planning preparation will be economical and safe. The area below the dam is relatively flat and people living on this area, specially near the river banks, shall be resettled at significantly far place.

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

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**List of Abbreviations**

ADLI Agricultural development led industrialization

ASCE American Society of Civil Engineers

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

SDF Spillway 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 percipitaion

$Q_p$  Peak Discharge

RS River Station

TFH Time of failure

USCOLD United states Committee on large dams

USDOI United states department of the interior

U/s Upstream

USACE US Army Corps of Engineers

WS Water Surface

WSEL Water Surface Elevation

WWDSE Water Works Design and Supervision Enterprise

## 1. Introduction

### 1.1 Background

Dams are an important part of this nation's infrastructure, providing flood control, water supply, irrigation, hydropower, navigation, and recreation benefits. Despite their many beneficial uses and value, dams also present risks to property and life due to their potential to fail and cause catastrophic flooding. To mitigate these risks, dam owners and regulators carefully analyze and inspect dams to identify potential failure modes and protect against them. Since no program for preventing failure can ever be certain, and because the potential for loadings exceeding design limits can never be eliminated, another essential part of risk mitigation is simulating potential failures and planning for them.

Generally, causes of failure are overtopping due to inadequate spillway capacity, foundation defects, 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 (Stedinger et al. 1996).

The Federal Government of Ethiopia has drawn up development plans and programs intended to improve the standard of living of the country's population. The most prominent of these plans are the commitment of the Government to a policy of an agriculture development led industrialization (ADLI), especially laying emphasis on the improvement of small holder agriculture by improving farming technology and boosting productivity with the aim of ensuring food security. This policy also aims at the development of agro-industry feeding on farm produces by small-holder farmers as well as large scale farms.

Major cereal food production in the country is based on rain-fed agriculture by smallholder farmers. Irrigated agriculture in Ethiopia makes very little contribution in food production. This situation has exposed the rural population to repeated cycles of famine as a result of annual crop failures due to drought (MOWIE, 2014).

The government has therefore, committed itself to increasing the role of irrigated agriculture in the country mainly for the production of food crops.

In this endeavor, the Water Sector Development Program for the period 2002 and 2016, gives irrigation a prominent place by proposing about 274,000 hectares of land to be developed in

the 15-year period of the program under large, medium and small scale irrigation (MOWIE, 2001).

As part of this program, the Federal and Regional governments at this time are financing the implementation of various irrigation projects in the country and other projects are in various stages of study and design.

One of these projects is the Gidabo Irrigation Project, designed by Water Works Design and Supervision Enterprise in association with Consulting Engineering Services (India) Private Limited and financed by the Ministry of Water irrigation and energy. An earthen dam is being constructed across Gidabo River to store 62.3 Million cubic meter of water at its full supply level. This large stored volume of water makes Gidabo 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 Gidabo 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. Software will be utilized in the reservoir components of the study and in unsteady flow routing through the downstream reaches.

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 as it has been found to be effective when applied during a dam burst in the southeastern Brazilian state of Minas Gera where catastrophe killed at least 17, 45 missing, town leveled by flood on 5 Nov, 2015.

Currently the Gidabo dam is under construction and the fill is on progress. The intake and outlet structures were built as designed and now the outlet is found settled by 23cm on 30/12/2015 and by 31cm by 13/1/2016. Immediately after this observation all works were suspended when top of embankment fill was 1217.40m and a detailed study was initiated by

the client on how the outlet settled, how it should be treated and how the construction shall proceed.

Following this, the consultant, Water Works Design and Supervision Enterprise has started a work to carry out an investigation on the cause of the problem and provide appropriate solution accordingly. The consultant has done detailed investigation and has come up with a report on March 2016 detailing cause of problem and appropriate measures. According to the consultant, the settlement or deformation along the conduit is attributed to the following factors (Settlement report, WWDSE March 2016);

- i. The weak foundation and back filled material;
- ii. Some compressible organic material is left during excavation and remain there in the foundation; and
- iii. Dewatering process on the right side of the conduit after the whole conduit work is completed.

The consultant has pointed out the following risks associated with progressive settlement of the conduit foundation.

- Structural damage to the conduit (cracking): if there is excessive settlement particularly (differential settlement) the strain in the rigid concrete will result cracking of the conduit.
- Piping along the conduit-dam body (mainly the core) contact
- Miss match of the intake opening and the conduit and associated hydraulic problems: this is a plausible risk and it might require the reconstruction or modification of the part of the intake structure that is constructed.(Settlement report, WWDSE March 2016)

The embankment of high earth dam weight together with that of the reservoir water overlaying the upstream slope causes settlements in the foundation ranging from a few cm at sites with hard rock to many meters at dams underlain by thick deposits of compressible soils. Depending on the foundation material, nearly all or only a fraction of the total foundation settlement may have occurred before the end of construction in any embankment dam. There for the settlements are believed to further continue until the end of construction and later. The following are possible remedial measures suggested by the consultant to reinstate the healthy condition of the dam;

- i. Remedial (treatment measures)
  - Treatment from inside of conduit
  - Treatment from top of embankment level
  - Treatment from the conduit foundation level
- ii. Alternative outlet at the left abutment
- iii. Removing and replacing the conduit
- iv. Finishing the dam construction upto the dam crest level without implementing any remedial measure

Comparison of this different remedial measures has been made and the pros and cons were discussed in detail. Preliminary cost estimation had been also carried out for different proposed options. Therefore, as per the indicated advantages and disadvantages the consultant has discussed with the client and reach on the fourth option as the most attractive one listing the following justifications;

- This option might be attractive in timely completion of the project and less costly provided that the concrete constituting the outlet conduit is safe from stress developed from foundation deformation due to embankment loading
- Demands rigorous investigation on the current status of the concrete with non-destructive technology which needs an experienced and specialized company
- Treatment cost from embankment might be costly if deemed necessary

Currently the contractor is preceding its work to finish the dam construction up to the dam crest level without implementing any remedial measure.



Figure 1-1: View from upstream intersection of conduit with intake foundation

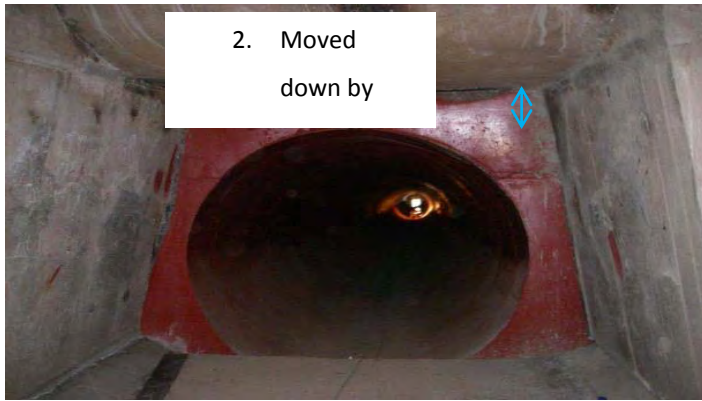


Figure1-2: Steel plate moved down as a result of conduit settlement

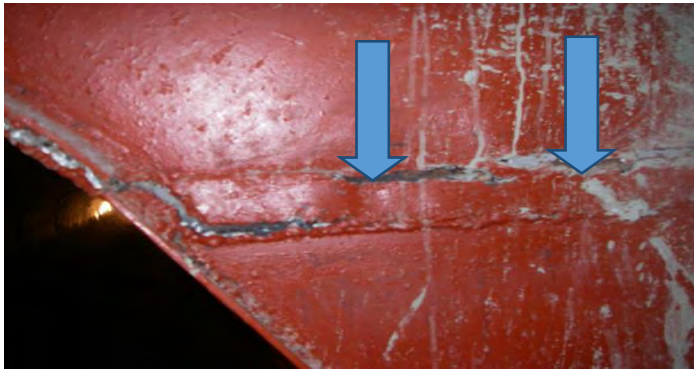


Figure 1-3: Effect of settlement on the welded front part of the conduit

Current dam status is discussed to emphasize the severity of the settlement problem faced during the construction stage of Gidabo dam and to justify that assuming failure by piping is reasonable. As discussed in this section measures were proposed to solve the problem and the final decision of the study concerning the settlement was to continue construction up to dam crest level and to treat the affected outlet. This decision is not a guarantee of further settlement stoppage, where if it occurs during the life time of the dam, as studied now, the outlet may still continue to be a direct victim of the situation. This may create detachment between the periphery of the outlet and the dam body where water may get loose parts to make a way and this may further lead to total failure of the dam.

### 1.2 Statement of the problem

The safety of dam cannot be secured only with provision of sufficient discharging capacity of the spillway or adequate hydraulic design of the whole dam because with all the necessary arrangements made still extreme cases like dam breaching should be expected and this research proposal is intended for such severe case. Breach analysis should be done for any dam in every certain period of time. Accurate prediction of breach parameters is crucial to

development of effective emergency action plans, design of early warning systems, and characterization of threats to lives and property.

In addition, it is discussed in the above section that the outlet is suffering from a considerable settlement of the foundation where if it continuous during the life time of the dam it is susceptible to failure by piping.

Simulation of such a dam break event and its resulting flood is crucial to characterizing and identifying threats due to potential failure. This is possible by preparation of inundation maps which accurately predict dam breach flood depths and arrival times at critical locations. But, Gidabo dam lacks the said studies. Hence, this thesis study is attempted to fill this research gap.

### **1.3 Objectives of the study**

#### **1.3.1 General Objective**

The general objective of this study is to model the dam breach process of the Gidabo dam and map the inundated area at the downstream.

#### **1.3.2 Specific Objective**

The specific objectives are:

1. To simulate the breach process applying different breach parameters
2. To run the model for different catastrophic scenarios and analyze the consequences of the most hazardous one.
3. To predict the outflow hydrograph just at the outlet of the breach.
4. To route the peak outflow hydrograph on downstream channel and map the area that shall be flooded.
5. To prepare various types of maps such as area, velocity, depth with time that can describe the nature of flood propagation and coverage.
6. Comparison of breach software results with HEC-RAS

### **1.4 Significance of the study**

Preparing dam safety plans and hazard management strategies are unquestionably vital, since lots of human lives have been lost and tremendous amount of economic crisis have been recorded from dam failure events throughout the world in history. 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.

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

- ✓ Flood early warning system can be developed after the disastrous dam breaching flood has been identified.
- ✓ 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.
- ✓ 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 factory plants.
- ✓ The thesis paper can be used as a reference for further detailed researches on the study area.

### **1.5 Scope of the study**

This thesis shall cover determination of dam breach parameters and catastrophic outflow hydrograph. Different scenarios of inflow, mode of failure and initial breach size will be analyzed and the one giving a worst condition shall be selected for further downstream routing and flooding.

From the known modes of failures, seismic type will not be considered in this study.

## 2. Literature review on dam breach analysis

### 2.1 Background

Dams are one of the most important structures in the field of water resources engineering. It is an evident historical fact that failure of dams have resulted in high casualties as indicated in the introduction part. 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.

### 2.2 Dam failures

In Ethiopia there is no dam breach event as far as concerned but some of the major dams are suffering from piping and structural stability problems which may lead to breach and considerable loss. To mention some among this dams Kesem is observed to have a  $1\text{m}^3/\text{s}$  flow along its left side abutment and Tendaho have a total leakage of nearly 500lts/s ( Tendaho dam leakage and associated problems, 2011).

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.

According to ICOLD 2012 ( International committee on large dams ), the various causes leading to the failure of earth dams can be grouped in to the following three classes;

- ✓ Hydraulic failure ( 40% )
- overtopping,
- erosion of upstream face,

- erosion of downstream face and
- erosion of toe of the dam
- ✓ Seepage failure ( 30% )
- Piping (internal erosion)
- Uplift
- Sloughing and subsequent failure of the dam
- ✓ Structural failure ( 25% )
- Embankment slope failures
- Foundation slide
- ✓ Others ( 5% )

### **2.3 Classification of hazard**

According to FEMA 2013 ( Federal emergency management agency ), 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 (Claudia C & Mark, 2010).

#### **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 Available dam break analysis methods**

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.

### **2.4.1 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**

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 (Wahl, 1997).

#### **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 2.2.

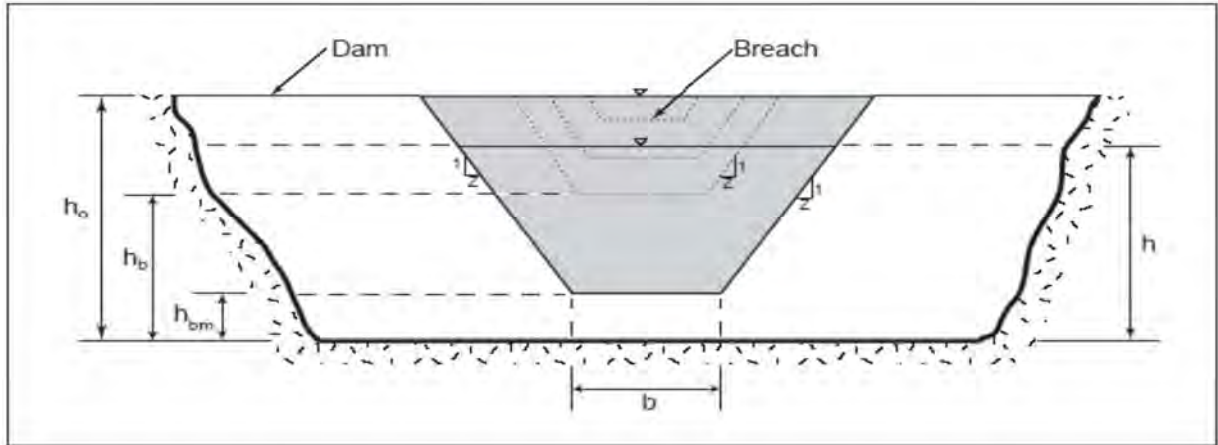


Figure 2-1 - Overtopping Breach Dimensions (Fread and Lewis, 1998)

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.

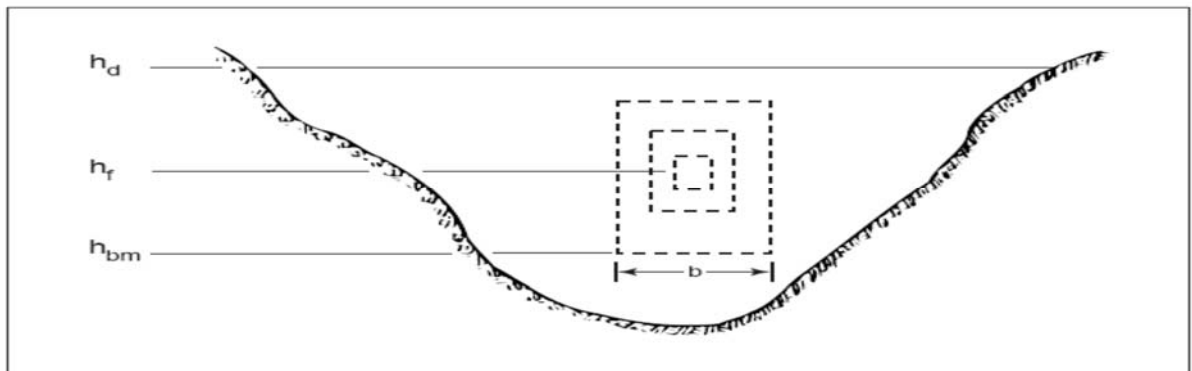


Figure 2-2 Piping Breach Dimensions (Fread, 1991)

Where:  $h_o$  = the breach top elevation at time  $t$

$h_d$  = elevation at the top of the dam

$h_b$  = the breach bottom elevation at time  $t$

$h_{bm}$  = lowest breach bottom elevation at time  $t$

$h_f$  = 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 (Morris, 2009).

#### **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. For the reasons explained above, these estimations are used as a preliminary estimate.

✓ **MacDonald et al**

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

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

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

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

$$t_r(\text{hr}) = 0.0179(V_{er})^{0.364}$$

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

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

$V_{eroded}$  = Volume of water eroded from the dam

$V_{out}$  = Volume of water that passes through the breach

$h_w$  = depth of water above the bottom of the breach

$h_b$  = Height of the dam to the bottom of the breach

$C$  = Crest width of the top of the dam

$$Z_3 = Z_1 + Z_2$$

$Z_1$  = Upstream Slope of the dam

$Z_2$  = Downstream Slope of the dam

$Z_b$  = 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 the peak flow equation that is developed from historic dam failures, by Mac Donald and langridge-Monopolis.

$$Q = 3.85(V_w h_w)^{0.411}$$

### ✓ Von Thun and Gillette

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

$$B_{avg}(m) = 2.5h_w + C_b$$

$$t_f(hr) = 0.015 * h_w \text{ for highly erodible dam}$$

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

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

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

Table 2-1: Values of  $C_b$  based on reservoir size

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

### ✓ 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 a set of equations to predict average breach width, side slopes and failure time. The data that Froehlich used for his regression analysis had the following ranges.

Height of the dams: 3.05 -92.96 meters

Volume of water at breach time: 0.0139 – 660.0 m<sup>3</sup> x 10<sup>6</sup>

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

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

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

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

$K_o$  = constant (1.3 for overtopping failures, 1.0 for piping)

$V_w$  = reservoir volume at time of failure (cubic meters)

$h_b$  = height of the final breach (meters)

$g$  = gravitational acceleration (9.80665 m/sec<sup>2</sup>)

$T_f$  = 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. Froehlich has suggested the peak flow as follows;

$$Q = 0.607V_w^{0.295}h_w^{1.24}$$

✓ **Xu and Zhng**

Height of the dams: 3.2 - 92.96 meters

Volume of water at breach time:  $0.105 - 660.0 \text{ m}^3 \times 10^6$

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

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

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

$V_w$  = reservoir volume at time of failure (cubic meters)

$h_b$  = height of the final breach (meters)

$h_d$  = height of the dam (meters)

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

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

$B_3$  = 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.

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

Where:  $B_t$  = Breach top width (meters)

$B_2$  = coefficient that is a function of dam properties

Breach side slopes can be computed with the following equation

$$Z = \frac{B_t - B_{ave}}{h_b}$$

The peak out flow discharge is suggested as follows,

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

**Orifice coefficient ( $C_p$ ):** The orifice coefficient defines flow through the piping hole before the models shift to weir flow as,

$$Q = C_p * A * \sqrt{2gH_p}$$

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

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

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

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

$$C_p = \sqrt{\frac{1}{1 + fL/D}}$$

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

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

**Weir Coefficients ( $C_w$ ):** after the dam crest collapses, the breach section erodes laterally and outflow through the breach is modeled as weir flow. During a piping mode, the configuration of the weir consists of a flat approach channel through the breach with a vertical drop-off to

the tail water section resulting from head-cutting during both the piping mode and weir flow. The maximum allowable weir coefficient for a broad-crested weir for all piping failure mode is 3.087.

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

#### ii. **Semi-Physically Based, Analytical and Parametric Models.**

These models are developed based on simplified assumptions of physical processes with an intention to reduce the large uncertainties associated with non-physically based models and to simplify the complex computational procedure in physically based models.

These models require the user to provide an erosion rate of the breach or final dimensions of the breach and time of failure of embankment. The model then predicts the growth pattern based on these user defined assumptions. Even though these models provide a more accurate flood hydrograph as compared to empirical equations, they are vulnerable to large degree of uncertainty due to the input data set.

#### iii. **Physically Based Models.**

These are numerical models that simulate failure of embankment dams based on the processes such as flow regimes, erosion and instability processes observed during failure time. Some of these models are BREACH , BRDAM, BEED, FLOW SIM1 and FLOW SIM2.

The advantages of physically based models are (Morris, 2009):

- The modeling incorporates aspects of hydraulics, sediment transport, soil mechanics and structural behavior.
- A real estimate of the breach process is predicted without predefining (constraining) the predicted growth process.
- Uncertainties within individual processes or parameters may be included within the model.

The breaching process is accomplished in two time phases. The initiation phase is the initial time at which overtopping or piping flow is observed. If no measures are taken to stop the water, erosion of embankment material will be triggered and any mitigation actions that are undertaken to stop will be valueless, this phase is called breach formation time.

It was observed from the collected historical dam failure events that in most of the cases the ultimate breach shape is of trapezoidal shape. Scholars concluded that for embankment dams, the breach shape can be assumed to be triangular up to the time that the base of the embankment is reached. Once the apex of the triangle reaches the foundation level, the breach develops forming a trapezoidal section extending due to lateral erosion (Wahl, 1998).

In spite of the incomplete understanding of breach formation processes and the limited capabilities of mathematical description of dam breaching mechanisms, various breach parameters estimation techniques are developed based on several assumptions. Some of these, along with their limitations, are discussed here under.

**i. Experimental method:** For better understanding the processes involved and to correctly describe the complex phenomena of embankment breaching, various field and laboratory experiments are carried out modeling gradual failure of dams and obtain data for calibration and validation of mathematical models. These field and laboratory results lead to extensive validation of the numerical models being used and under development (Zagonjoli, 2007).

**ii. Comparative Analysis:** If the dam under consideration is very similar in size and construction to a dam that failed, and the failure is well documented, appropriate breach parameters or peak outflows may be determined by comparison. This method is not highly reliable for large dams because of lack of accurate and comprehensive case study data on large dam (Wahl, 1998).

#### 2.4.2 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 are 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. From the physically based models, the model **BREACH** is presented below and the principles and steps it follows while analyzing the breaching process are discussed in detail.

### 2.4.3 Dam-Break maximum breaching outflow verification

After the analysis is completed and the hydrograph developed, it is necessary to check the reasonableness of the maximum breaching outflow  $Q_{max}$ . There are a few commonly known techniques to check  $Q_{max}$ : historical predictor equations, parametric models, physically based erosion methods, direct comparison techniques, customized prediction equations, classical equations, FERC recommended equations and current OES recommended equations. The rule of the thumb is to check  $Q_{max}$  obtained in one method with the result of the other techniques. In addition to the peak flow equations, one can also compare computed model peak outflows to envelope curves of historic failures. When comparing computed results to the envelope curve shown below keep in mind that this envelope curve was developed from only fourteen data sets, and may not be a true upper bound of peak flow versus hydraulic depth (USACE,2014).

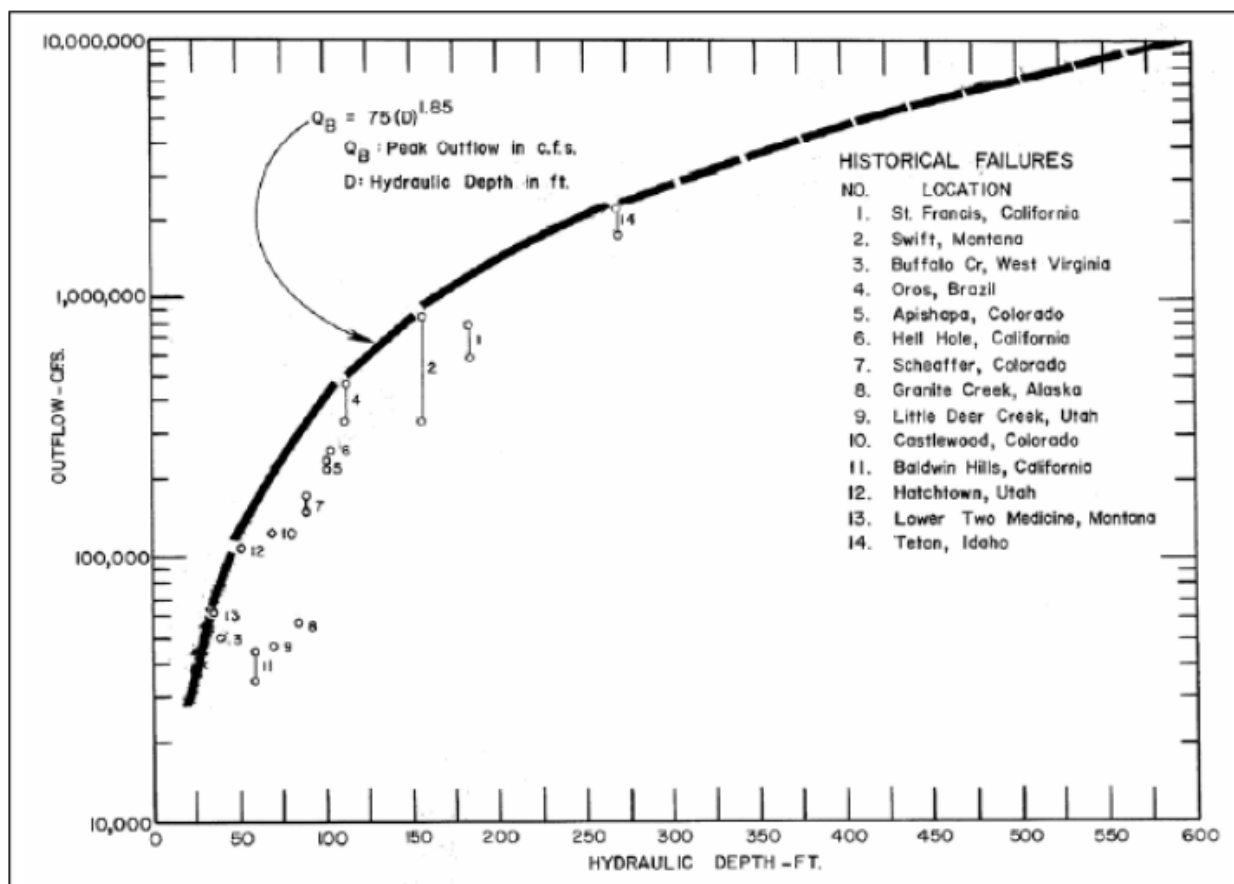


Figure 2-3 - Envelope of Experienced Outflow Rates from Breached Dams (USACE, 2014)

## 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:

$$Q_b = 3B_o(H - H_c)^{1.5}$$

where; H - elevation of the breach bottom

$Q_b$ - flow into the breach channel

$B_o$  – 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 ( $H_p$ ) 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,

$$Q_b = A \left[ \frac{2g(H - H_p)}{(1 + \frac{fL}{D})} \right]^{0.5}$$

in which;

$Q_b$  - flow (cfs) through the pipe

g - Gravity acceleration constant

A - Cross-sectional area (ft<sup>2</sup>) of the pipe channel

(H- $H_p$ ) - 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 ( $B_0$ ) is governed by the following relation:

$$B_0 = B_r y$$

in which  $B_r$  is a factor based on optimum channel hydraulic efficiency and  $y$  is the depth of flow in the breach channel. The parameter  $B_r$  has a value of 2 for overtopping failures while for piping failures,  $B_r$  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 ( $\alpha$ ) with the vertical, see Figure 2-1 Front 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 ( $\Phi$ ), cohesion ( $C$ ), and unit weight ( $\gamma$ ), i.e.

$$H'_k = \frac{4C \cos \Theta \sin \theta'_{k-1}}{\gamma [1 - \cos(\theta'_{k-1} - \Theta)]} \dots K = 1, 2, 3$$

where;

$K$  – Denotes three successive collapse conditions

$\Theta$  – 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.

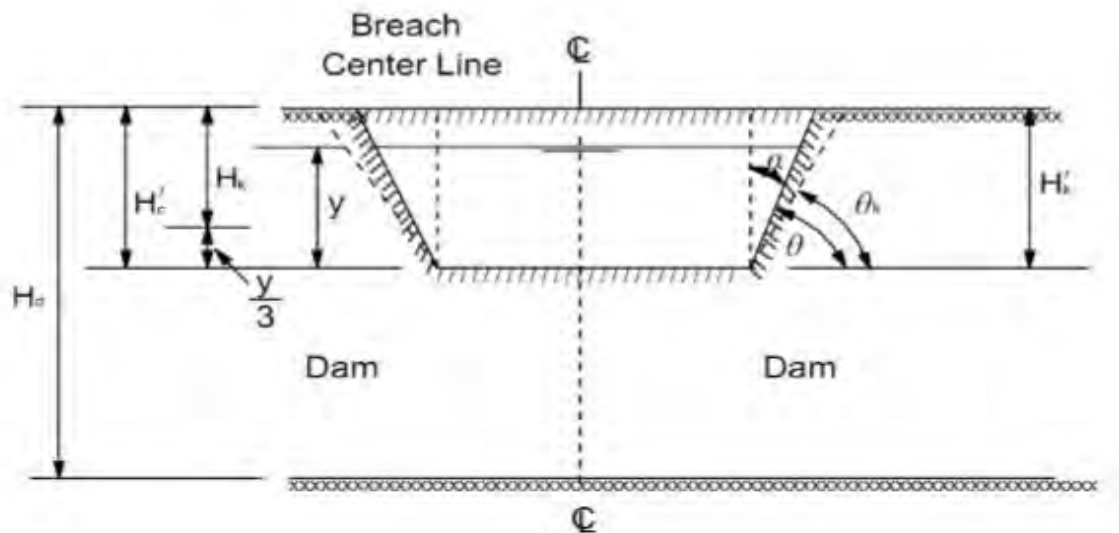


Figure 2-4 Front View of Dam with Breach (Fread D. , 1991)

## 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 ( $Q_i$ ), spillway outflow ( $Q_{sp}$ ), crest overflow ( $Q_o$ ), breach outflow ( $Q_b$ ), and the reservoir storage characteristics. The conservation of mass over a time step ( $\Delta t$ ) in hours is represented by the following:

$$Q_i - (Q_b + Q_{sp} + Q_o) = S_a = \frac{\Delta H 43560}{\Delta t 3600}$$

In which  $\Delta H$  is the change in water surface elevation during the time interval  $\Delta t$ , and  $S_a$  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  $t - \Delta t$ .

## 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 each  $\Delta t$  time

step, i.e.

$$Q_b = \frac{1.49S^{0.5}A^{1.67}}{nP^{0.67}}$$

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.,

$$Q_s = 3.64 \left( \frac{D_{90}}{D_{30}} \right)^{0.2} P \frac{D^{\frac{2}{3}}}{n} S^{1.1} (DS - \Omega)$$

in which  $Q_s$  is the sediment transport rate (cfs);  $D_{30}$ ,  $D_{50}$ ,  $D_{90}$  (mm) are grain sizes at which 30, 50, and 90 percent of the 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 BREACH 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  $\Delta t$  time step during the

simulation. The collapse check consists of assuming an initial value for  $Y_c$  of 10 and then summing the forces acting on the wedge of height,  $Y_c$ . The forces are those due to the water pressure ( $F_w$ ) and the resisting forces which are the shear force ( $F_{sb}$ ) acting along the bottom of the wedge, the shear force ( $F_{ss}$ ) acting along both sides of the wedge, the force ( $F_{cb}$ ) due to cohesion along the wedge bottom and ( $F_{cs}$ ), the force due to cohesion acting along the sides of the wedge, Figure 2 . Thus, collapse occurs if:

$$F_w > F_{sb} + F_{ss} + F_{cb} + F_{cs}$$

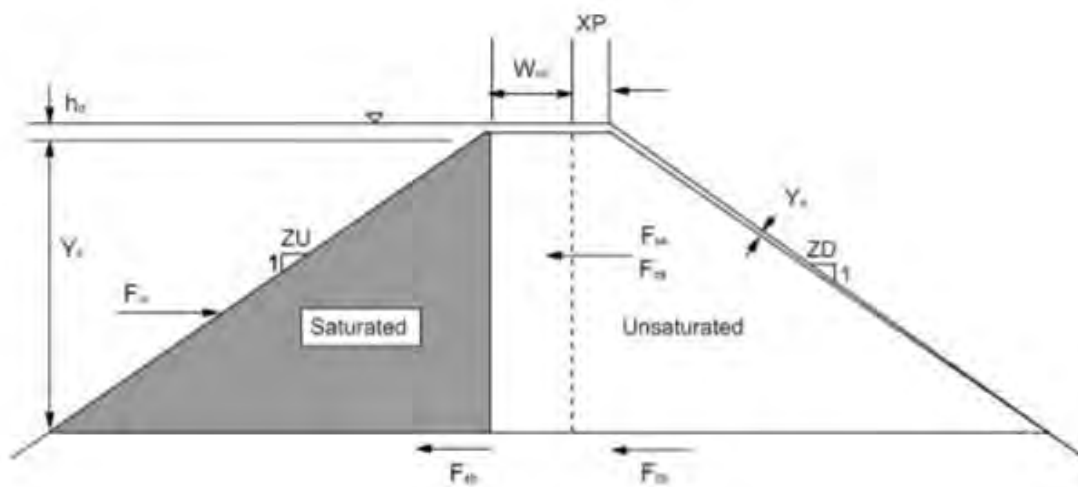


Figure 2-5 Forces Determining the Possible Collapse of the Upper Portion of the Dam (Fread D.1991)

## Vi. Computational Algorithm

The sequence of computations in the model are iterative since the flow into the breach is dependent on the bottom elevation of the breach and its width while the breach properties are dependent on the sediment transport capacity of the breach flow, and the transport capacity is dependent on the breach size and flow.

A simple iterative algorithm is used to account for the mutual dependence of the flow, erosion and breach properties. The computational procedures are summarized below and a flow chart showing structure of the algorithm is given on Figure 2-6.

Step 1 make initial estimates on incremental erosion depth,  $\Delta H_c'$ , change in reservoir elevation,  $\Delta H'$  and increase the time to  $t=t'+\Delta t$ .

Step 2 Compute  $H_c = H'_c - \Delta H'_c$  and  $H = H' + \Delta H'$

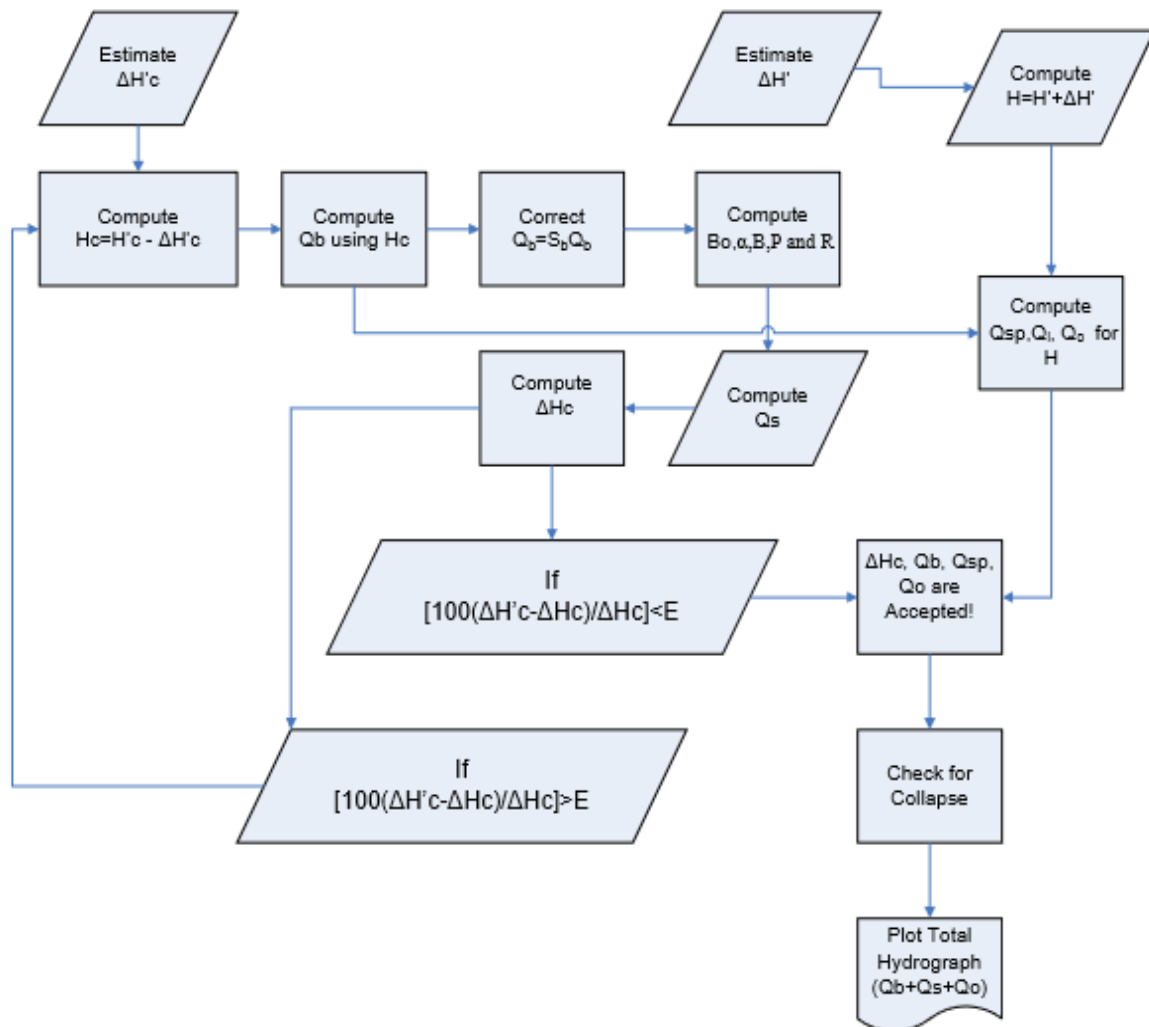


Figure 2-6 Process Flow Diagram

Step 3 Compute  $Q_{sp}, Q_i, Q_o$  for  $H$

Step 4 Compute  $\Delta H$  using equation of conservation of mass and previously computed  $Q_b$

Step 5 Compute  $H = H' + \Delta H$

Step 6 Compute  $Q_b$  using either weir or orifice equations

Step 7 Correct breach flow,  $Q_b$  for downstream submergence;

$$Q_b = S_b Q_{b0}$$

$$\text{where: } S_b = 1 - 2.78 \left( \frac{y_t - H_c}{H - H_c} - 0.67 \right)^3$$

In which  $Y_t$  is tail water depth and is computed using Manning's equation applied to the tail water cross section.

Step 8 Compute breach parameters;  $B_o$ ,  $\alpha$ ,  $B$ ,  $P$  and  $R$  and sediment transport  $Q_s$ .

Step 9 Compute  $\Delta H_c$  using;

$$\Delta H_c = \frac{3600 \Delta t Q_s}{[P_o L (1 - P_{or})]}$$

Where;  $L$  is length of breach channel,  $P_{or}$  is porosity of breach material and  $P_o$  is total perimeter of the breach given by;

$$P_o = B_o + \frac{2(H_u - H_c)}{\cos \alpha}$$

Step 10 Compare  $\Delta H_c$  with initially estimated  $\Delta H_c'$  using error tolerance  $E$  (between 0.1 and 1.0); if

$$\frac{100(\Delta H_c' - \Delta H_c)}{\Delta H_c} < E$$

Then, the solution for  $\Delta H_c$  and associated values of  $Q_b$ ,  $Q_s$  and  $Q_o$  are accepted!

If the inequality is not satisfied, all the above steps are repeated using the newly computed  $\Delta H_c$  as a new estimate in place of  $\Delta H_c'$ . The cycle is repeated until convergence is attained.

Step 11 Plot the total outflow hydrograph, ( $Q_b + Q_s + Q_o$ )

## 2.4.4 General approaches to dam break analysis

### 2.4.3.1 Event-Based approach

An event-based approach is a deterministic method that requires the use of a specific or series of specific precipitation and non-precipitation events for the evaluation of dam failure and downstream inundation mapping. These events include extreme rainfall and runoff events that can lead to natural floods of variable magnitude. The maximum flood for which a dam is to be designed or evaluated is often dependent on its existing hazard potential classification or size classification (FEMA, 2013).

Typically, several hydrologic and non-hydrologic (fair weather) events are evaluated as part of an event-based dam safety analysis. For hydrologic failure events, an extreme flood event ranging from the 50-year event for low-hazard dams up to the PMF for high-hazard dams is selected based on the potential for loss of life due to a dam failure or for significant economic and environmental losses. Typically, the hazard potential classification of the dam is used to select the extreme hydrologic failure event. The PMF is the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the drainage basin under study. The Probable Maximum Precipitation (PMP) is an estimate of the maximum possible precipitation depth over a given size catchment for a given length of time (Stedinger et al. 1996).

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

#### **a. Fair Weather (Non-Hydrologic) Failure**

As defined by FEMA (2013) a fair weather (Sunny Day) breach is a dam failure that occurs during fair weather (i.e., non-hydrologic or non-precipitation) conditions. A fair weather breach is analyzed by establishing an initial reservoir water level and commencing a breach analysis without additional inflow from a storm event. A fair weather breach is typically used to model piping failures for hydrologic, geologic, structural, seismic, and human-influenced failure modes.

Base flow conditions for a fair weather failure are typically ignored because of the small discharge and volume compared to that of a dam breach. As a general guidance, base flow can be ignored if the dam breach flow is two times greater than the base flow. Where base flow is considered, the discharge is typically estimated based on reported base flows through the dam's outlet works or from stream gage records. The three most common initial water level elevations for fair weather breach analyses are;

- ✓ Normal Pool Elevation (invert of the highest elevation of the primary outlet)

A breach at the normal pool elevation of the reservoir is used to estimate the volume and associated breach discharge that would result from a failure event during fair weather

conditions. For an embankment dam, this type of event is modeled as piping/internal erosion failure, whereas for a concrete dam, this event is modeled as a monolith collapse resulting from sliding, foundation instabilities, or a seismic event.

- ✓ Invert of Auxiliary Spillway (lowest uncontrolled spillway)

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

- ✓ Top of Dam / Maximum High Pool

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

Various Federal agency publications provide guidance for establishing the initial water surface elevation of a reservoir during a fair weather failure event. Each of these specified elevations is used to characterize different failure modes as well as the potential volume of the reservoir at the time of failure. The normal pool elevation is recommended as the default volume for the fair weather failure. States should consider a larger storage volume for dams where the primary and emergency spillway systems are considered susceptible to blockage resulting in a higher water surface elevation and volume during a non-hydrologic event.

## **b. Hydrologic Failure**

Hydrologic breaches that occur with extreme precipitation and runoff are termed “rainy day” or hydrologic failures. Hydrologic failures that cause dam breach events are generally analyzed based on the IDF established by the dam’s hazard potential and hazard size classification, typically a PMF for high-hazard potential dams. For significant-hazard potential dams, the breach event may include a breach of the PMF and IDF that could range

from the 1-percent-annual-chance flood event (often called the 100-year flood) to a percentage of the PMF (FEMA, 2013).

#### **2.4.4.2 Risk-Based (Consequences-Based) approach**

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

A benefit of the risk-based approach is that it may demonstrate, via an incremental damage assessment, that areas located downstream of a dam may be marginally affected by the reduction in the SDF or IDF design standard for a dam. By lowering the SDF or IDF requirements, limited funding for needed rehabilitation measures can be used for more dams, resulting in an overall increase in dam safety.

A disadvantage of the risk-based approach is that by reducing the SDF or IDF to less than the full PMF based on downstream consequences, new development in the downstream breach inundation zone could alter the consequences, resulting in the need for future dam rehabilitation measures to increase spillway capacity. Effective risk communication as a component of the local development approval process can assist in reducing the occurrence of “hazard creep,” an occurrence where new downstream development in a dam breach inundation zone increases the dam’s hazard potential classification or SDF/IDF design requirement (FEMA, 2013).

#### **Inflow Design Flood and the Incremental Hazard Evaluation**

IDF is defined as “the flood flow above which the incremental increase in water surface elevation downstream due to failure of a dam or other water retaining structure is no longer considered to present an unacceptable additional downstream threat” (FEMA, 2004). Therefore, incremental hazard evaluation and the establishment of the IDF is, in essence, a risk-based approach.

The selection of the IDF is based on the evaluation of the magnitude of several flood events (FEMA, 2013). The incremental hazard evaluation begins with simulation of a dam failure during a hydrologic flooding condition, typically beginning with the PMF or percentage of the PMF as specified by the State hazard potential classification requirements. The same

hydrologic event is then run for non-failure conditions. The water surface elevations for both the breach and non-breach events are compared to determine the increase in the water surface elevation resulting from the dam breach. If the incremental increase in downstream water surface elevation between the failure and non-failure scenarios results in an acceptable increase in consequences, ( as defined by State requirements) a smaller percentage of the PMF flood inflow or other magnitude flood is then used to repeat the process. The process is repeated until the incremental increase in consequences due to failure falls within acceptable requirements.

Once the appropriate IDF for the dam has been selected, the IDF is then routed through the dam to determine whether the flood can be safely passed without failure. Should the IDF pass safely, then no further evaluation or action is required; however, if the IDF cannot pass safely, then measures must be taken to enable the project to safely accommodate all floods up to the IDF to alleviate the incremental increase in unacceptable additional consequences a failure may have on areas downstream.

New guidance in Selecting and Accommodating Inflow Design Floods for Dams includes the recommendations shown in Table 2.1.

Hazard Potential Classification	Definition of Hazard Potential Classification	Inflow Design Flood
High	Probable loss of life due to dam failure or disoperation	PMF <sup>(1)</sup>
Significant	No probable loss of human life but can cause economic loss, environmental damage, or disruption of lifeline facilities due to dam failure or disoperation	0.1-percent-annual-chance exceedance flood (1,000-year flood) <sup>(2)</sup>
Low	No probable loss of human life and low economic and/or environmental losses due to dam failure or disoperation	1-percent-annual-chance exceedance flood (100-year flood)

Table 2-2: Recommended IDF Requirements for Dams Using Prescriptive Approach (FEMA, 2012)

PMF = Probable maximum flood

<sup>(1)</sup> Incremental consequence analysis, risk-informed decision making, or site-specific PMP studies may be used to evaluate the potential for selecting an IDF lower than the prescribed minimum. An IDF less than the 0.2-percent-annual-chance exceedance flood (500-year flood) are not recommended.

<sup>(2)</sup> Incremental consequence analysis or risk-informed decision making studies may be used to evaluate the potential for selecting an IDF lower than the prescribed minimum. An IDF less than the 1-percent-annual-chance exceedance flood (100-year flood) are not recommended.

#### **2.4.4.3 Tiered dam breach analysis**

A tiered approach to dam breach analyses can be used to establish an initial dam hazard potential classification and to produce dam breach inundation zone mapping for EAPs. The tiered dam breach analysis structure is not appropriate for use in dam design (FEMA, 2013).

A tiered study approach was developed by the USDOJ and is presented in their report titled Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report (USDOJ, 2011). The tiered dam breach analysis approach presented in this document adapts the USDOJ approach and provides additional detail.

The NDRSB EAP Workgroup (2009) noted that the cost of detailed dam breach studies is consistently cited as the primary impediment to EAP development and, therefore, many States have adopted a form of simplified and conservative inundation maps for use in EAPs. The NDRSB EAP Workgroup also stated that although detailed studies often provide a more precise representation of potential flooding for a given set of assumptions, a more accurate representation of dam failure flooding is not necessarily provided (FEMA, 2013).

In their effort to increase the number of EAPs for dams, a tiered approach in dam inundation modeling has gained popularity with many State and Federal dam safety programs. Unlike the event- and risk-based approaches discussed, the tiered approach is not used to determine the appropriate flood event to use in a dam failure analysis. Instead, the tiered approach is used to determine the appropriate level of complexity in the assessment, modeling, and mapping of a dam failure based on a dam's hazard potential, size, and the complexity of the downstream area under investigation.

The level of analysis for the tiered approach should correlate the sophistication and accuracy of the analyses with the scale and complexity of the dam and downstream area under investigation. Therefore, analysis of high-hazard potential dams located upstream of populated areas or complex floodplains should use more sophisticated modeling and additional sensitivity studies to properly assess the consequences of a dam failure; whereas,

analysis of low-hazard potential dams situated upstream of sparsely populated areas may rely on more approximate methods of analyses.

In general, as the sophistication of the modeling increases, so does the level of effort, time, and cost necessary to conduct the analysis. Table 2-2 provides guidance to determine the tier level for analysis for dam failure inundation modeling and mapping. The dam failure analysis should be continued downstream to a point where the breach flood no longer poses a risk to life and property damage, such as the confluence with a large river or reservoir with the capacity to store the flood waters.

Tier Level	Applicable to	Breach Parameter Prediction	Peak Breach Discharge Prediction	Downstream Routing of Breach Hydrograph
Tier 1 –Basic level Screening and Simple Analysis	<ul style="list-style-type: none"> <li>• Low-hazard potential / small size</li> <li>• First level screening for significant- or high-hazard dams</li> </ul>	Empirical Equations	Simplified Models (SMPDBK, GeoDam-BREACH, or Technical Release [TR]-66) or HEC-HMS	GeoDam-BREACH, SMPDBK, DSAT,1D HEC-RAS Steady State, or HEC-HMS Hydrologic Routing
Tier 2 –Intermediate	<ul style="list-style-type: none"> <li>• Significant-hazard potential / intermediate size</li> <li>• High-hazard dams with limited population at risk</li> </ul>	Empirical Equations	HEC-HMS or HEC-RAS Unsteady Model	HEC-RAS (Steady or Unsteady Modeling) 1-D or 2-D models
Tier 3 –Advanced	<ul style="list-style-type: none"> <li>• High-hazard potential / large size dams with sufficient population at risk to justify advanced analyses</li> </ul>	Empirical Equations, NWS BREACH, or WinDAM	HEC-RAS Unsteady Model	HEC-RAS Unsteady Model or 2-D models

Table 2-3: Tiered Approach Dam Breach Inundation Mapping for use in EAPs (FEMA, 2013)

Tier 1 and 2 analyses are most appropriate for low-hazard potential / small sized and significant-hazard potential / intermediate-sized dams with a limited number of structures. More detailed surveying or modeling may be warranted for Tier 3 analyses for high-hazard potential / large-sized dams, those with a large population in the evacuation area, or those with significant downstream hydraulic complexities.

## 2.5 Review of selected model

### 2.5.1 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 simulate unsteady flood wave propagation and identifies flood prone areas as the areas where the ground is lower than the computed water elevation and allows the user to visualize the flood propagation in real time, thus making the software ideal for dam breach modeling.

The HEC-RAS 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 (ASCE, 2014).

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-GeoRAS which is an Arc-GIS extension, (Sredojevic and Simonovic, 2009).

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 Gidabo dam break.

### **2.5.2 HEC-GeoRAS**

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.

### 2.5.3 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.

### 2.5.4 The Breach model

**BREACH:** Fread (1988) developed the physically-based BREACH model to more realistically simulate breaches initiated by overtopping or piping. The model uses the Meyer-Peter and Muller sediment transport equation as modified by Smart (1984) for steep channels. The model permits specification of three different embankment materials: an inner core, an outer portion (downstream shell), and a vegetated cover or riprap protective layer on the downstream face of the dam. Flow through the breach section is determined by orifice or weir equations, and flow down the face of the dam is modeled as a quasi-steady uniform flow with roughness determined from the Strickler equation for Manning's  $n$ . The model uses a much simpler computational algorithm than that of Ponce and Tsivoglou (1981). The model accounts for spillway flows around man-made dams and includes the effect of tail water depth on breach outflows. The model introduces two structural mechanisms that may contribute to breach formation: the breach shape may be impacted by slope stability of the breach side slopes, and possible collapse of the upper portion of the dam by shear and sliding is analyzed.

Today, the BREACH model is probably the best known physically based model. The most commonly applied breach prediction method in practice is probably the use of the uniform breach formation rate routines contained in DAMBRK and other models.

## 2.6 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.

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).

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 (Zagonjolti, 2007).

### **3. Methodology**

#### **3.1 General description of the study area**

This chapter describes the methodology used in order to conduct the study. It describes the types of data collected and analysis and the reasons for why these methods were chosen in comparison to the other alternative methods. The chapter consists of four sections. The first section presents the research approach. The second outlines important Data and gathering, the third is about dam breach analysis procedures and the final section outlines flood hydrograph routing.

##### **3.1.1 Location**

The project area is located in the Abaya-Chamo sub-basin of the Rift Valley Lakes Basin situated in the southern part of Ethiopia, within the administrative Regions of Oromiya and SNNPR. To be more specific, it falls in Abaya district of Borena zone of Oromiya region and Dale district of Sidama zone of SNNPRS (near Dilla town to east of Lake Abaya). The project area lies in the low lands, very close to the Dure and Gola marsh. It lies approximately between 6°20' and 6° 25' N Latitude and 38° 05' and 38°10'E Longitude, a short distance to the east of Lake Abaya and just south of Gidabo river flood plain, and at an average elevation of 1190 m a.m.s.l. The upper part of the catchment is covered with thick bushes and shrubs. However middle catchment is occupied with small and scattered bushes and grasses. The lower part mainly comprises of delta and swamp area and is covered with tall grasses. The proposed project area is situated in this part. Some parts of this area serve as dry season grazing for pastoralist.

The irrigation command area as found from the earlier studies may be located about 32 Km from Dilla town in Dibicha PA of Abaya woreda and Bokito PA of Dale districts. Most of the command area at present is covered with savanna grasses and bushes. It is at present a grazing area for pastoralist and agriculture is seen in the area.

##### **3.1.2 Climatic characteristics**

Most of Ethiopia is characterized by tropical climate moderated by altitude with a marked wet season. The southwestern Ethiopia is characterized by humid tropical climate with rainfall higher than 1,000 mm. In the highlands of Ethiopia, temperatures are reasonably warm the year round but rarely hot.

In Ethiopia in general there are three seasons: the first is the dry season (locally known as Bega) which prevails from October to January; the second is the small rainy season (Belg) that runs from February to May and the third is the main rainy season (Kiremt) which prevails from June to September. Rainfall is above 1,000 mm a year almost everywhere in the highlands and it rises to as much as 2,000 – 3,000 mm in the wetter southwestern parts. Annual rainfall decreases when one moves to the east and north of the country. Night time temperatures fall nearly or below freezing in mountainous areas (higher than 2,500 m). In the northern lowlands, Danakil depression, the southern lowlands, and Ogaden, rainfall is low (below 300 mm/year) and temperatures are high (higher than  $> 30^{\circ}\text{C}$ ) during the whole year. Since Ethiopia is situated in the north-eastern part of Africa, it is influenced from the northeast to the Southeast by monsoons bringing moisture from the Indian Ocean. In the northern hemisphere summer, moisture laden winds gradually penetrate into the countries as the African sector of the Inter-Tropical Convergence Zone (ITCZ) progresses northward. The orographic influence on rainfall depth is also marked in the mountainous areas that surround the Project. As a result, rainfall varies highly both seasonally and annually. Rainfall over Megech watershed is mono-modal. Nearly 79% of the annual rainfall occurs in the period of June-September. Annual variations of rainfall are high, with an average of 1,091 mm, a minimum of 710 mm and a maximum of 1,510 mm.

According to the study by federal water works design and supervision enterprise, the monthly mean rainfall in the project area varies from 23 mm in December to 117 mm in April. While monthly mean minimum temperature varies from  $12^{\circ}\text{C}$  to  $16^{\circ}\text{C}$  and monthly mean maximum temperature varies from  $28^{\circ}\text{C}$  in July to  $32^{\circ}\text{C}$  in March. According to available climatic data, annual mean evapo-transpiration amounts to 1623 mm, whereas annual average rain fall totals about 745 mm. The annual mean sunshine hour is 7.4 while the mean annual wind speed is 1.4 m/s.

### **3.1.3 Gidabo watershed**

The upstream watershed of the proposed Gidabo reservoir is highly degraded and easily erodable. Particularly, the sub basins of Kilkile, Raba and Amaleka streams require immediate action to restore the degraded parts. Integrated watershed management including area closure, constructing terraces, sediment check dams, gully reclamation and planting shrubs and grasses to increase ground cover would be very important to reduce siltation

problem of the reservoir. Annual load of sediment at Gidabo River is found to be 96,834.5 tones (WWDSE Gidabo hydrology report, Nov 2008).

### 3.1.4 Gidabo river

The intended primary source of water supply to the project under study is the Gidabo River. This river is composed of numerous tributaries originating in the mountain ranges enclosing the river basin. The main stem of the river, the Gidabo, flowing from the northeast is gauged near the town of Aposto, where it crosses high way (the main road to Dila). The mean flow of the river at this station is estimated to be 17.35 M<sup>3</sup>/sec while the mean minimum flow is 3.7 M<sup>3</sup>/sec (WWDSE Gidabo hydrology report, Nov 2008).

### 3.1.5 Land use

The project area is covered with bushes and shrubs with few patches of grass land. The wet and dry season grazing areas are Bolle (near Lake Abaya) and Aleme areas which are located some distance from the project area. The over all land use of Abaya woreda is presented in Table 3-1. (Federal water works design and supervision enterprise, November 2008)

Land use type	Area in ha	%
Farm land	44302.85	23.7
Forest	10105.2	5.4
Grazing land	63999.8	34.2
Water body	62925	33.6
others	5801.15	3.1
Total	187133	100

Table 3-1 The land use of Abaya woreda

There are different kinds of approaches that are used in conducting a given dam break research. These are sensitivity analysis on dam breach parameters, sensitivity of manning's n on breach out flow hydrographs, checking if a dam will breach or not, identifying possible mechanism of breach, identifying possible worst breach for a dam and preparing inundation maps and others.

Sensitivity analysis on dam breach parameters approach focuses primarily on analyzing the response of the peak discharge upon reduction and increment of TFH (time of failure in hours), SS(side slope) and other parameters. Evaluation of the relative influences of these two parameters specially, and other parameters on the maximum discharge is also the focus of such kind of studies.

Sensitivity upon a percentage reduction and increment of Manning's  $n$  to assess its effect on peak flows at the river stations is also another approach where different researchers have gone through and come up with relatively approximate results.

Different researchers have also tried to conduct a study on the probability for a certain dam to breach or not based on certain initial construction or design failure investigations. The other and most frequently applied approach is to assume a flood which is much greater than the design flood for the dam to breach and to examine the worst case scenario. This kind of studies is done specially to prepare emergency action plan prior to dam failure so that loss of life and property damage may greatly be minimized.

In this paper the researcher used the outlet construction failure as a possible reason for piping failure since the client has decided for the construction to proceed where a settlement of 30cms is observed and have potential to continue settling at the outlet entrance. The piping case have different scenarios and each scenario is treated independently. In addition to piping failure the researcher has included overtopping scenarios where a flood more than the spillway capacity is assumed to breach the dam and to analyze the possible inundation extents, depth and velocity of bursting flood.

### **3.2 Data gathering**

The correctness and accuracy of the data included in breach modeling must be considered in order to predict the failure impact analysis. Some necessary information needs to be collected to model dam breach and to determine the effects of a dam failure on the downstream flood plain. The main purpose of gathering data in creating a modeling methodology is for defining the size, type, elevation and storage relations of the subject dam, and the geometries of the downstream river reaches. This section identifies and discusses the types and source of data required for the study, and their analysis.

#### **3.2.1 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 (USGS department of the interior, 2010). This model is prepared in the

united states and the 30\*30 DEM is now available for the rest of the world on the website. 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. Although DEMs are providing unsubstitutable functions, they suffer from considerable accuracy faults and needs to be corrected based on manually collected elevation points.

For this research, A 30\*30m DEM by name n06\_e038\_1arc\_v3.tif, is downloaded from the [www.earthexplorer.usgs.com](http://www.earthexplorer.usgs.com) website. In addition to this, 370 manually collected points are gathered from WWDSE to compare and examine the DEM's elevation error and to develop correction factor to minimize DEM's error.

The correction factor is obtained by comparing this manually surveyed 370points with the corresponding dem points. Basically this two kinds has no difference in (x,y), the difference is the elevation. To find this 370 points on the dem, global mapper is used where the user is supposed to locate ( X,Y ) and the elevation will be extracted from the image. This set i.e (X,Y,Z) from the image are called to excel and subtracted from the manually collected points. The subtracted 370 results are averaged to reach at a single result. This averaged point is the final correction factor. This correction factor is applied on points which are extracted by HEC-GeoRAS and are send to HEC-RAS for further process. On HEC-RAS it is possible to see a tabulated list of this points and also to copy it on an excel sheet. After this points are exported to excel, they are corrected by the previously obtained correction factor. Finally this corrected points are copied to HEC-RAS for further analysis.

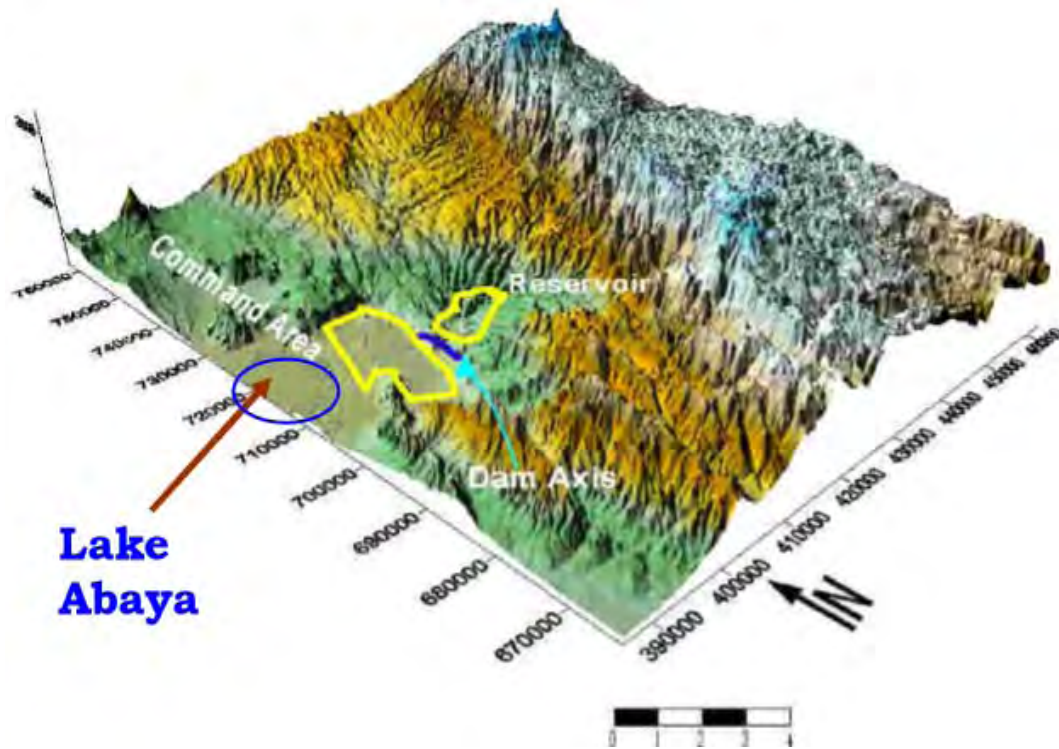


Figure 3-1 Physiographic Map of Gidabo Project Area

### 3.2.2 Hydrologic data

The hydrologic data category includes the flood events hydrograph (inflow hydrograph), PMF and reservoir capacity is provided by WWDSE.

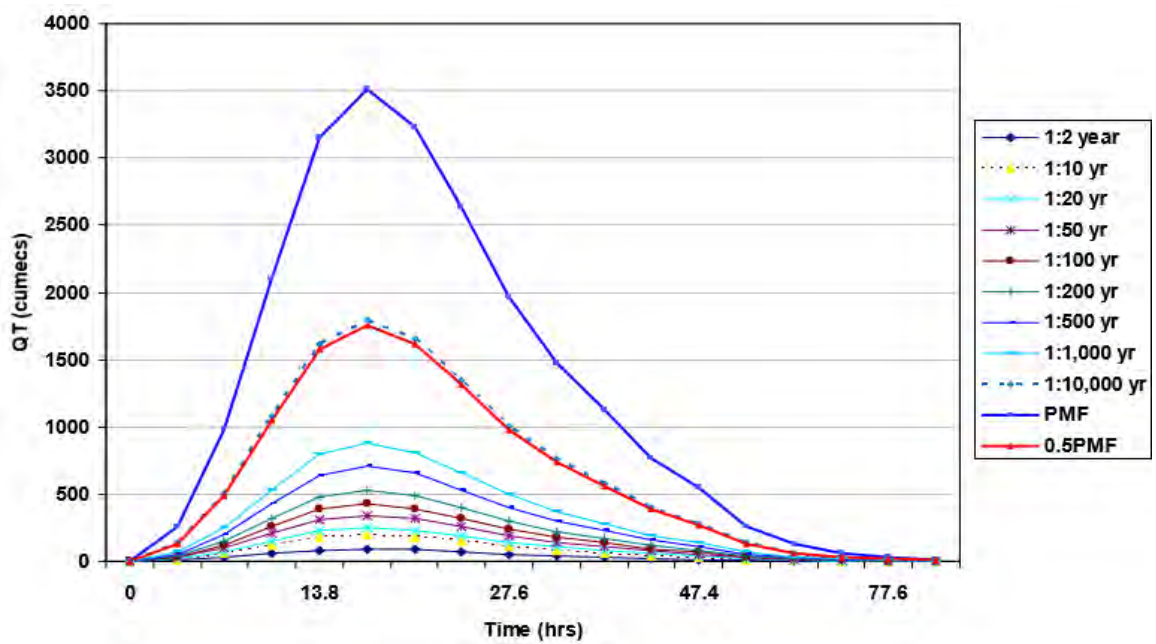


Figure 3-2 The inflow hydrograph to Gidabo dam for various return periods

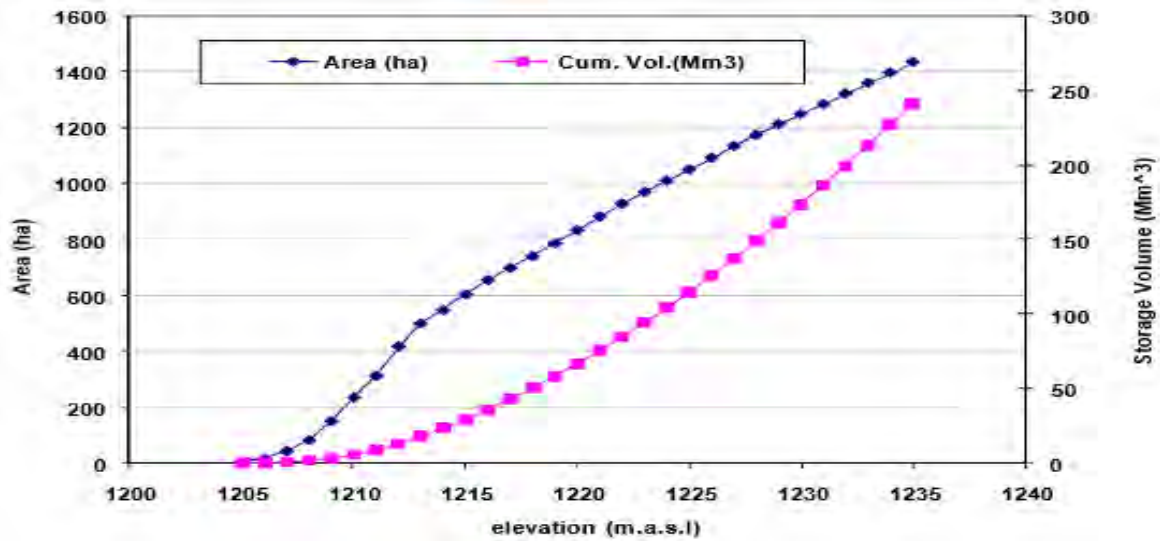


Figure 3-3 Elevation Area Volume curve of Gidabo dam site

### 3.2.3 Downstream command information

According to the irrigation study of WWDSE, the command area of Gidabo Irrigation Project is identified on both sides of the river course. On the left side 7113 ha area is lying and on the right side the total (gross) command area sums up to 2763 ha. Downstream of the dam is lying both an irrigable command and considerable swampy area. The catchment downstream of the dam is characterized by a terrain of relatively same elevation. The outlet of the catchment is at abaya lake, and a tributary river joins the main gidabo river before it enters Lake Abaya.

### 3.2.4 Population at risk

The target group for the project constitutes the people living in the command as well as the reservoir areas in Woredas, specifically people living in Dibicha laluncha Kebele of Abbaya Woreda, Oromiya region and Bukito Kebele of Loka Abbaya woreda of SNNPR.

Loka Abaya woreda is one of the new woredas of Sidama zone. The people in the command area are by large Sidama living in the woinadega area. The communities in the area do not have permanent residences, they usually build temporary shelter. The number of households according to the report of “the 2007 population and housing census of Ethiopia is 356 and the population is estimated to be 956 in both sexes, from which 453 were male and 503 were female.

Abbaya woreda is one of the eight woredas of Borena zone. The people in the command as well in the reservoir areas are by large Guji Oromos. The bukito kebele of Loka Abbaya woreda is found in the command and according to the study the population size is 322, where 159 were male and 163 were women.

In addition to the above discussed two woredas there are people in the command area who live in search of grazing land for their animals. This community do not have permanent residences, they usually build temporary shelter. At the time of the survey there were seven villages having 20 (minimum) to 56 (maximum) households, According to “the 2007 population and housing census of Ethiopia”.

Downstream risk analysis is not the focus of this research but in order to categorize and asses the dam’s hazard level its imperative to study downstream population size, main facilities and infrastructures. The large population size, the downstream irrigation structures and the different infrastructures provided for the community alone can't be used to classify the dam as high hazard dam. In fact the above listed points are basic points but a study of emergency action planning reveals if really the dam is high hazard dam or not.

### **3.3 General process of dam breach modeling**

The procedures followed in any dam breach modeling and inundation mapping process is more of the same. In this thesis, the researcher has first selected four famous and widely accepted empirical equations to determine breach parameters. Computation of the parameters by the equations and comparison among results is done. Using google earth trace river bank lines and stream centerlines 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.

In the course of this general procedure, a tributary with considerable discharge is identified to join the main river while breach flood is inundating downwards. To account for this river, the watershed is delineated using HEC-GeoHms, and the unit hydrograph is computed by;

One dimensional full dynamic flow routing is used for Gidabo dam breach analysis. 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, where as for piping mode of failure location is assumed to be at an elevation of 1209.55 where settlement is expected to form seepage line along the periphery of the outlet's external body and the dam.

### 3.3.1 Hydraulic model development

The HEC-RAS hydraulic modeling software execute unsteady flow river routing of dam breach hydrographs through the downstream channel. For this analysis process, hydraulic model (Hec-Ras 5.0) is used. In addition to this hydraulic modeling Arc-Map 10.0 is used for all GIS related tasks. Hec-GeoRas serves as an interface between GIS and Hec-Ras.

### 3.3.2 Hec-Geo Ras development

The modeling domain for the purpose of this thesis study is composed of Gidabo dam and its reservoir at the upstream boundary, the downstream most cross-section of gidabo river up to end of Abaya lake. The reach starts at reservoir full extents 12.8kms far from the dam. 5.8kms below the dam another tributary joins the river to make a confluence point and to flow to an outlet at abaya lake 17.8kms far from their joining point.

Hence, the modeling reach is made up of 12.8kms upstream of the dam, 6Kms downstream of the dam upto a tributary confluence point and 17.93kms downstream of this confluence point upto an outlet at lake abaya for a total of 23.9kms. There are several Agricultural Villages, Sub-Heads, Head-Quarter and towns located in the vicinity of the dam command. The following figure presents the HEC-RAS model setup for case study dam and settlements under flooding risks.

The Hec-GeoRas tool creates a file of geometric data for import into Hec-Ras and enables viewing of exported results from Ras. The import file is created from data extracted from data sets(Arc-Gis layers) and from a digital terrain model (DTM). Hec-GeoRas requires a DTM represented by a triangular irregular network tin (TIN) or a GRID. The layers and DTM are referred collectively as the Ras layers. Geometric data are developed based on the intersection of the Ras layers.

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.

### 3.3.2.1 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. 174 cross sections were taken at an approximate interval of 150m. 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.

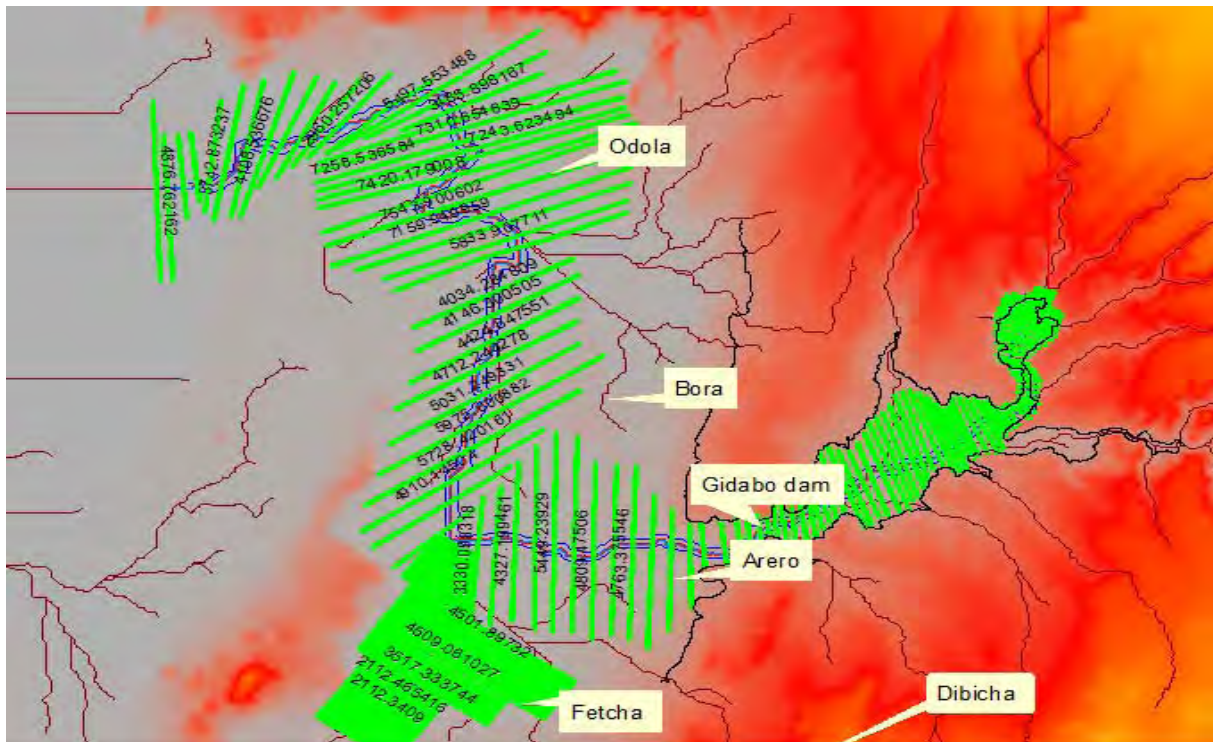


Figure 3-4 Gidabo river geometry created by Hec-Geo Ras

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.

### 3.3.2.2 Manning's roughness coefficient

It's difficult to assign a specific representative friction slope value even for small areas. Especially when we come to natural channels it's possible to find a wide range of land use land cover types within small reaches. It is because of this fact that some dam breach analysis studies are focused on sensitivity analysis of the peak breaching flood upon increasing or decreasing the manning coefficient.

Chow (1959), however, suggests that roughness coefficient for major streams (i.e. surface width at flood stage more than 100 feet~30.48m) is usually less than for minor streams (surface width less than 100 feet) of similar description on account of less effective resistance

offered by irregular banks or vegetation on banks and adds that values of “n” may be somewhat reduced.

The Manning roughness value ranges suggested by Chow (1959) for main channels and the floodplains in case of “Minor Streams” were identified as follows:

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

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

**Flood plains (adjacent to natural streams)**

Light brush and trees:

a) Winter.....0.050-0.060

b) Summer.....0.060-0.080

In Gidabo dam’s case, the smallest top-width at flood stage is quite approximate to 40-50m (WWDSE, gidabo hydrology study). 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. Taking into consideration the top-width only the manning roughness will have a higher amount but to be reasonable it is important to take into consideration the considerable shrubs coverage which when considered alone may lower the roughness value to lesser amounts. A trade of between these two justifications has led the researcher to adopt a roughness value of 0.04 for main channels and 0.06 for flood plains.

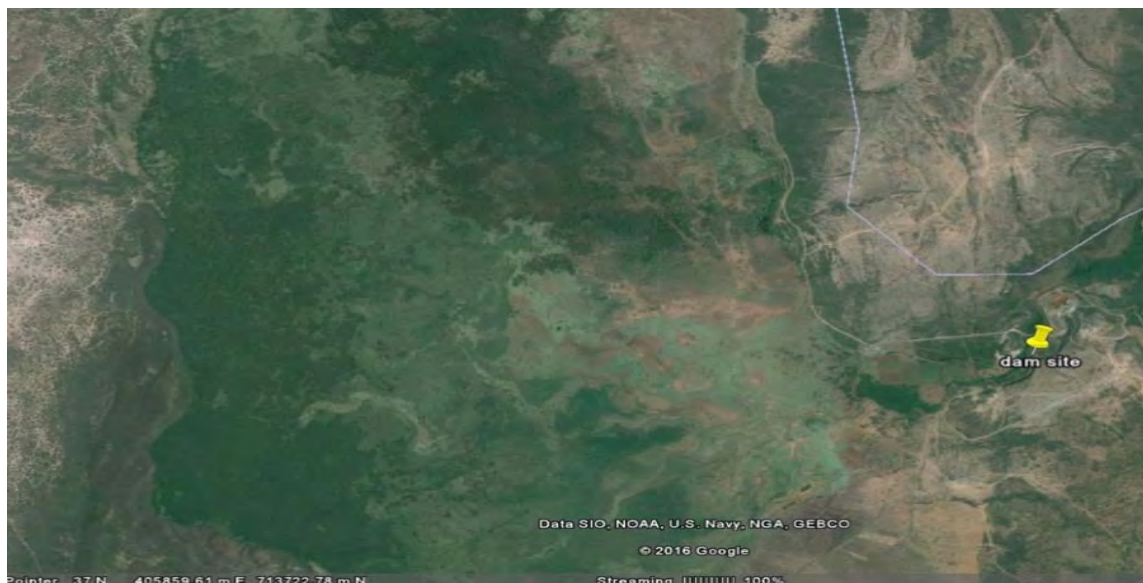


Figure 3-5google earth aerial photo

### 3.3.3 HEC-RAS development

#### 3.3.3.1 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 four abilities use 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 different breach parameter results 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.

#### 3.3.3.2 Failure initiation criteria

As has been discussed in literature review generally 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.

As for the piping failure cases, the failure initiation criteria is selected as a critical WSEL of 1219.5 m, which is equal to Gidabo dam's normal water level. Currently the intake and outlet structures are built as designed and now the outlet is found settled by 23cm on 30/12/2015 and by 31cm on 13/1/2016. In addition the contractor is allowed to continue its work to finish the dam construction up to the dam crest level without implementing any remedial measure as discussed in the introduction part. The "Gidabo outlet settlement investigation report"

studied and prepared by WWDSE, clearly states that the outlet may continue to settle considerably. This conclusion and the continuation of the work up to dam crest level can be taken as an input to assume possible sunny day failure.

As for overtopping, HEC-RAS user's manual (HEC, 2016), there are two alternatives to the user as failure initiation criteria. The user can either specify a time at which the dam failure will commence or a certain critical height of water above the crest elevation of dam, which will indicate commencement of dam failure if the elevation of water trapped behind the reservoir, exceeds it. In this study, the option of specifying a critical height of water above the dam crest elevation, which will be referred to as “critical WSEL” hereinafter, is preferred.

### **3.3.3.3 Boundary conditions**

For solving the unsteady flow equations in HEC-RAS, boundary conditions need to be defined. There are two external boundary conditions for the gidabo 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 is done by establishing an initial reservoir water level (i.e normal pool elevation) and commencing a breach analysis without additional inflow for piping and considering the PMF for overtopping for both main gidabo river and tributary fetcha river. Downstream boundary condition for gidabo is set to normal depth with a slope of 0.002 for both overtopping and piping scenarios.

### **3.3.3.4 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 400m (which is between 1223.8m/FRL and 396m/MDDL) for gidabo dam reservoir; and initial flows of

1391.94m<sup>3</sup>/s for upper Reach, 454.79m<sup>3</sup>/s for tributary and total flow 1846.739m<sup>3</sup>/s for lower Reach 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.

### 3.3.3.5 Geometry Data

The following figure shows the geometry in HEC\_RAS immediately after export from HEC-GeoRAS.

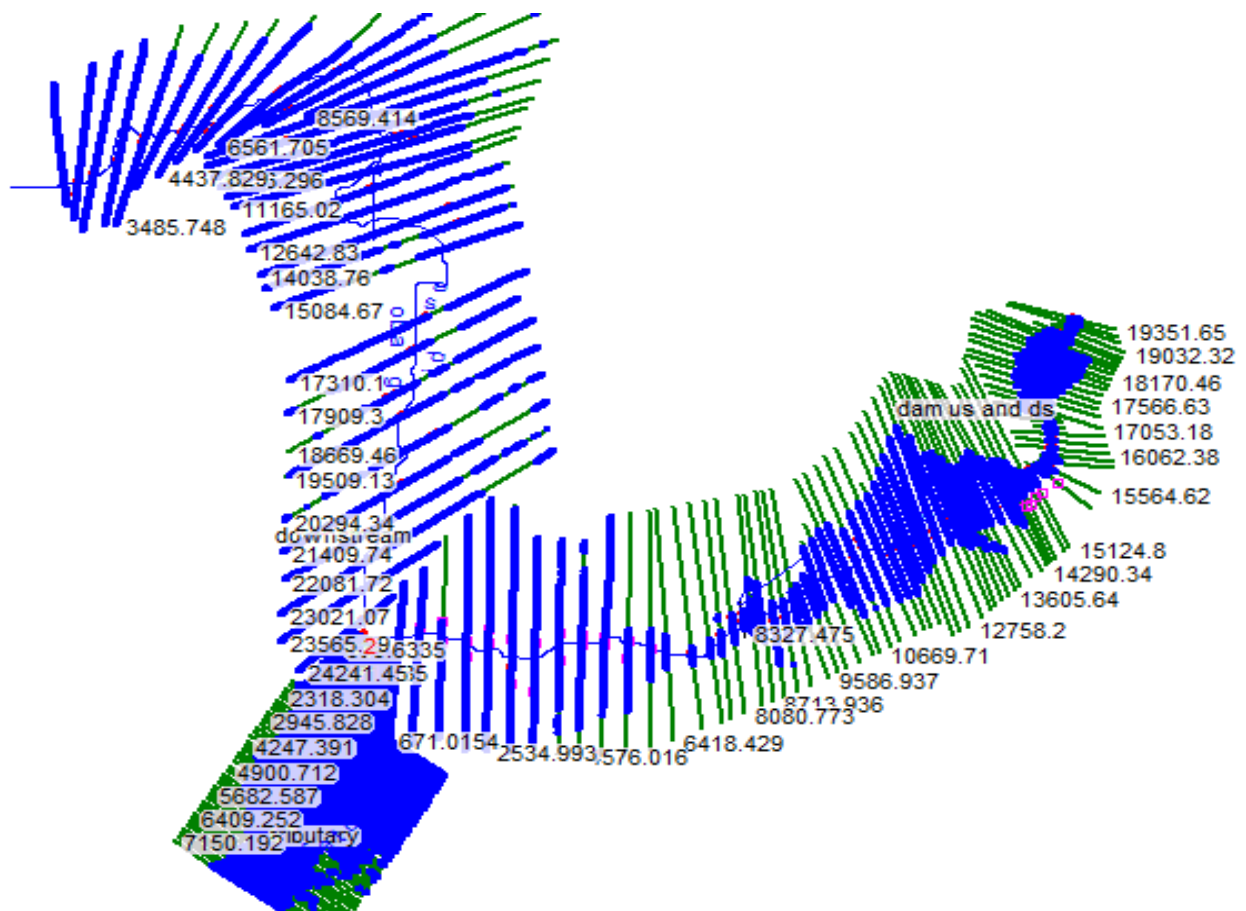


Figure 3-6; River cross-section Geometry in Hec-Ras window

### 3.3.3.6 Routing breach outflow hydrographs through downstream reaches

Dam-break flood hydrograph is a dynamic and unsteady phenomenon. Therefore, the preferred approach is to utilize a fully developed Unsteady State flow routing model. In order to accurately model the flows, the unsteady flow computer program, HEC-RAS was used to route breaching outflow hydrographs through natural waterways.

The implicit formulation of the St. Venant equation is well-suited from the standpoint of accuracy for formulating unsteady flows in a natural channel. Therefore, HEC-RAS is chosen for unsteady state flood routing, and this technique simultaneously computes the discharge, water surface elevation, and velocity throughout the river reach.

For this study two failure scenarios were used, i.e. overtopping and piping modes of failure. The reservoir conditions, normal pool (spillway crest level) and maximum storage elevation (top of the dam crest ) were considered.

### 3.3.4 Setting up model by “Breach”

The model BREACH simulates both overtopping and piping mode of failures separately. These modes of failures are user defined on the model depending on the prevailing conditions of the dam and interest of the modeler.

#### Data Required

The model requires various data such as geometry (of dam, upstream reservoir and tail water section), engineering properties of the embankment material, spillway rating table, inflow hydrograph and initial breach parameter estimates.

#### i. Geometry of Upstream and Downstream Topography

Topographic data of upstream reservoir area is given as area-elevation-volume curve as shown on Figure 3-3. The area at different elevations required by the model is extracted from the graph. Regarding to downstream river cross section, data is taken from DEM.

#### ii. Engineering Properties of Dam Material

The main engineering properties of Gidabo Embankment Dam are listed on Table 3-3 below. The model uses US units and hence values are given accordingly. Most of these data are taken from design report of the dam and the rest from laboratory results of tests conducted in situ during construction period of the dam. These data are collected from site office of the project.

Description	Inner material	Outer material
D50(mm)	0.01	65
Porosity ratio	0.35	0.4
Unit weight (lb/ft <sup>3</sup> )	95.5	143.25

Internal friction angle, degree	0.02	0.028
Cohesive strength (lb/ft <sup>2</sup> )	418	188

Table 3-3 Engineering properties of the Dam

**iii. Reservoir Inflow**

One point five times of the probable maximum Flood (PMF) is used as inflow while simulating overtopping. The 24 hr. based PMF is used for further analysis.

**iv. Spillway Discharge Data**

The discharge through spillway at different elevations given on Table 4-3 is extracted from spillway rating curve obtained from feasibility study document of the project (WWDSE, 2005) as shown on subsequent Figure 3-7

Spillway Elevation, m	Spillway discharge, m <sup>3</sup> /s	Spillway Elevation, m	Spillway discharge, m <sup>3</sup> /s
1201	0	1205	600
1202	30	1206	980
1203	180	1207	1393
1204	350	1208	1500

Table 3-4 Spillway discharge Vs Elevation

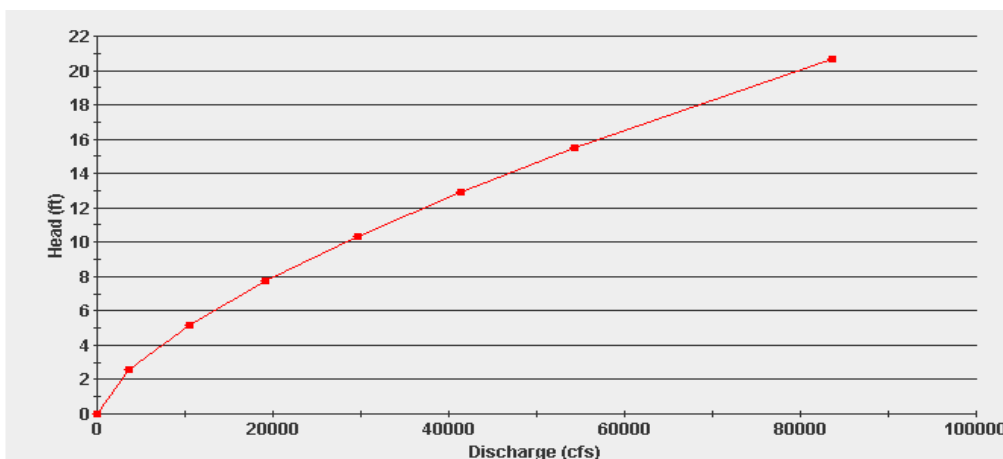


Figure 3-7 Spillway rating curve

**v. Reservoir Characteristics**

Data on reservoir elevation and area are extracted from Capacity-Area-Elevation curve. These curves are shown on Figure 3-3above.

### 3.3.4.1 Overtopping failure

Gidabo Dam has been tested for overtopping failure using the model BREACH and applying a PMF 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 the PMF passes it recedes back to its initial level. From this it can be concluded that the spillway can safely evacuate the incoming PMF for which it has been designed. It was then assumed the dam fails at PMF, for any reason.

### 3.3.4.2 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. The Model BREACH simulates piping failure with elevation of piping line being defined by the user. Other geometry of the dam and reservoir and engineering properties of the embankment material are as defined in the case of overtopping failure discussed earlier.

As explained in chapter one, Gidabo dam has suffered from outlet settlement above expected threshold limit. 1205.55m elevation is assumed susceptible point of start for piping if further settlement is happening in the future. Using this point elevation as a level at which the piping phenomena starts and keeping all other input data as explained in previous section, failure simulation has been conducted using the model. All results and discussions are given under chapter 4.

## 3.4 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
- Dam crest elevation.....1225.8m.a.s.l

- Crest length ..... 335.52m
- Crest Width ..... 8m
- Dam height above the river bed level..... 25.8m
- Dam Bed Width (Maximum).... 210.38m
- Normal Water Level..... 1219.5m.a.s.l
- Upstream face slope.....3H:1V
- Downstream face slope.....3H:1V
- Maximum Water Level ...1223.8m.a.s.l
- Reservoir capacity (NWL) ..... 62.3Mm<sup>3</sup>

The model is able to calculate the parameters as per the most commonly applicable regression (empirical) equations i.e, Mac-Donald et al, Froelich (2008) and von thun and Gillete equations.

The breach bottom width, breach side slopes and formation time are tabulated and discussed under chapter four.

## 4. Result and discussion

### 4.1 General

To investigate a potential risk of Gidabo dam-failure induced flood to the settlements in the downstream reaches, the modeling domain was tested for a set of scenarios under overtopping and piping failures. These scenarios are simulated using breach parameter results from five types of methods namely; MacDonald et al, Frohlich (2008), Von thun and Gillete and Xu and Zhang. This procedure will result in 8 computer runs and corresponding results.

### 4.2 Overtopping mode of failure simulation results

Table 4-1; PMF event breach peak outflows for overtopping mode of failure

Methods	Peak flow (m <sup>3</sup> /s)	Time to peak, hr (Starting at 5/1/2116, 0900)
MacDonald et al	24966.21	5/1/2116, 1450
Frohlich (2008)	18942.30	5/1/2116, 1530
Von thun and Gillete	23844.84	5/1/2116, 1450
Xu and Zhang	16039.97	5/1/2116,1510

In all cases a fictional failure is assumed to occur on 5/1/2116. Overtopping scenario is assumed to occur when the catchment receives a PMF flood event starting at 0900 Gmt. When the catchment above the dam is under this event, it is obvious that the whole catchment above and below the dam will be in the same event. Based on this consideration the tributary river, Fetcha, is expected to be in the PMF event. This PMF in the tributary and the burst flood from the dam has resulted the following tabulated results.

Table 4-2; maximum flow, time to peak, rate of flow and flood height for some Gidabo river stations below Gidabo dam during overtopping (MacDonald et al), starting time at 5/1/2116, 0900

Stations	Peak flow (m <sup>3</sup> /s)	velocity (m/s)	flood height (m)	time to peak Hr
At Dam	24966.21	1.43	40.02	5/1/2116, 1450
0.21179km	24966.21	4.416	33.28	5/1/2116, 1450
1.11479km	24926.26	2.90	15.11	5/1/2116, 1500
3.1794km	24547.76	1.46	12.28	5/1/2116, 1510
6.22212km	20345.68	13.65	25.53	5/1/2116, 1530
7.45173km	14172.87	1.44	12.94	5/1/2116, 1540
14.42583km	9705.42	0.23	29.56	5/1/2116, 1800
23.93722km	4707.23	0.20	19.45	5/2/2116, 0900

NB: All distances are measured from dam location

As shown in the table the peak flow is decreasing starting from the dam downwards up to the lake entry. The flood height shows an increased value at the dam and it continues to decrease for some distance and it highly increases suddenly at 6km from the dam. This is because a tributary is joining the coming flood at this point. The time to peak when analyzed by this method is found to be 14:50hr from the initial flood beginning time 09:00. According to this analysis the whole process i.e the PMF event beginning upstream of the dam, the dam breach and flood attenuation in the downstream regions has taken 24 hours.

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

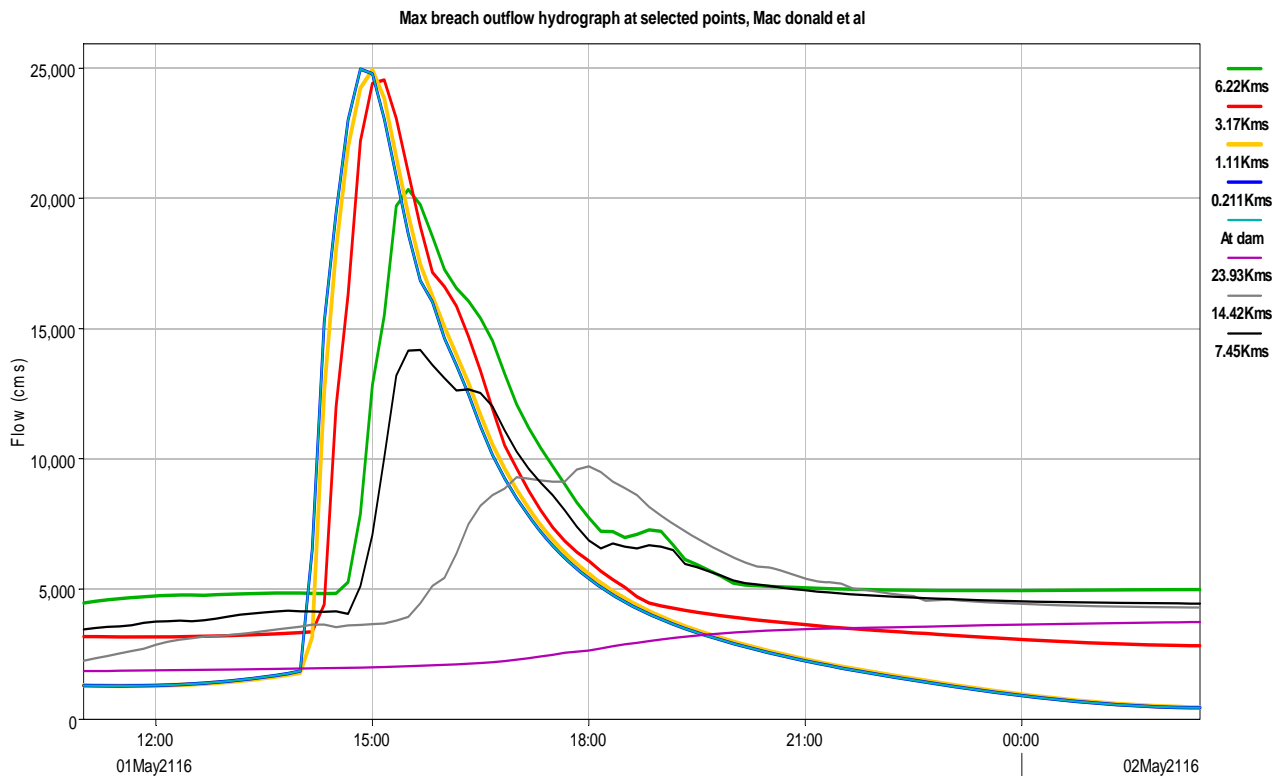


Table 4-3; maximum flow, time to peak, rate of flow and flood height for some Gidabo river stations below Gidabo dam during overtopping ( Frohelich 2008 ), starting time at 5/1/2116, 0900

Stations	Peak flow (m <sup>3</sup> /s)	velocity (m/s)	flood height (m)	time to peak hr
At Dam	18942.30	0.458	40	5/1/2116, 1530
0.21179km	18942.30	3.655	32.87	5/1/2116, 1530
1.11479km	18911.11	7.182	18.04	5/1/2116, 1530
3.1794km	18873.85	1.346	18.09	5/1/2116, 1540
6.22212km	16772.54	1.506	11.82	5/1/2116, 1610
7.45173km	12023.70	1.66	12.43	5/1/2116, 1640
14.42583km	8739.21	0.44	27.74	5/1/2116, 1820
23.93722km	3109.59	0.31	13.62	5/2/2116, 0000

NB: All distances are measured from dam location

As shown in the table the peak flow is decreasing starting from the dam downwards upto the lake entry. A significantly high velocity of 7.182m/s is observed at 1.11Km from the dam. The flood height shows an increased value at the dam and it continues to decrease for some distance and it highly increases suddenly at 7.45km from the dam. This is because a tributary is joining the coming flood at this point. The time to peak when analyzed by this method is found to be 15:30hr from the initial flood beginning time 09:00. According to this analysis the whole process i.e the PMF event beginning upstream of the dam, the dam breach and flood attenuation in the downstream regions has taken 15 hours.

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

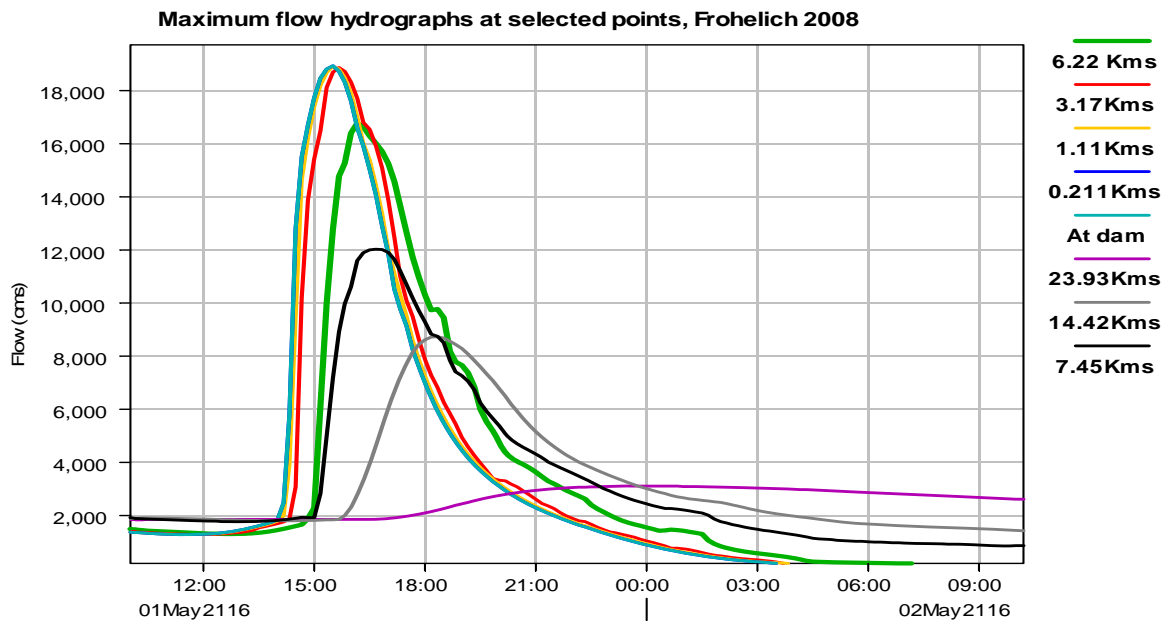


Table 4-4; maximum flow, time to peak, rate of flow and flood height for some Gidabo river stations below Gidabo dam during overtopping (Von thun and Gillete), starting time at 5/1/2116, 0900

Stations	Peak flow (m <sup>3</sup> /s)	velocity (m/s)	flood height (m)	time to peak hr
At Dam	25426.42	1.43	40.02	5/1/2116, 1440
0.21179km	25426.42	3.06	33.49	5/1/2116, 1440
1.11479km	25582.65	14.11	25.61	5/1/2116, 1450
3.1794km	24829.07	2.21	17.66	5/1/2116, 1500
6.22212km	19868.88	1.53	11.72	5/1/2116, 1520
7.45173km	12804.64	1.56	12.32	5/1/2116, 1530
14.42583km	8558.52	0.44	27.72	5/1/2116, 1750
23.93722km	3103.98	0.17	17.62	5/1/2116, 2350

NB: All distances are measured from dam location

Among the non-physically based overtopping results von thun has resulted the maximum peak flow of 25426.42m<sup>3</sup>/s and flood height of 25.61m as shown in the table above. This method's results ( i.e peak flow, velocity and flood height ) have same rising and falling patterns as other methods but with different magnitudes.

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

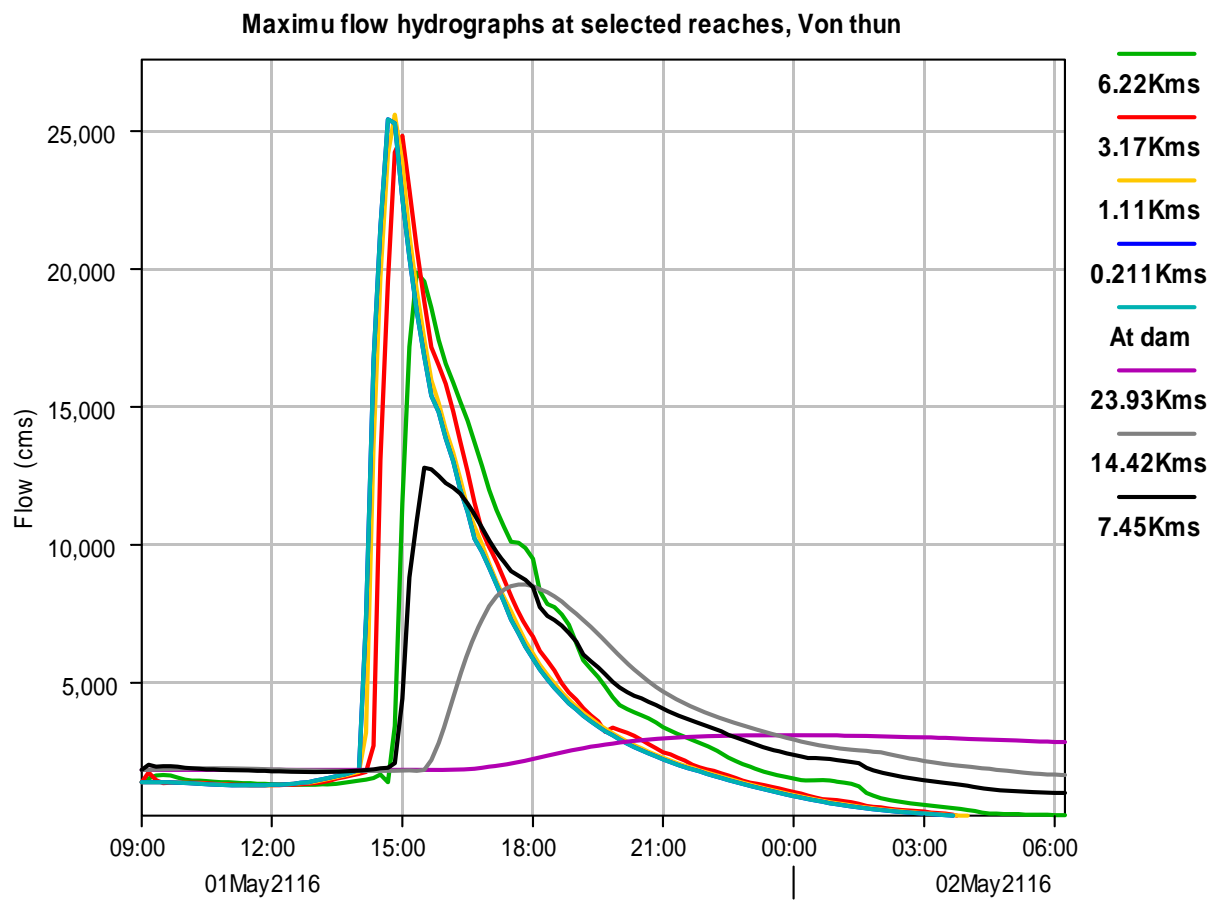


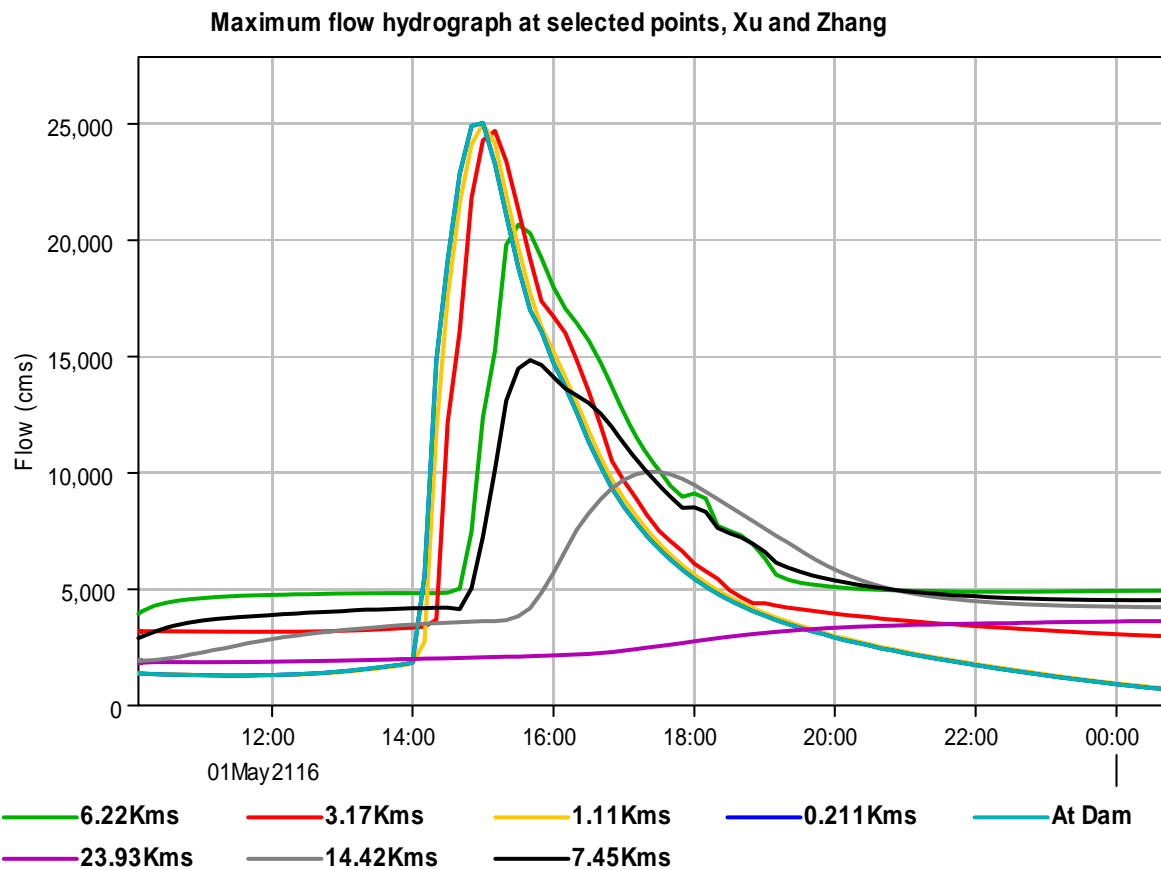
Table 4-5; maximum flow, time to peak, rate of flow and flood height for some Gidabo river stations below Gidabo dam during overtopping (Xu and Zhang), starting time at 5/1/2116, 0900

Stations	Peak flow (m3/s)	velocity (m/s)	flood height (m)	time to peak hr
At Dam	25057.67	1.48	40.24	5/1/2116, 1500
0.21179km	25057.67	2.78	31.83	5/1/2116, 1500
1.11479km	25012.64	10.69	24.37	5/1/2116, 1500
3.1794km	24701.63	2.38	14.32	5/1/2116, 1510
6.22212km	20665.23	1.38	11.51	5/1/2116, 1530
7.45173km	14847.13	1.48	12.1	5/1/2116, 1540
14.42583km	10040.70	0.44	27.71	5/1/2116, 1720
23.93722km	4731.71	0.17	17.612	5/15/2116, 0900

NB: All distances are measured from dam location

According to this analysis the whole process i.e the PMF event beginning upstream of the dam, the dam breach and flood attenuation in the downstream regions has taken 24 hours. This duration of time result is similar to the Mac donald et al's method result.

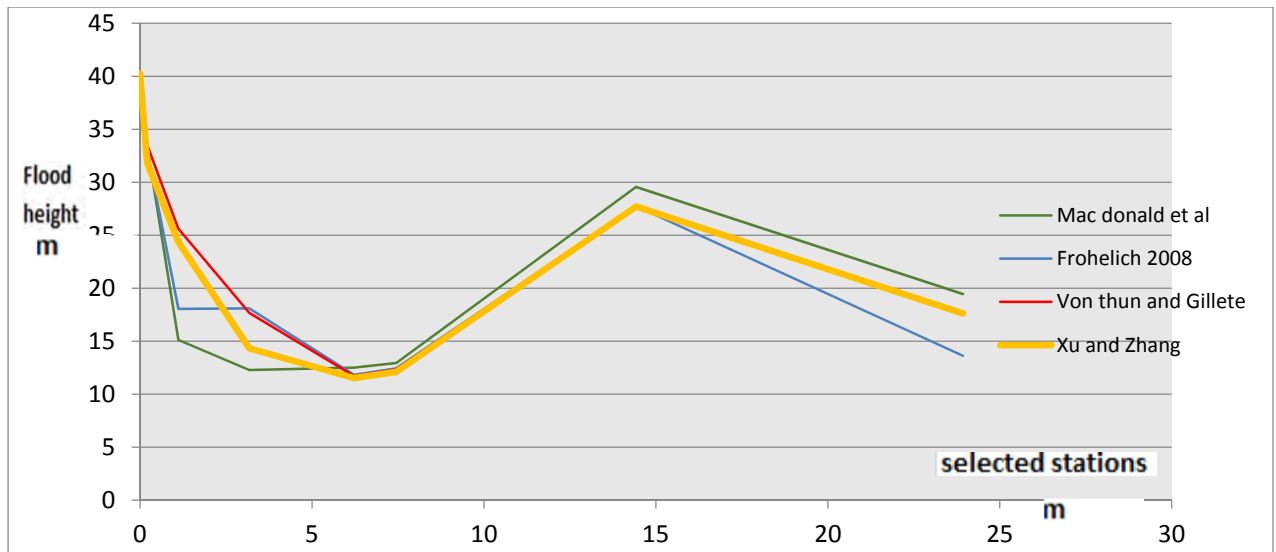
Figure 4-4; Maximum outflow hydrograph at selected points, overtopping by Xu and Zhang



As shown in Figure 4-5, The flood heights at selected stations are presented on one graph when analyzed by different methods. the selected stations are at the dam, 0.211km, 1.11km, 3.17km, 6.22km, 7.45km, 14.42km and 23.93km from the dam. It is clear that all the graphs follow same trend i.e the lines rise and fall in a some how same pattern.

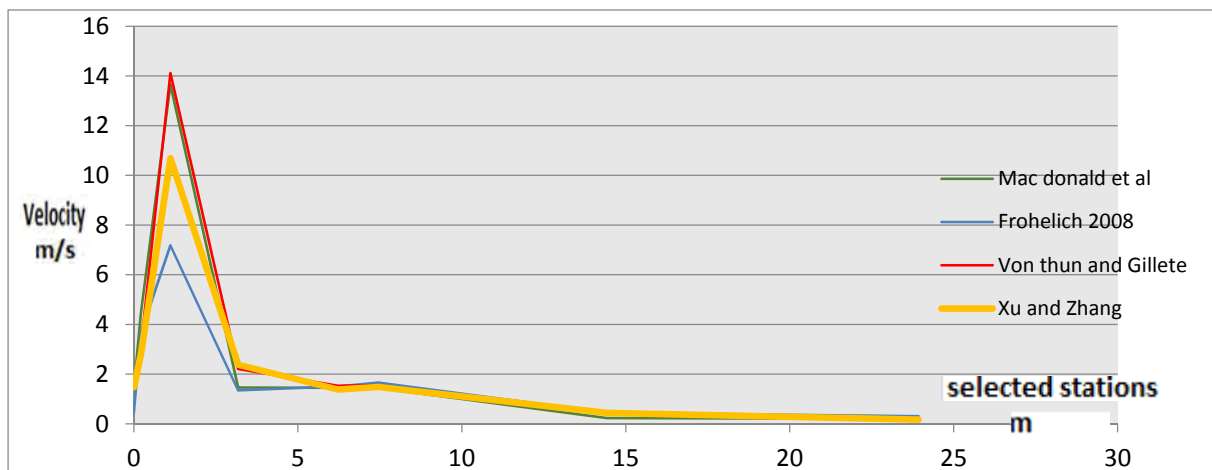
From this pattern high flood levels always ( i.e by any method ) occur at 14.42km from the dam. Low flood depths occur at 6.22Km from the dam. The rise of flood height beyond the 6.22km, station is explained by the fact that a tributary is joining the bursting flood. Downstream of this peak depth at 14.42km, the flood propagates to join Abaya lake faced by a relatively flatter land where depth decreases rapidly and the accumulated water scattered on the flat land.

Figure 4-5; Selected stations versus flood height for each of the methods



The velocity pattern is as shown in figure 4-6. In all cases the velocity suddenly rises to reach its peak at 1.114km from the dam and a rapid decrease immediately beyond this point. This is explained by the fact that the water behind the dam prior to failure is under high hydrostatic pressure, so when it gets a relief hole through the dam it dissipates this energy through a pronounced flood velocity. In all cases the velocity slowly decreases beyond the 3.179km station point until attenuation.

Figure 4-6; Selected stations versus flood velocities for each of the methods



### 4.3 Piping mode of failure simulation results

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

Methods	Peak flow (m <sup>3</sup> /s)	Time to peak, hr
MacDonald et al	15882.16	5/1/2116, 1130
Frohlich (2008)	14785.74	5/1/2116, 1150
Von thun and Gillete	17348.94	5/1/2116, 1040
Xu and Zhang	9843.82	5/1/2116, 1330

Table 4-7; maximum flow, time to peak, rate of flow and flood height for some Gidabo river stations below Gidabo dam; breach by piping (MacDonald et al), starting time at 5/1/2116, 09:00

Stations	Peak flow (m <sup>3</sup> /s)	velocity (m/s)	flood height (m)	time to peak hr
At Dam	17458.61	1.26	39.02	5/1/2116, 1130
0.21179km	17458.61	3.98	30.03	5/1/2116, 1130
1.11479km	17469.86	11.16	24.57	5/1/2116, 1130
3.1794km	17123.69	2.44	14.44	5/1/2116, 1140
6.22212km	14645.83	1.45	11.27	5/1/2116, 1200
7.45173km	10148.74	1.45	11.89	5/1/2116, 1220
14.42583km	6788.63	0.42	27.37	5/1/2116, 1400
23.93722km	2826.21	0.16	16.28	5/1/2116, 2210

NB: All distances are measured from dam location

Figure 4-7; Maximum outflow hydrograph at selected points, piping by Macdonald et al

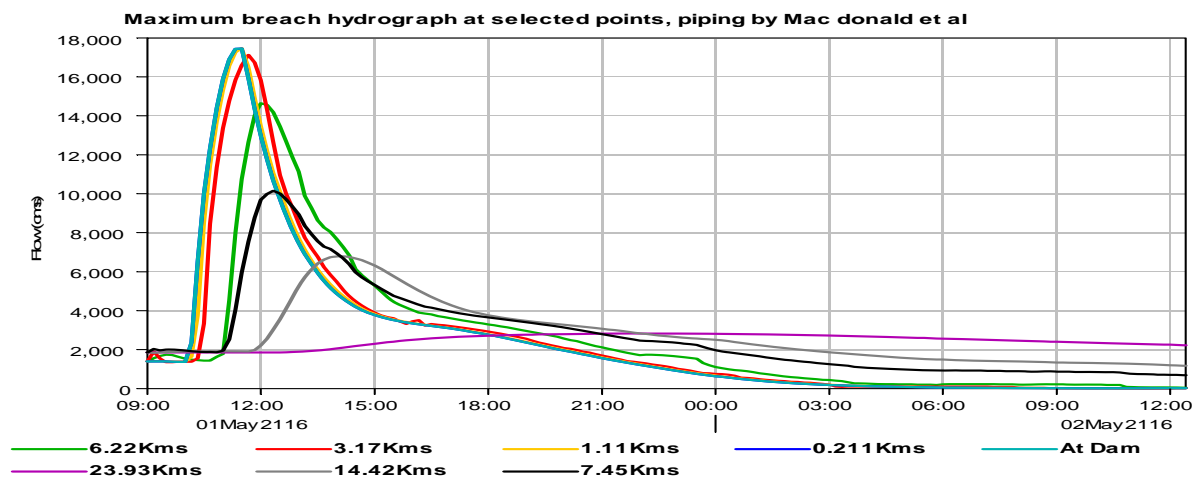


Table 4-8; maximum flow, time to peak, rate of flow and flood height for some Gidabo river stations below Gidabo dam; breach by piping (Frohelich 2008), starting time at 5/1/2116, 0900

Stations	Peak flow (m <sup>3</sup> /s)	velocity (m/s)	flood height (m)	time to peak hr
At Dam	14946.64	1.21	39.02	5/1/2116, 1150
0.21179km	14946.64	3.8	31.18	5/1/2116, 1150
1.11479km	14893.62	10.27	24.14	5/1/2116, 1150
3.1794km	14795.80	1.56	16.16	5/1/2116, 1210
6.22212km	13290.04	1.4	10.22	5/1/2116, 1230
7.45173km	9425.48	1.42	11.77	5/1/2116, 1240
14.42583km	6517.28	0.43	27.38	5/1/2116, 1430
23.93722km	2828.10	0.16	16.27	5/2/2116, 2230

NB: All distances are measured from dam location

Figure 4-8; Maximum outflow hydrograph at selected points, piping by Frohelich 2008

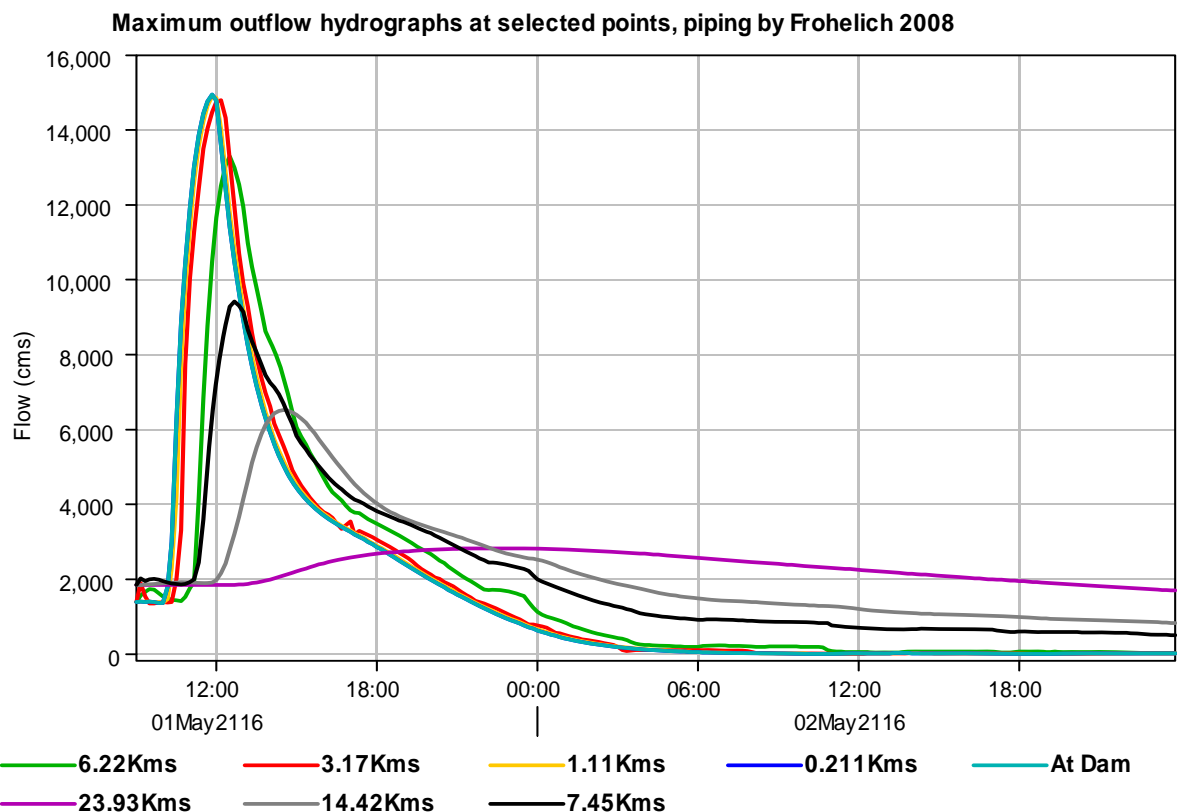
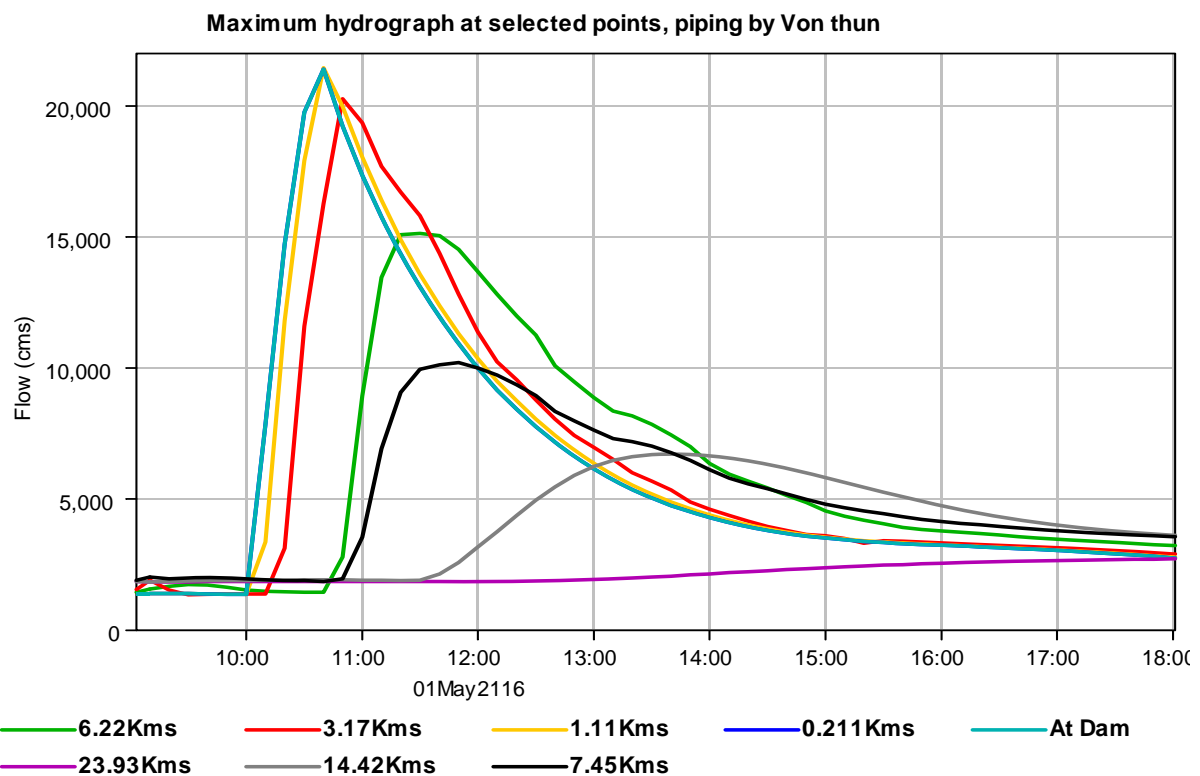


Table 4-9; maximum flow, time to peak, rate of flow and flood height for some Gidabo river stations below Gidabo dam; breach by piping (Von thun and Gillete), starting time at 5/1/2116, 0900

Stations	Peak flow (m3/s)	velocity (m/s)	flood height (m)	time to peak hr
At Dam	21401.90	1.33	36.93	5/1/2116, 1040
0.21179km	21401.90	4.24	31.63	5/1/2116, 1040
1.11479km	21447.52	12.46	25.15	5/1/2116, 1040
3.1794km	20262.68	2.61	18.24	5/1/2116, 1050
6.22212km	15128.43	1.45	11.25	5/1/2116, 1130
7.45173km	10196.27	1.458	11.84	5/1/2116, 1150
14.42583km	6718.14	0.42	27.37	5/1/2116, 1340
23.93722km	2820.07	0.16	17.26	5/1/2116, 2210

NB: All distances are measured from dam location

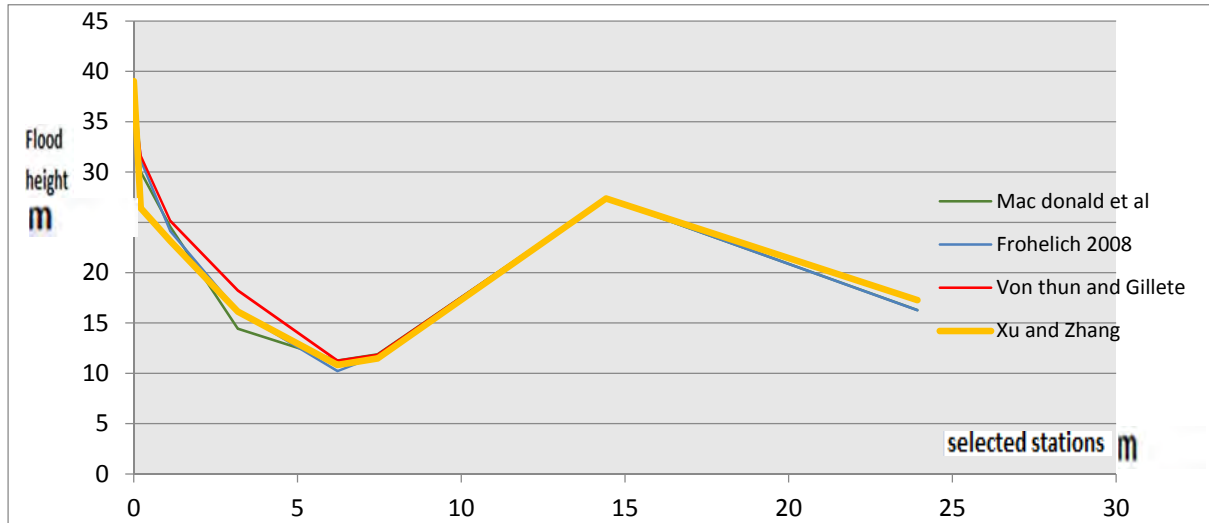
Figure 4-9; Maximum outflow hydrograph at selected points, piping by Von thun and Gillete2008





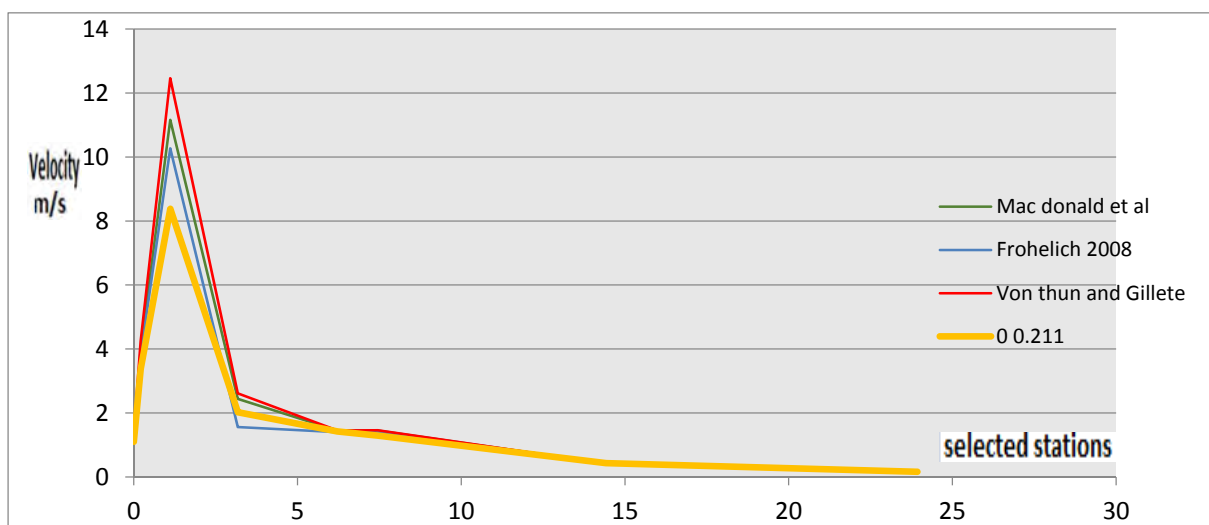
always ( i.e by any method ) occur at 14.42km from the dam. Low flood depths occur at 6.22Km from the dam.

Figure 4-11; Maximum outflow hydrograph at selected points, piping by Xu and Zhang



The velocity pattern is as shown in figure 4-6. In all cases the velocity suddenly rises to reach its peak at 1.114km from the dam and a rapid decrease immediately beyond this point. This is explained by the fact that the water behind the dam prior to failure is under high hydrostatic pressure, so when it gets a relief hole through the dam it dissipates this energy through a pronounced flood velocity. In all cases the velocity slowly decreases beyond the 3.179km station point until attenuation.

Figure 4-12; Maximum outflow hydrograph at selected points, piping by Xu and Zhang



#### 4.4 Breach Modeling

Both overtopping and piping failure modes were checked. When overtopping failure mode is simulated, the PMF could be safely evacuated through the spillway and no breaching was observed. Hence, the dam is safe against failure induced by overtopping of flood ranging up to 1.5 of PMF.

The following major outputs are obtained for overtopping mode of failure.

Table 4-11; Major outputs of the model for overtopping mode of failure

Description	Unit	Quantity
Top width	m	251.24
Bottom width	m	179.99
Depth	m	78.09
Bottom elev.	m	1201.99
Side slope	m/m	1.5
Peak outflow	m <sup>3</sup> /s	49119.64

The dam could potentially fail due to piping flow emerged at weak point formed by further settlement of the outlet. The piping elevation at which the dam fails giving the most catastrophic flood is 1205.55m a.m.s.l. An estimate was made based on unexpected settlement observed during construction and the predicted further settlement after construction

Table 4-12; Major output of the model for piping mode of failure

Description	Unit	Quantity
Top width	m	152.36
Bottom width	m	120.32
Depth	m	78.09
Bottom elev.	m	1202.2
Side slope	m/m	1
Peak outflow	M <sup>3</sup> /s	34068

### 4.5 Breach outflow verification

#### 4.5.1 Overtopping outflow verification

peak overtopping outflows obtained are compared to the envelope curve as a taste of reasonableness as shown below in figure 4-11. A closer look at the graph reveals that Von thun and Gillette's method has resulted in a more realistic result when compared to the envelope curve with an error of 5.25%. Frohlich's, Macdonald's and Xu and Zhang's method are found to have 16.38%, 10.20% and 29.19% error respectively in relative to the envelope result.

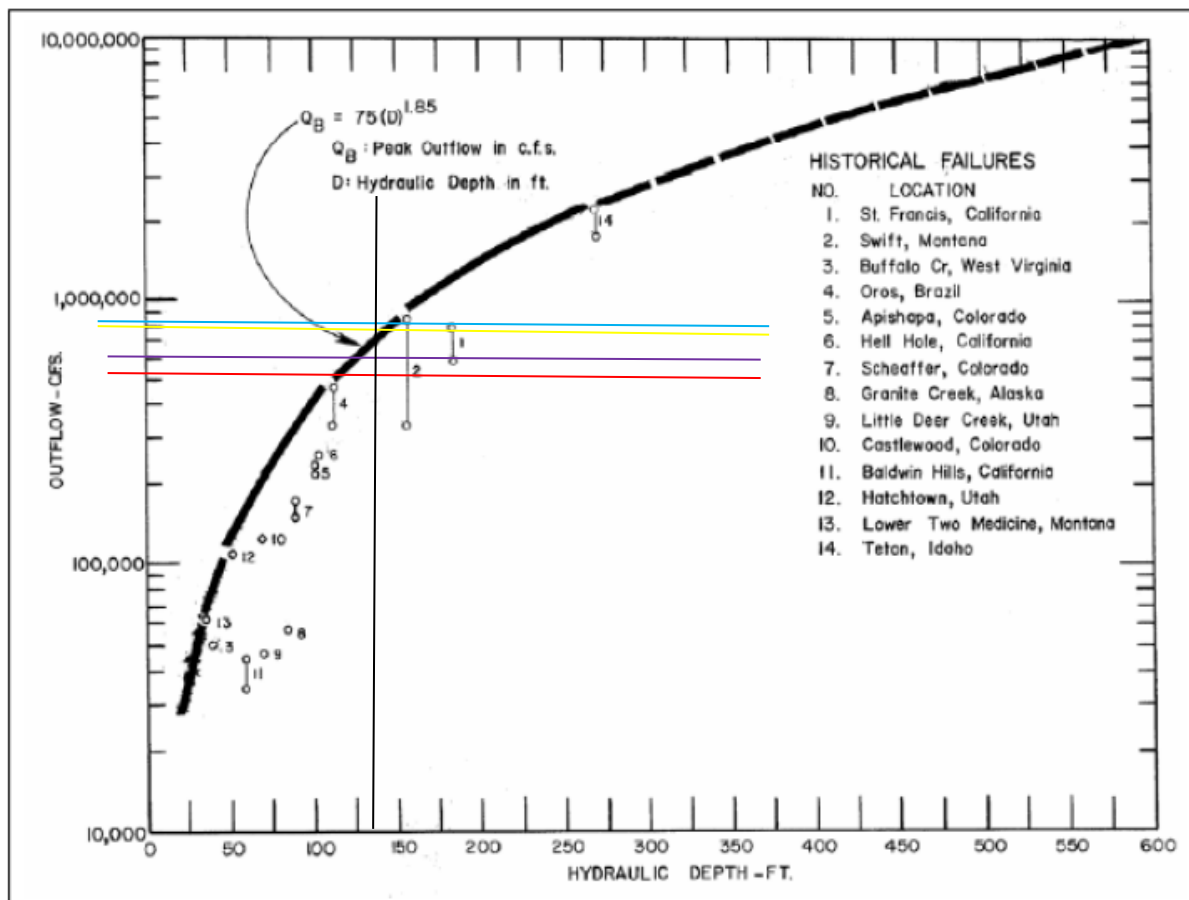


Figure 4-13; Verification of overtopping outflows using experienced outflow rates envelope

\_\_\_\_\_ Xu and Zhang method 16039.97m<sup>3</sup>/s or 566446.19ft<sup>3</sup>/s

\_\_\_\_\_ Frohlich (2008) method 18942.30m<sup>3</sup>/s or 668941.011ft<sup>3</sup>/s

\_\_\_\_\_ Macdonald et al method 24966.21m<sup>3</sup>/s or 881673.38ft<sup>3</sup>/s

\_\_\_\_\_ Von thun and Gillete method 23844.84m<sup>3</sup>/s or 842072.57ft<sup>3</sup>/s

\_\_\_\_\_ Hydraulic depth 130ft

### 4.5.2 Piping outflow verification

Peak piping outflows obtained are compared to the envelope curve as a taste of reasonableness as shown below in figure 4-12. Comparison reveals that Xu and Zhang's method has resulted in a more realistic result when compared to the envelope curve with an error of 13.09%. Frohlich's, Macdonald's and Von thun method are found to have 30.53%, 40.21% and 53.16% error respectively in relative to the envelope result.

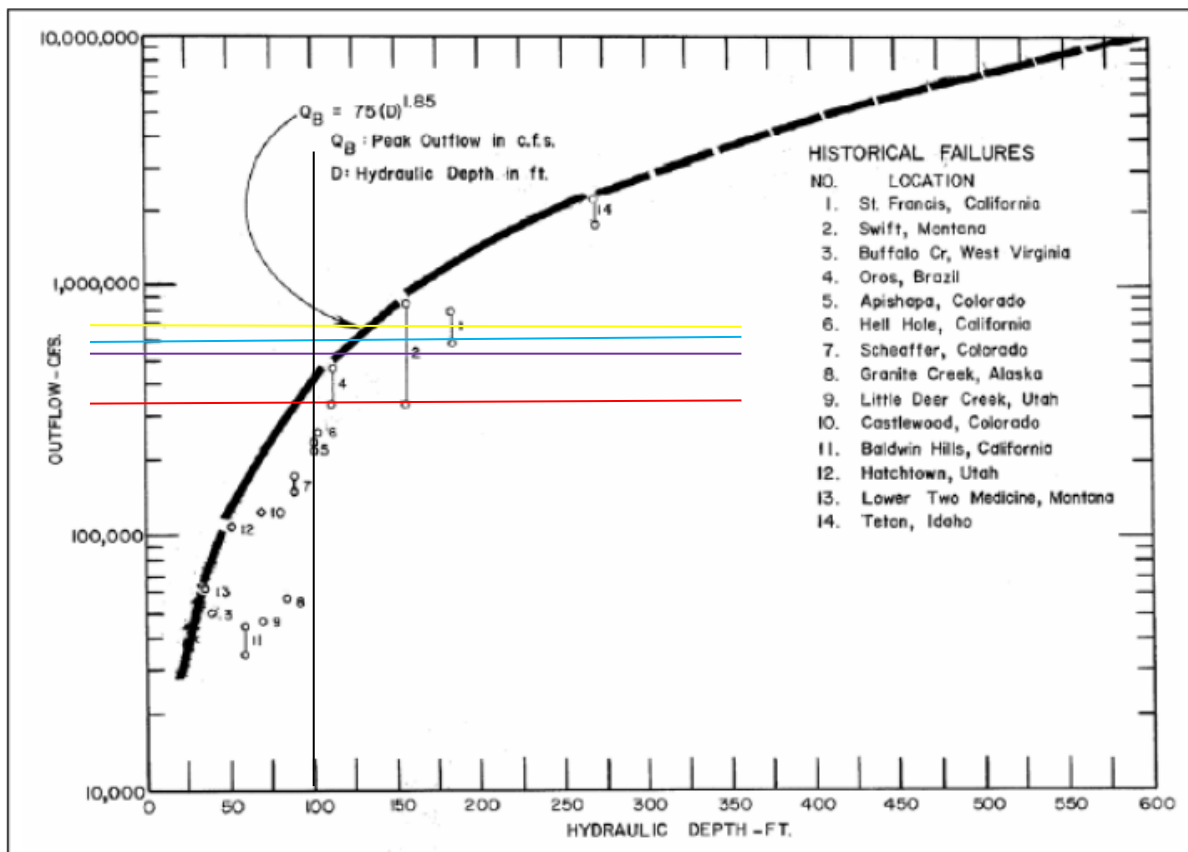


Figure 4-14; Verification of piping outflows using experienced outflow rates envelope

- \_\_\_\_\_ Xu and Zhang method 9843.82m<sup>3</sup>/s or 347631.22ft<sup>3</sup>/s
- \_\_\_\_\_ Frohlich (2008) method 14785.74m<sup>3</sup>/s or 522153.48ft<sup>3</sup>/s
- \_\_\_\_\_ Macdonald et al method 15882.16m<sup>3</sup>/s or 560873.18ft<sup>3</sup>/s
- \_\_\_\_\_ Vonthun and Gillete method 17348.94m<sup>3</sup>/s or 612672.03ft<sup>3</sup>/s
- \_\_\_\_\_ Hydraulic depth 98.25ft

#### 4.6 Comparison among results

Physically based models are relations developed from historical dam failures that correlate breach size and development time with dam and reservoir characteristics as explained in literature review. The empirical approaches rely on statistical analysis of data obtained from documented failures. 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 Parameters have significant meaning while preparing EAP for a dam breach. Reduction of Economy and optimal safety achievement should be taken into consideration while selecting appropriate method. Here below a range of values is tabulated for further use in emergency action plan preparation.

Table 4-13; summary of breach parameters

Method	Peak flow (m <sup>3</sup> /s)	Breach bottom width (m)	Side slopes (H:V)	Breach development time (hrs)
overtopping				
MacDonald et al	24966.21	235	0.5	2.18
Xu and Zhang	25057.67	111	1.45	4.35
Froehlich (2008)	18942.30	139	1	2.66
Von thun and Gillete	25426.42	110	0.5	0.80
Breach model	49119.64	179.99	1.5	5.6
Piping				
MacDonald et al	17458.61	235	0.5	2.18
Xu and Zhang	10006.63	69	0.86	4.62
Froehlich (2008)	14946.64	109	0.7	2.66
Von thun and Gillete	21401.90	110	0.5	0.8
Breach model	34068	152.36	1	5

Simulating the breach by the selected four methods have resulted in the following breach outflow hydrographs as shown in the figure below.

Figure 4-15; Breach outflow comparison during overtopping by different methods

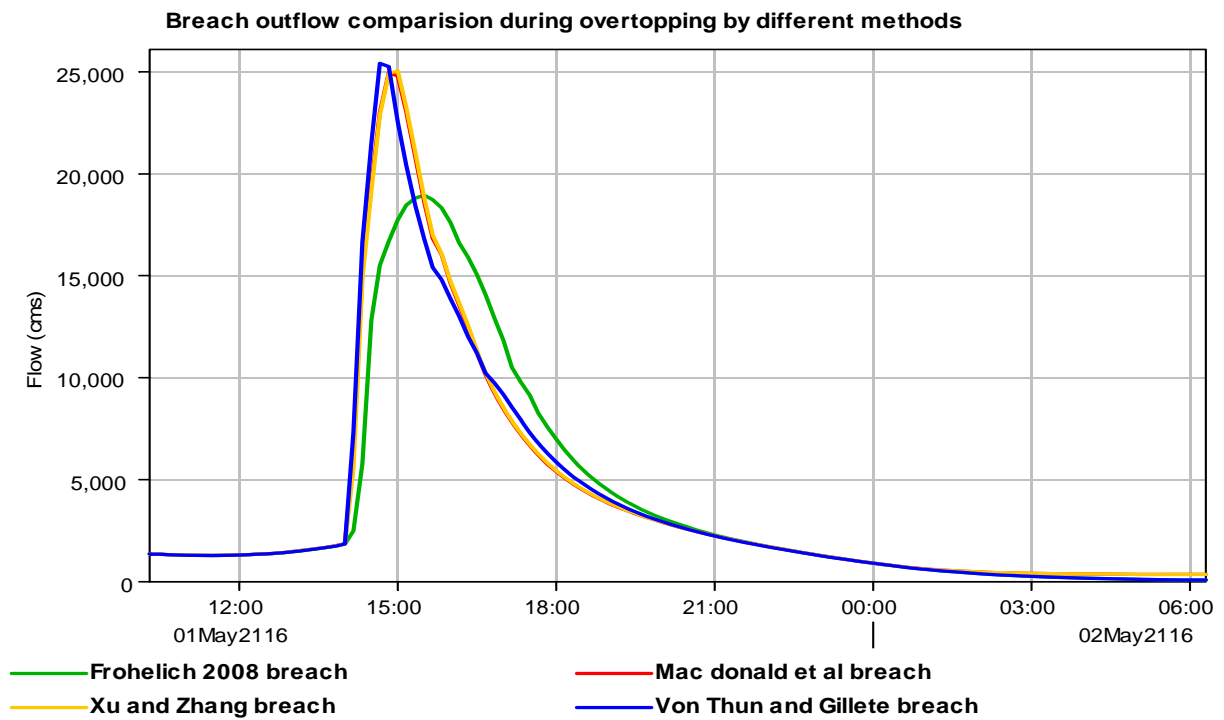
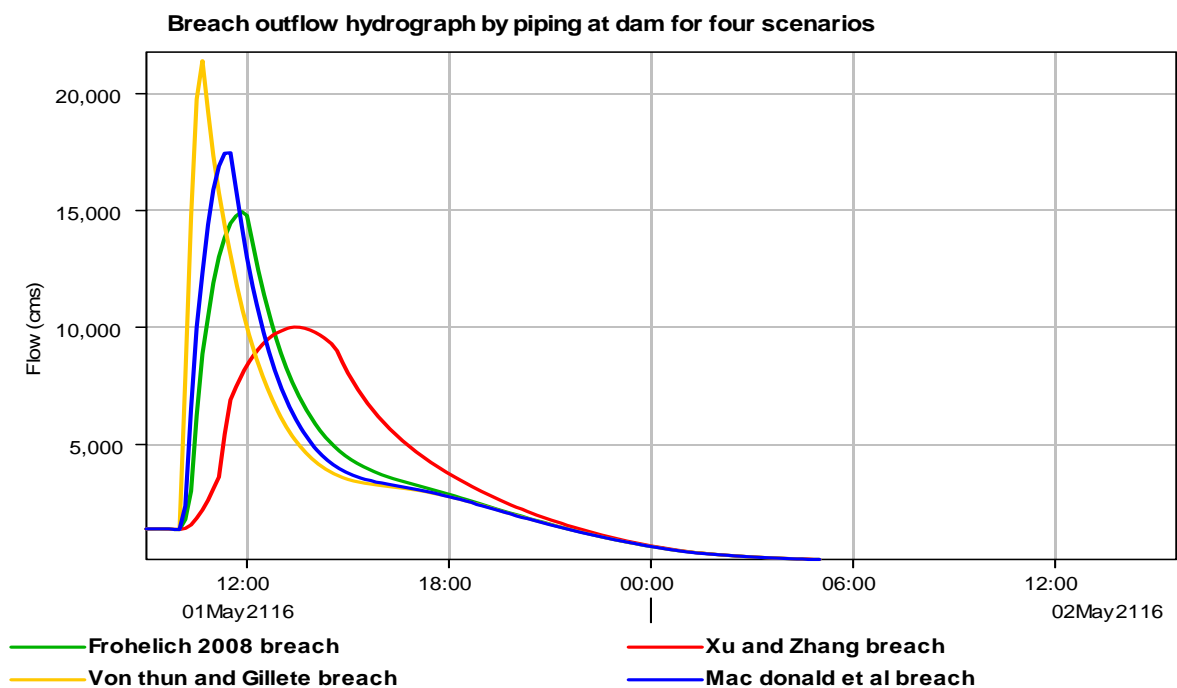


Figure 4-16; Breach outflow comparison during piping by different methods



## 4.7 Flood inundation mapping

An extended area downstream of the dam would be affected as a result of Gidabo dam failure. The flood inundation map for the entire study area for Gidabo dam failure is presented for overtopping type failure and for piping type failure. Flood inundation area in both failure modes is significantly 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 and Von thun and Gillette method of parameter estimation has resulted in the highest discharge values in both overtopping and piping modes of failure. The breach model results are very much exaggerated when compared with the other ones. Piping mode of failure has obviously resulted in lesser magnitudes than overtopping does.

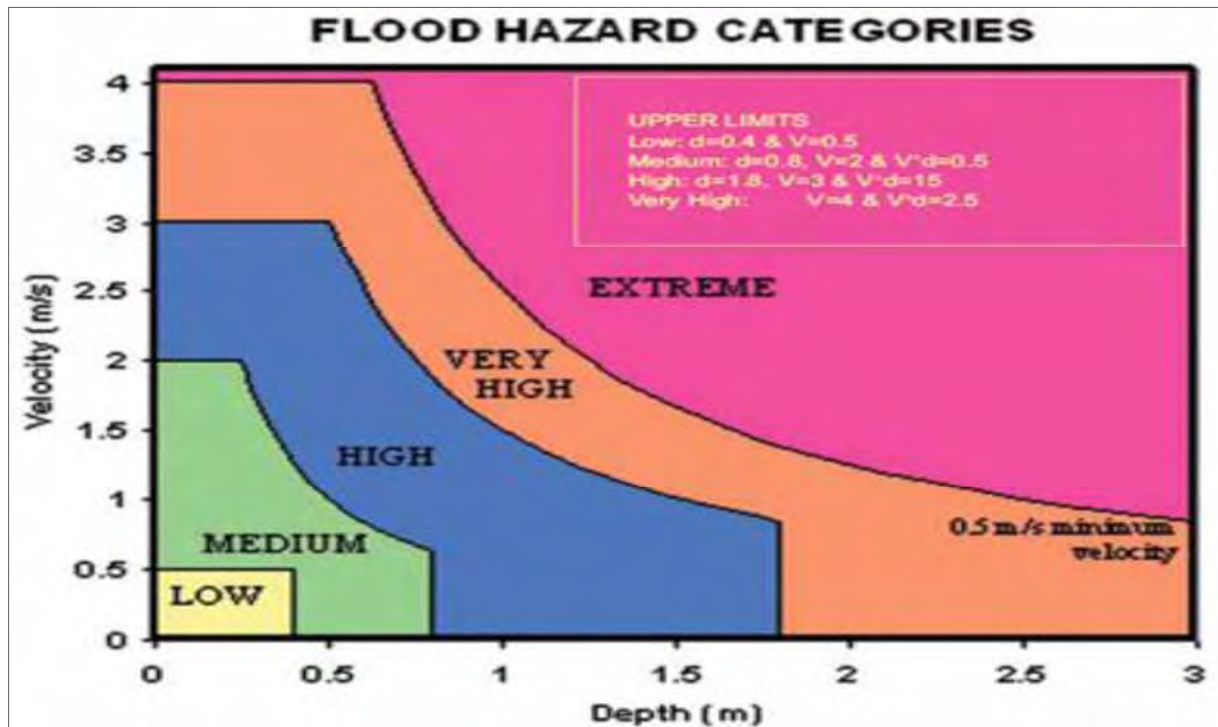
Selection of appropriate and reasonable method for emergency action plan preparation depends on the decision of the EAP preparing body but the results have a direct influence on the economy of flood hazard minimization plan.

Important areas of flood depth and velocity for case of overtopping failure mode is shown in the figure below for Von thun and Gillete results. The flood Hazard map according to FEMA (2014) classification is also prepared for overtopping failure mode as shown in Figure 4-18.

### **Flood hazard map**

Once the model is run, 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 arrival time and duration. 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-GeoRAS 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.

Figure 4-17; Flood hazard categories



Different studies and guidelines have different flood hazard classification limits and justifications. The FEMA (2014) guide line is used for this study. Flood severity grid prepared by FEMA is shown in the figure below in order to obtain upper limits of the depth \* velocity product for each category of hazard level.

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.”

Table 4-14; Simplified Flood Depth and Velocity Severity Grid Symbolization Categories  
(Source FEMA (2014))

<b>Flood Severity Category</b>	<b>Depth * Velocity Range (m<sup>2</sup>/sec)</b>
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

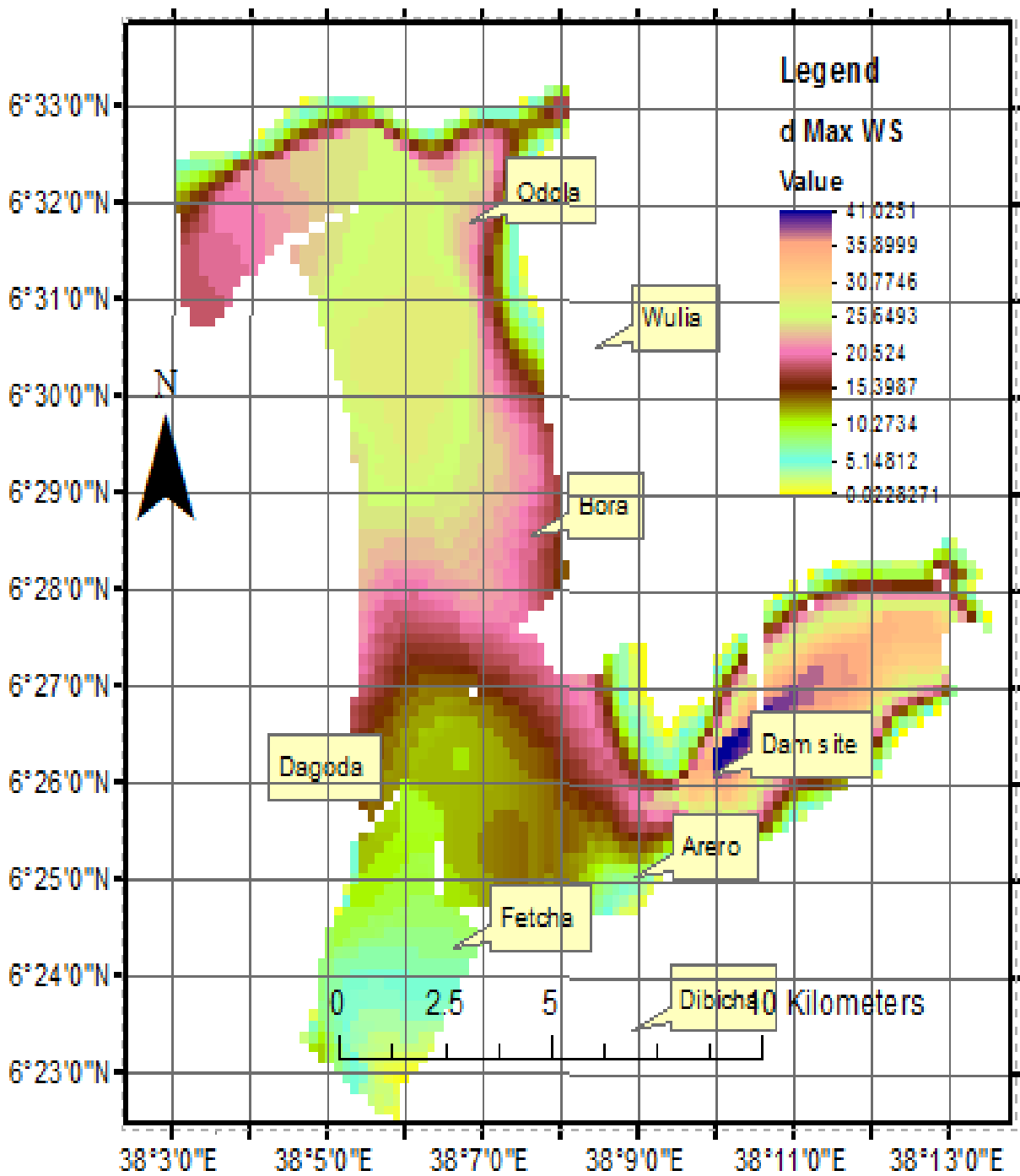


Figure 4-18; Overtopping, breach by Von thun and Gillette depth map

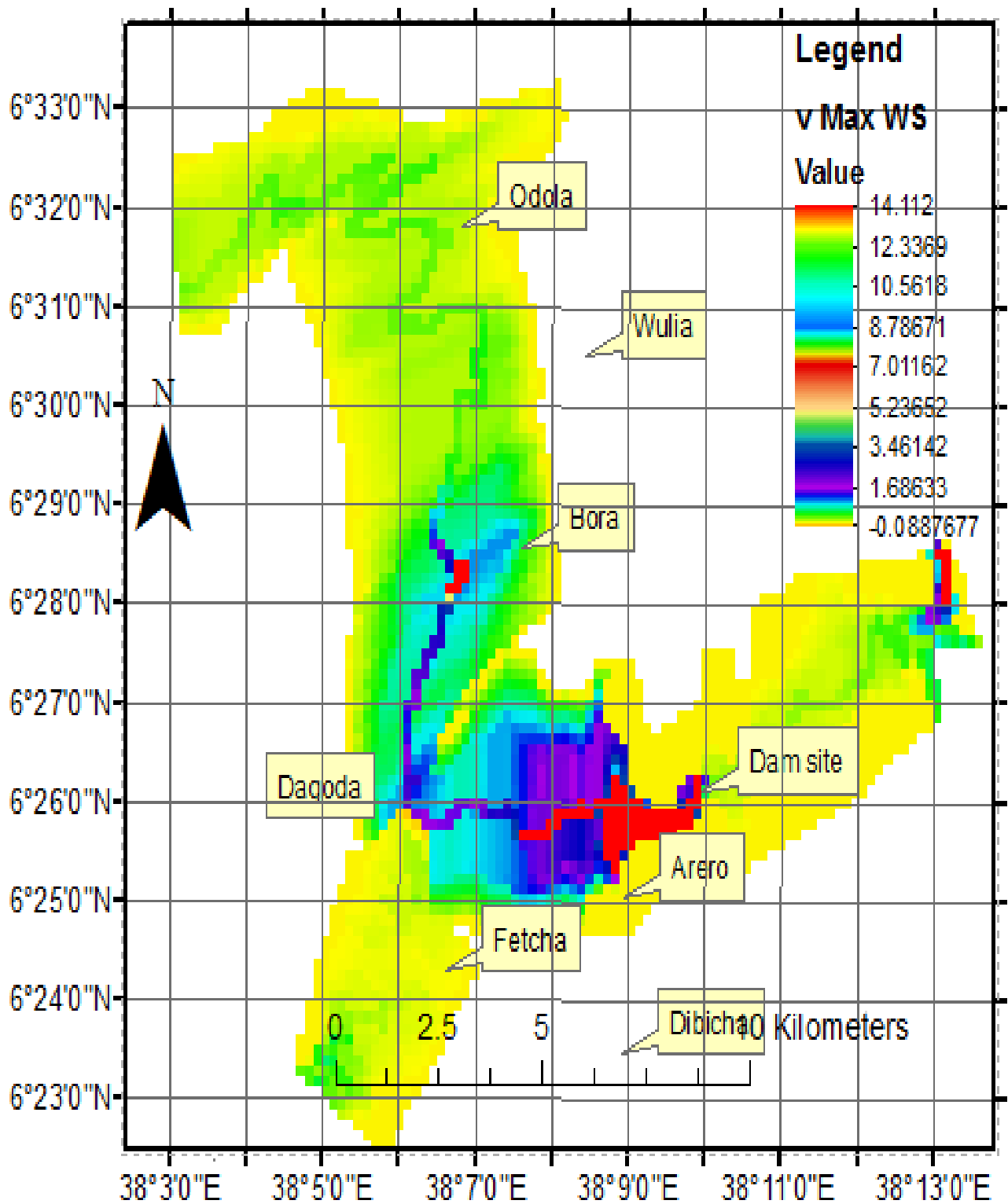
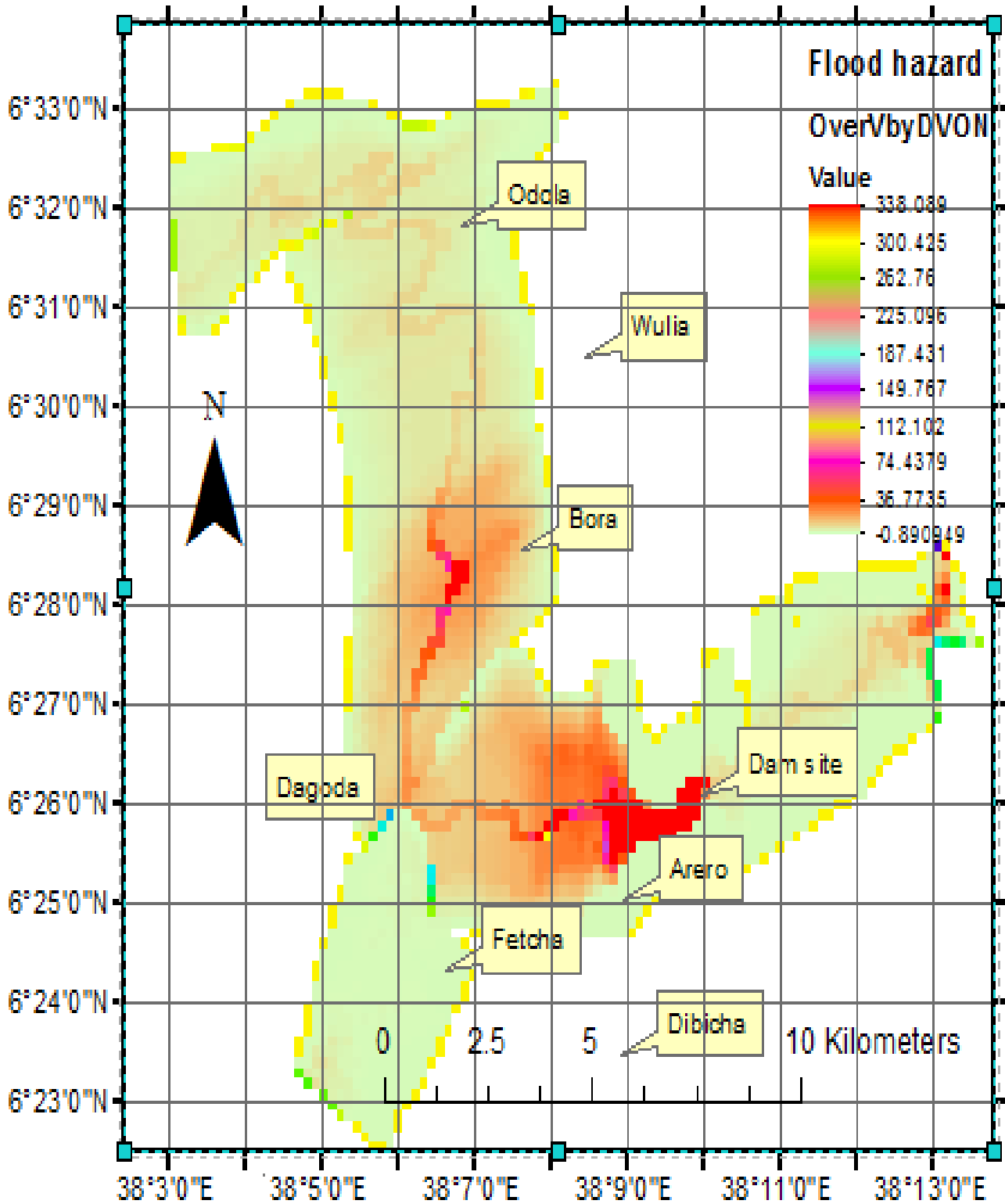


Figure 4-19; Overtopping, Von thun and Gillette velocity map

Figure 4-20; Overtopping, Von thun and Gillette flood hazard map



## 5. Conclusion and Recommendation

### 5.1 Conclusion

Flood management planning and implementation should consider information regarding consequential damages if a dam fails. This should involve accurate estimation of the severity and extent of dam break flood prior to the construction of a dam. Therefore, damages that could occur in the surrounding settlements, agricultural areas, and industries on both lives and infrastructure can be minimized and even controlled.

The severity level of a dam breach depends on the size of reservoir, the incoming flood, the method implied to forecast breach and the settlement magnitude and types on the downstream reaches. This thesis has tried to see the effect of model selection on the severity level of breach.

Accordingly, ten computer runs were made to see the hazardness of breach when treated with different methods. The methods implied basically are for overtopping and piping modes of failure. Five of the runs are for overtopping and the rest five for piping mode of failure. Two of the runs are generally categorized under “physical based models” and follows principles of sediment transport, observed flow regime, erosion and instability concepts to breach the model. This model is executed by a software called “breach”.

The rest eight runs are grouped under “non physically based empirical models”. This methods use statistics of historic dam failures and generate results by approximating the dam to similar other failed dams based on given data. Four methods proposed by Frohlich 2008, Mac Donald et al, Von thun and Gillette and Xu and Zhang were selected to determine breach parameters and to compare and examine associated results.

Overtopping and piping modes of failure were simulated for each of the above named methods and has resulted in eight scenarios. 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.

- The physically based method of breach has resulted in an exaggerated peak flow and time to peak flow in both overtopping and piping modes of failures with a difference of 23693.22 m<sup>3</sup>/s and 12666.1m<sup>3</sup>/s respectively when compared with the next lowest values among the results.

- In general piping modes of failures are obviously found to result in lowest values of breach parameters when compared with overtopping results.
- The four non physically based empirical methods have resulted in peak flow values within a range of 24966.21m<sup>3</sup>/s and 16039.97m<sup>3</sup>/s during overtopping and 17348.94m<sup>3</sup>/s and 9843.82m<sup>3</sup>/s during piping. This results can be said more close when compared with the physically based model results.
- Vonthun method of breach estimation has resulted in a peak flow of 24966.21m<sup>3</sup>/s and Frohlich 2008 has resulted in a flow of 16039.97m<sup>3</sup>/s among the non physically based methods during overtopping.
- Once again Von thun method has resulted in a peak flow of 17348.94m<sup>3</sup>/s and Mac Donald et al has resulted in a minimum flow of 9843.82m<sup>3</sup>/s during piping.
- Side slopes are found to be in a range of 0.5H:V both sides to 1.45H:V both sides, all trapezoidal.
- Time of failure in hours is found to be in a range of 0.8hrs to 5.6hrs.
- All cases of inundation were mapped and in all cases similar villages ( odola, bora, arero and dagoda ) were inundated and two villages ( Wulia and Dibicha ) are found to be safe in any Sevier scenario.

## 5.2 Recommendation

- Cross-section measurements data at required River station were not available at dam owner (MoWIE) and engineer (WWDSE) offices. To fill this data gap cross-section geometry were generated from low resolution DEM (30mx30m details) by using HEC-GeoRAS software. There would, however, be further need to improve the generated cross-sections by taking cross-section measurements on reach or by generating cross-sectional geometry from high resolution DEM for the study area. This is because cross-sectional data generated from low resolution DEM (30m) might not be of sufficient span-width or maximum elevation required for the analyses. The topographic maps should be produced by actual topographic surveying or generated from finer DEM if possible.
- Since Gidabo dam is sensitive to break, as connected to its outlet settlement problem, further study should be continued and emergency action plan (EAP), shall be prepared.
- The area below the dam is relatively flat and people living on this area, specially near the river banks, shall be resettled at significantly far place.
- In any dam breach study, it would be good to check the breach by different scenarios, to see the range of hazardness when treated with different methods and to point out the optimal catastrophe so that emergency action planning preparation will be economical and safe.

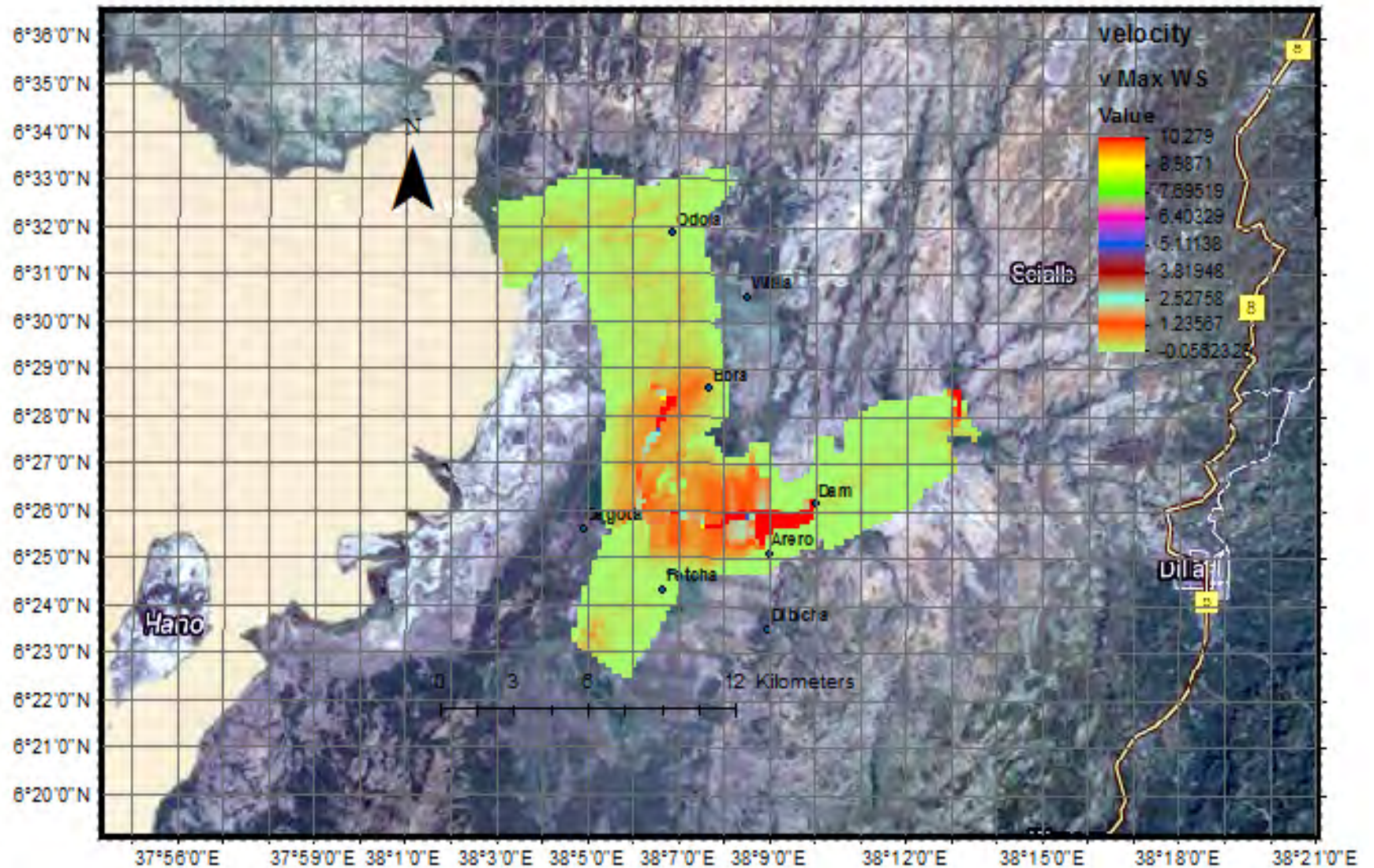
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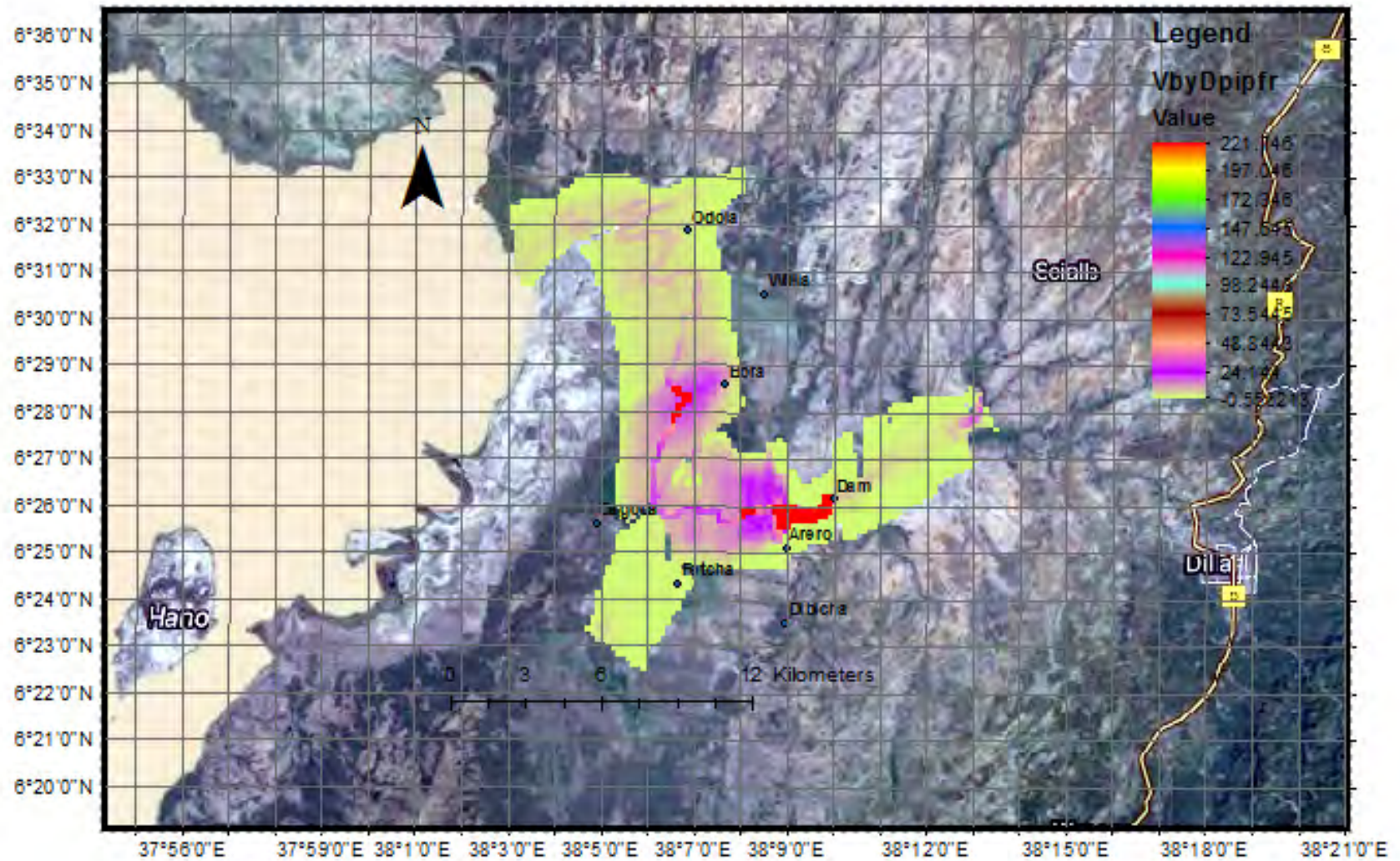
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Appendix A- Flood velocity, depth and hazardness maps due to piping when analyzed by different methods

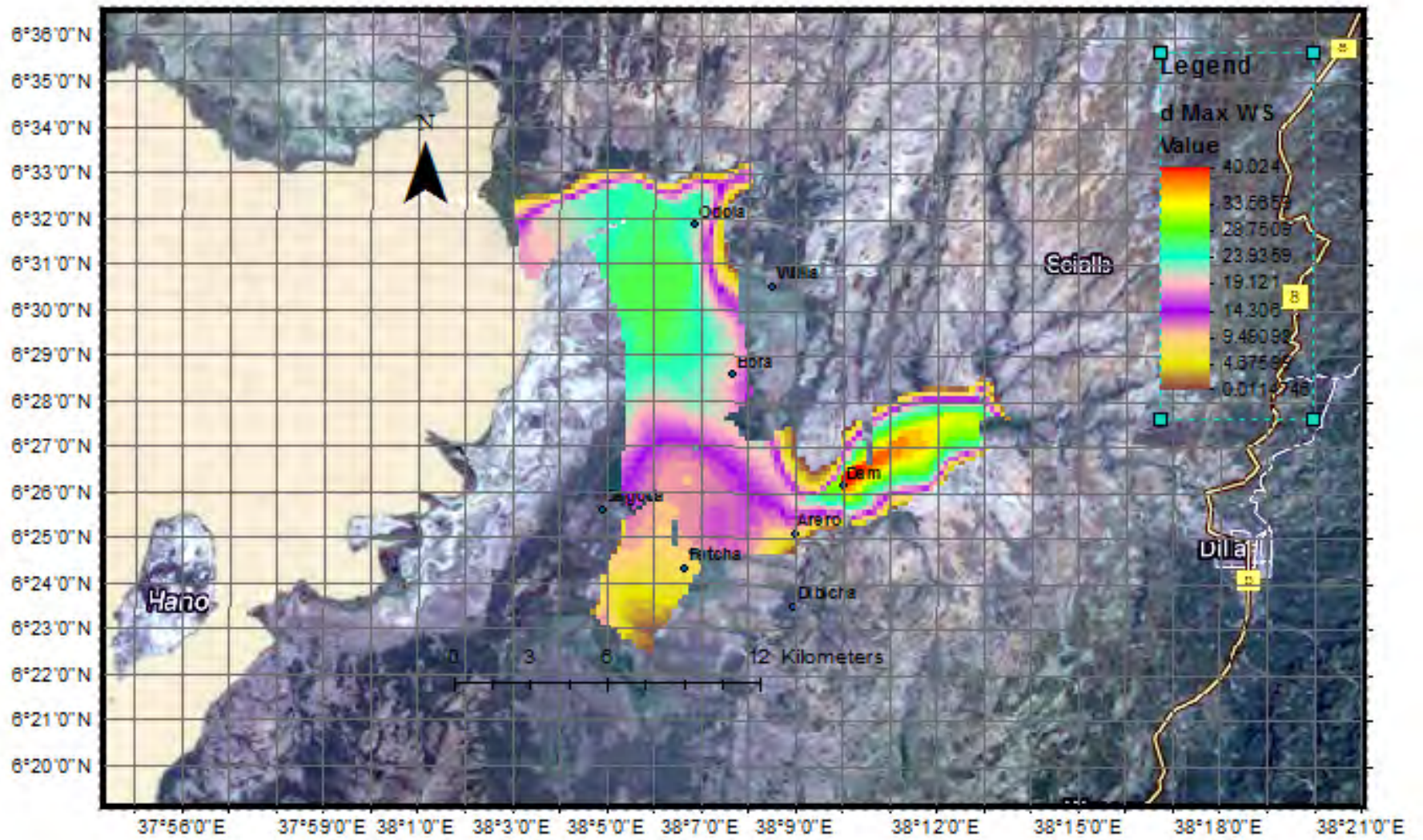




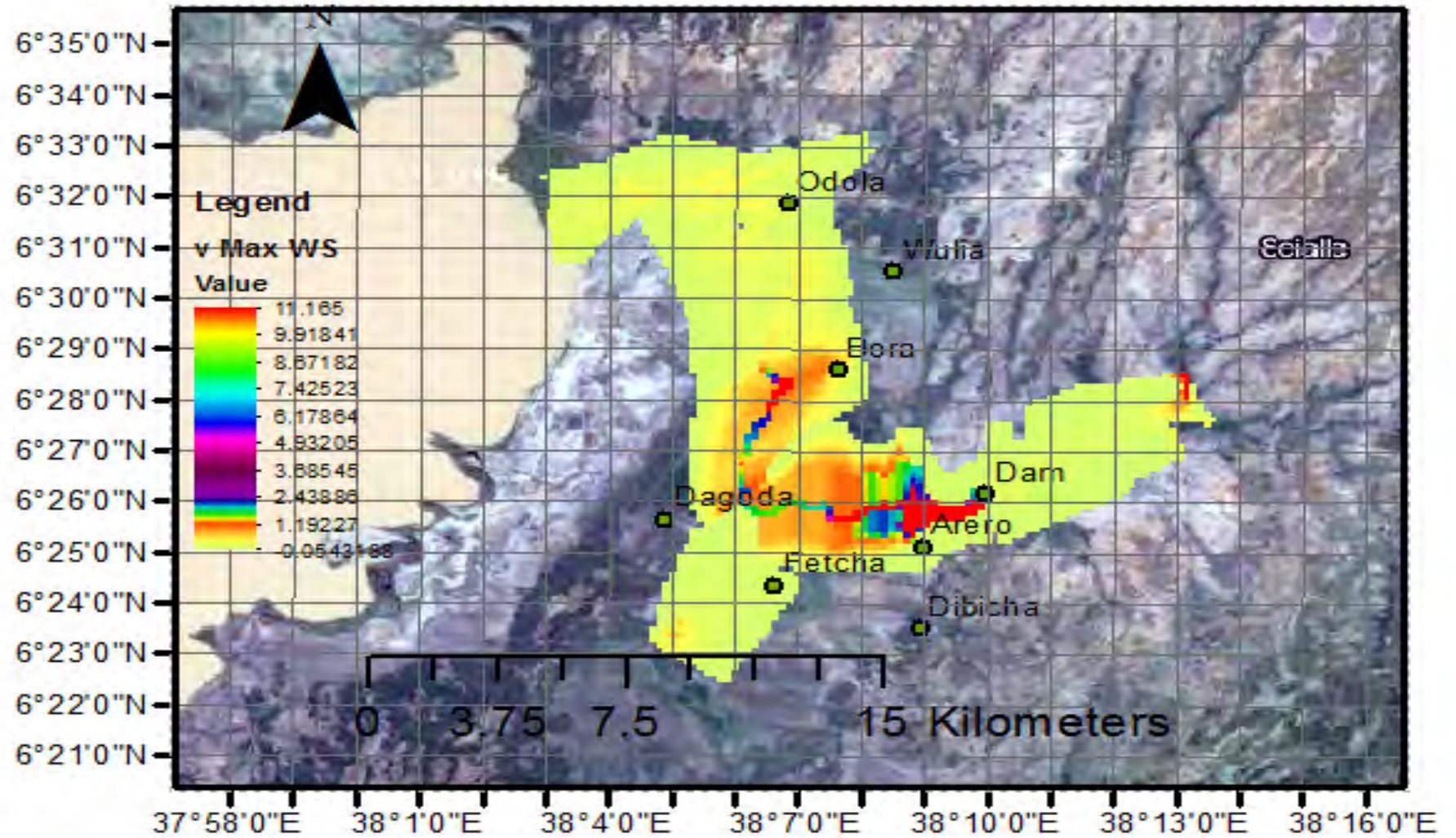
Velocity map of piping failure by Frohlich 2008 method



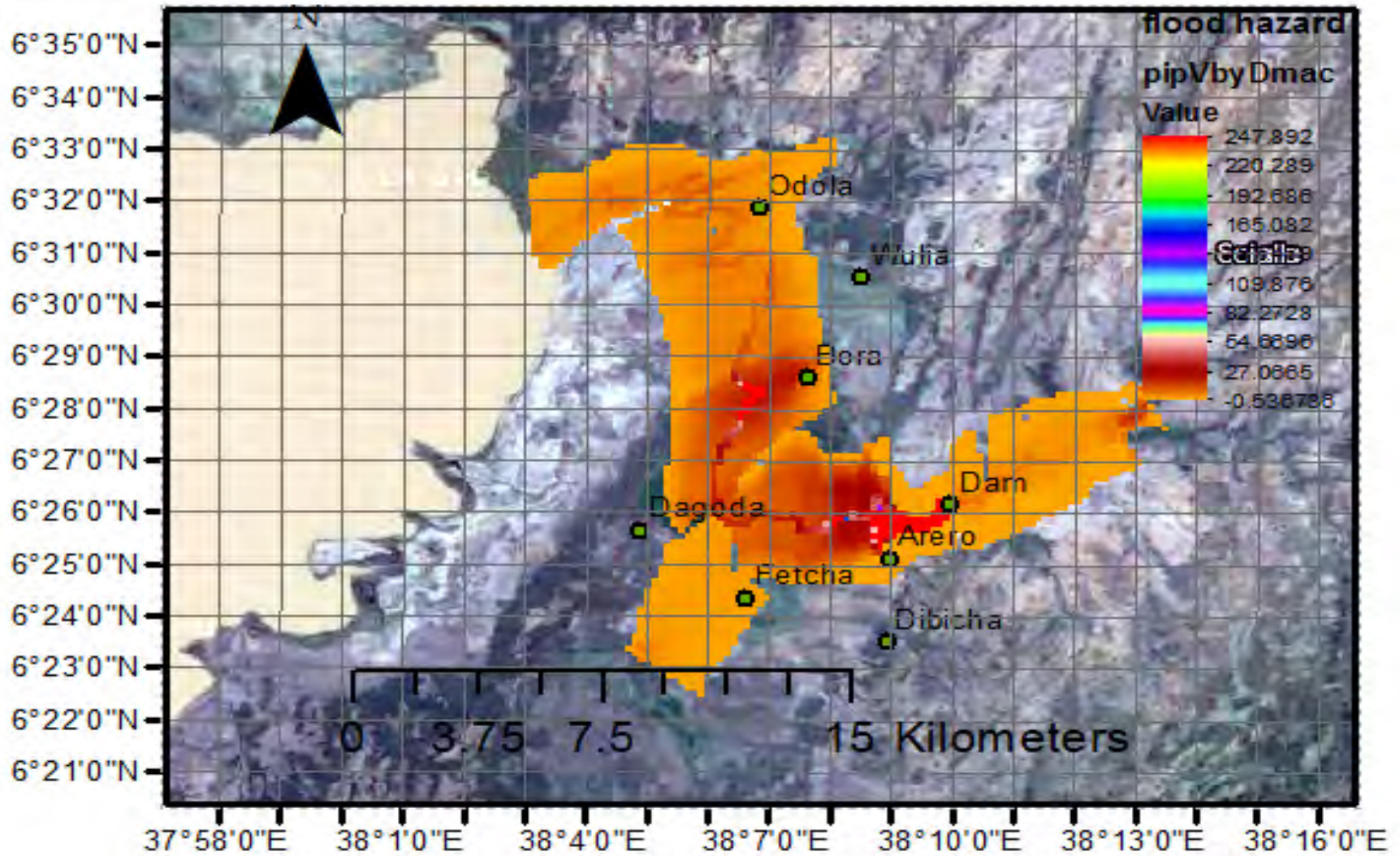
**Flood hazard map of piping failure by Frohelch 2008 method**



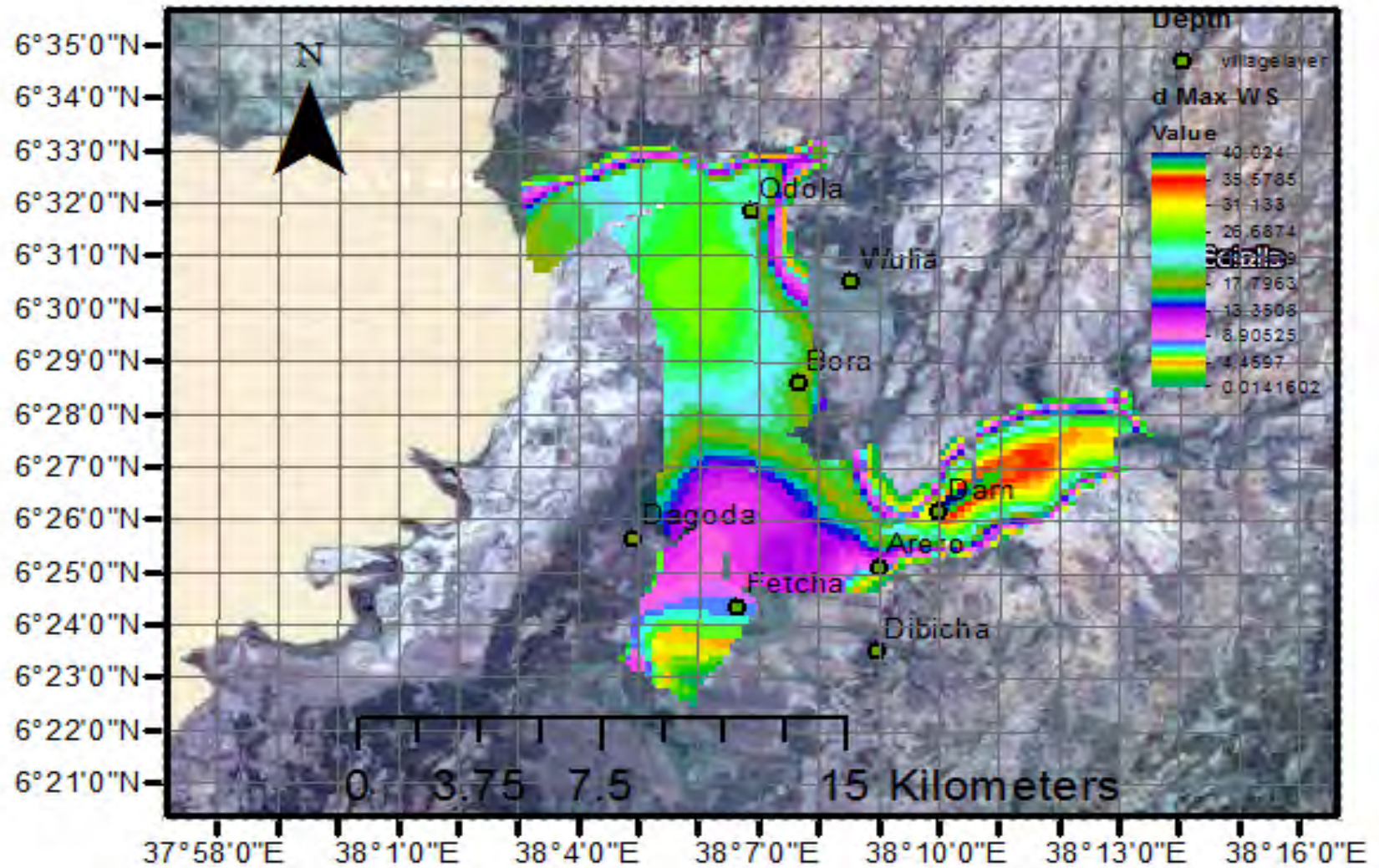
Depth map of piping failure by Froehlich 2008 method



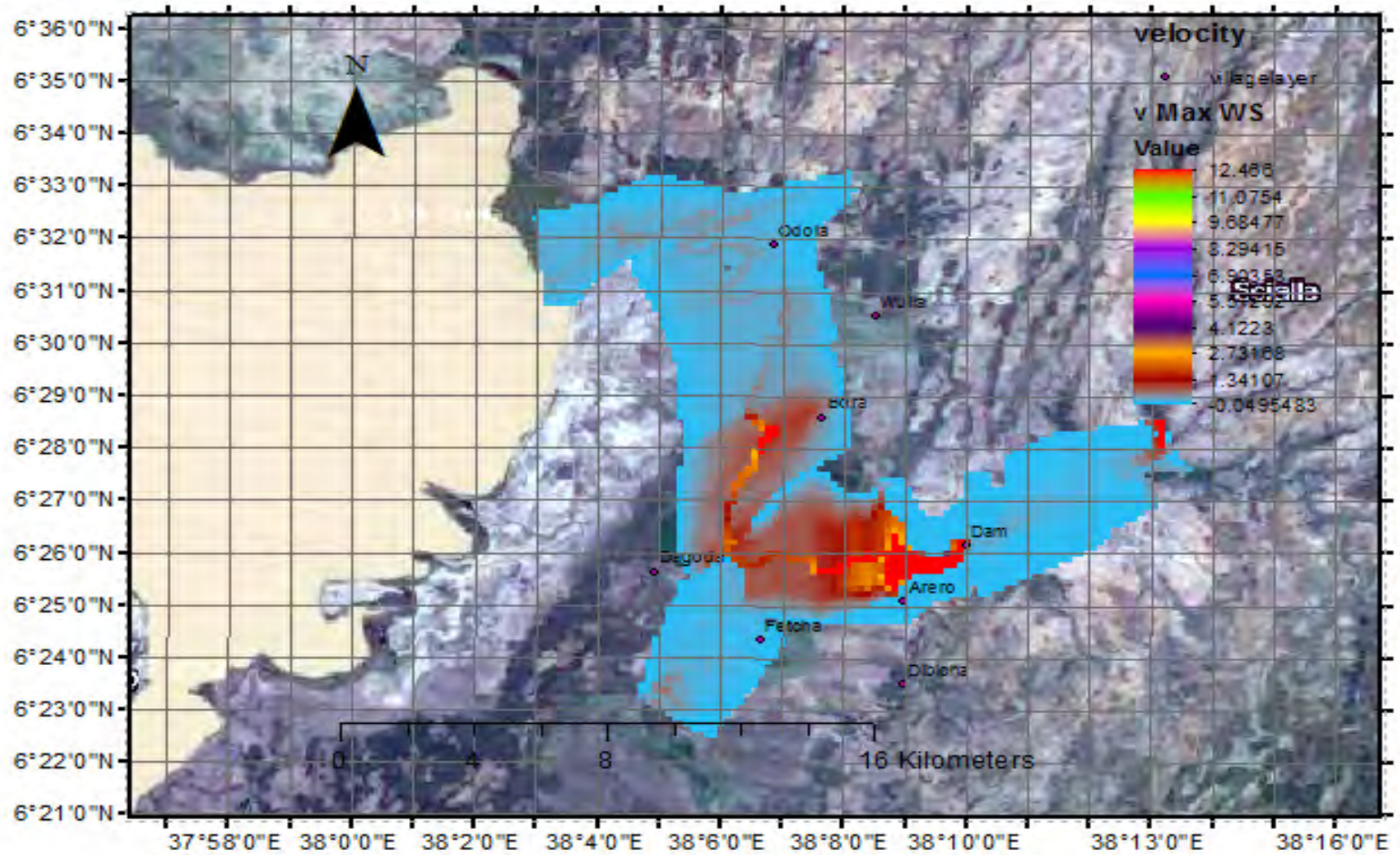
Velocity map of piping failure by Macdonald et al method



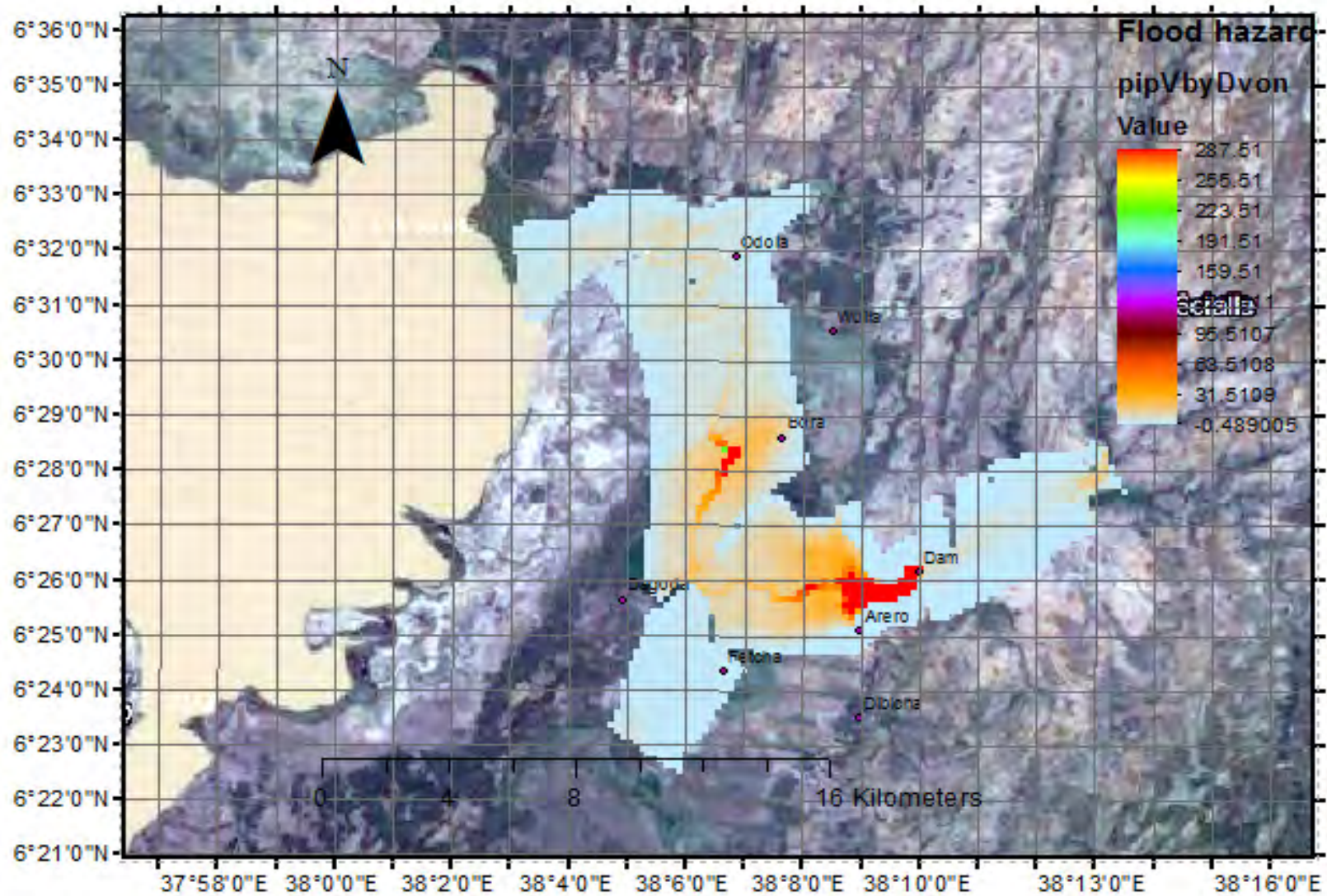
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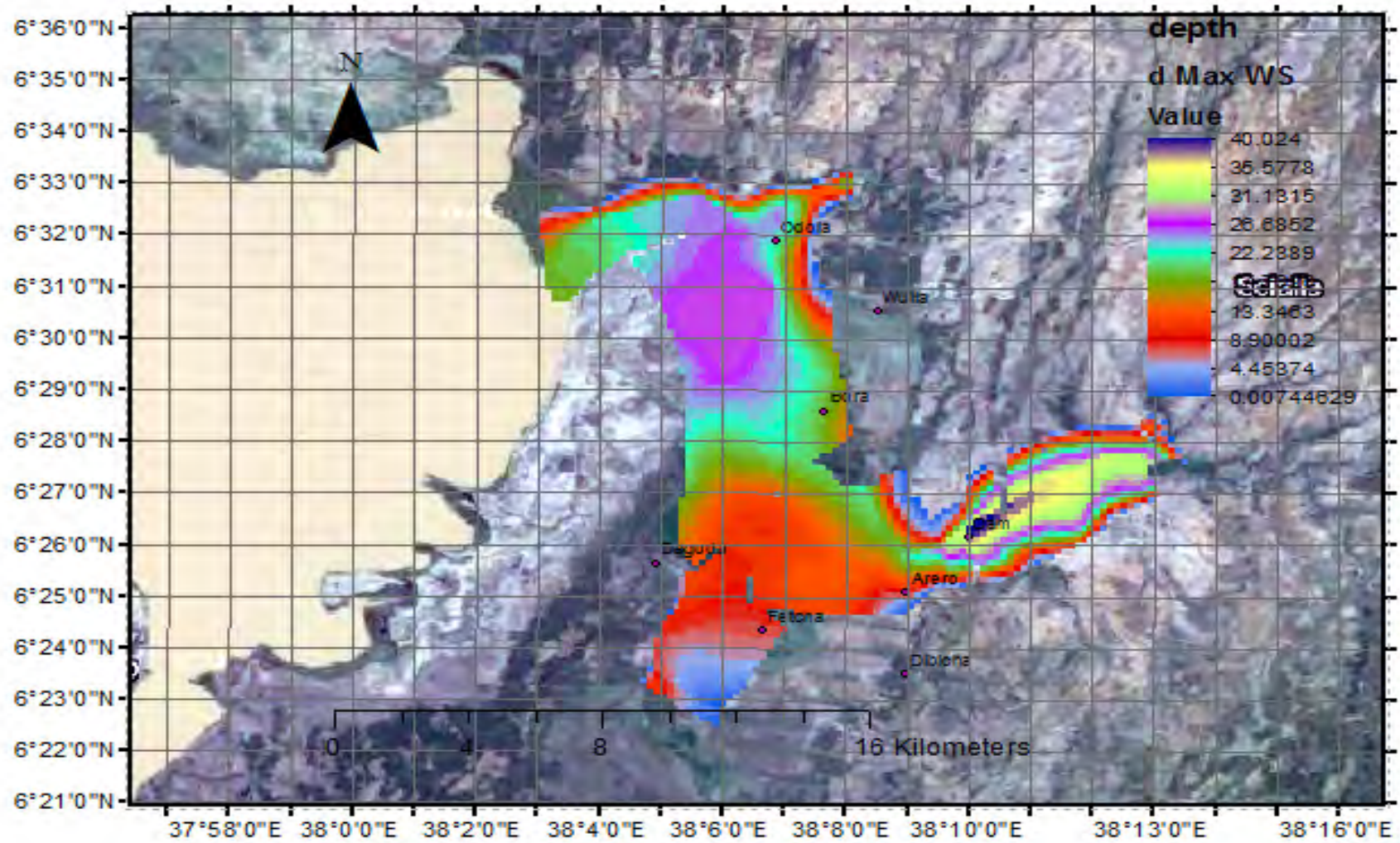
Depth map of piping failure by Macdonald et al method



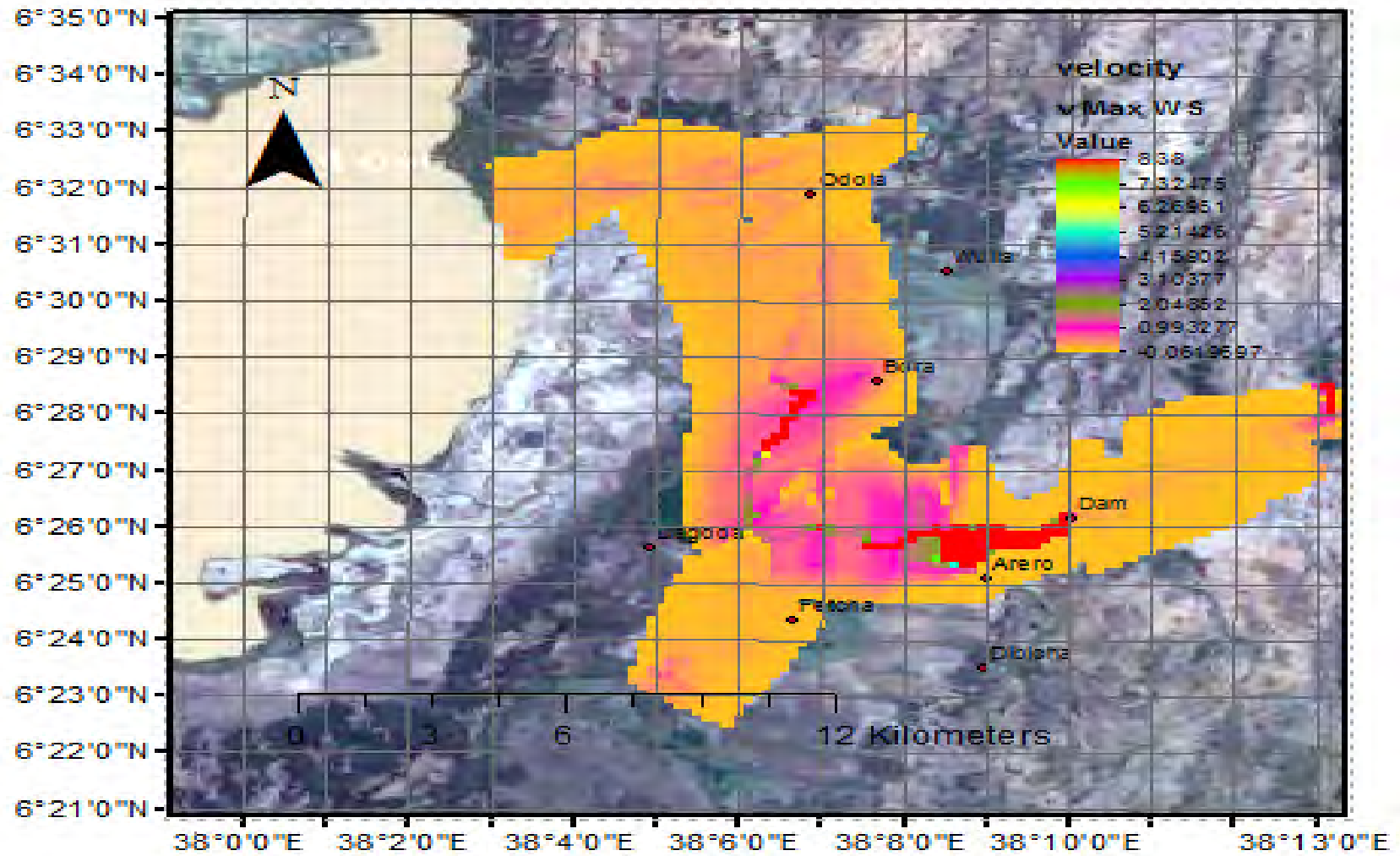
Velocity map of piping failure by Von thun and Gillette method



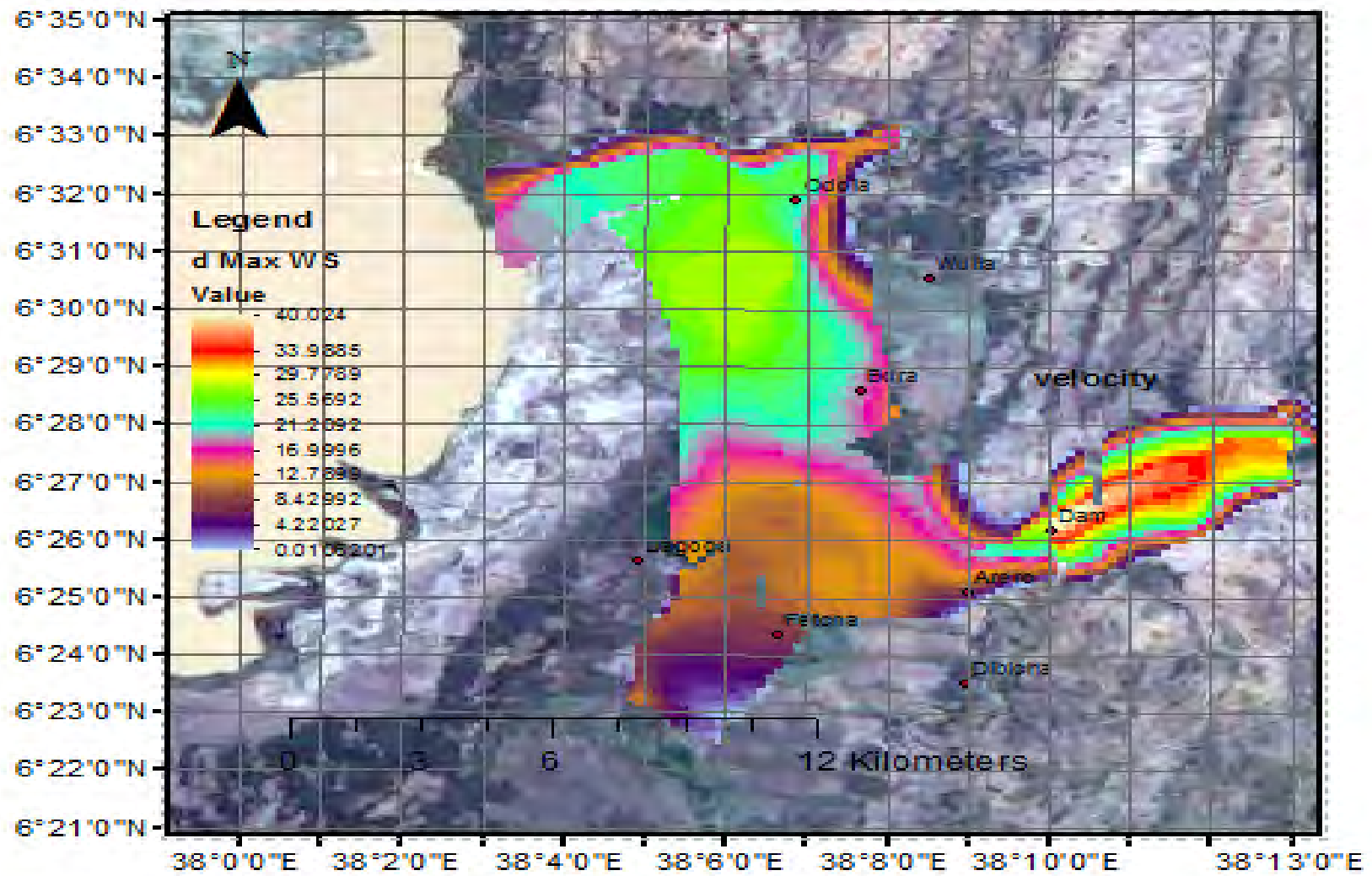
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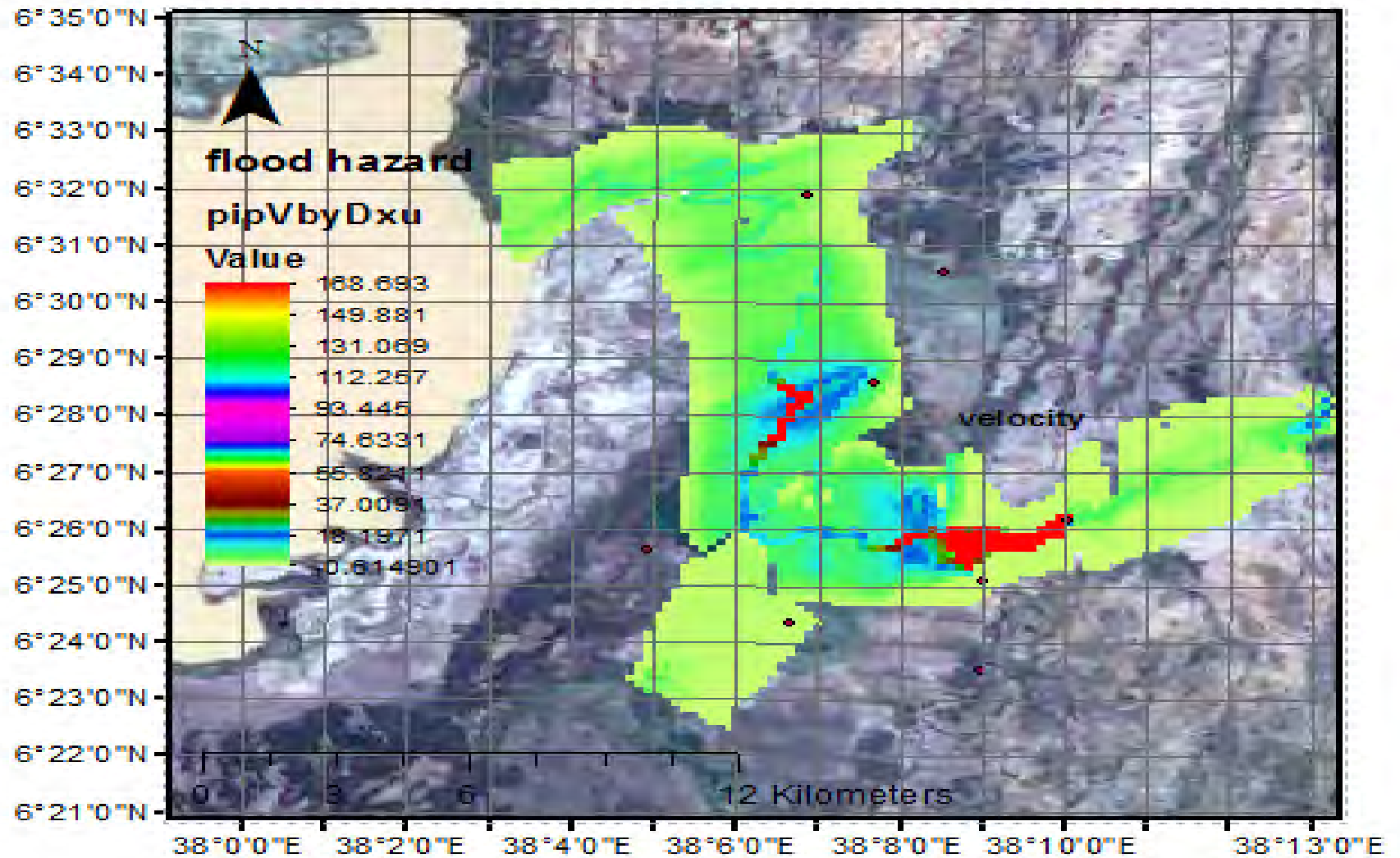
Depth map of piping failure by Von thun and Gillette method



Velocity map of piping failure by Xu and Zhang method

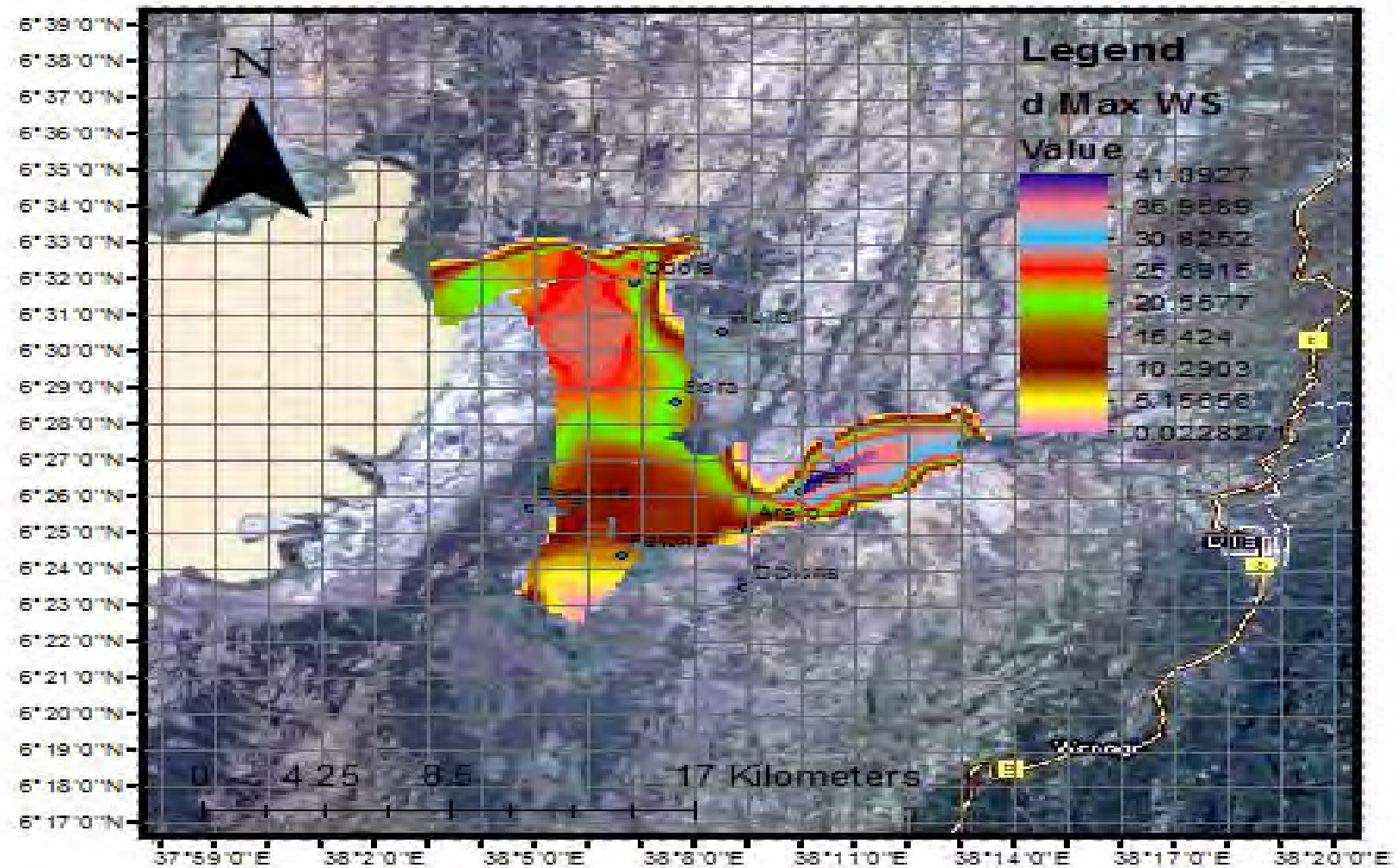


Depth map of piping failure by Xu and Zhang method

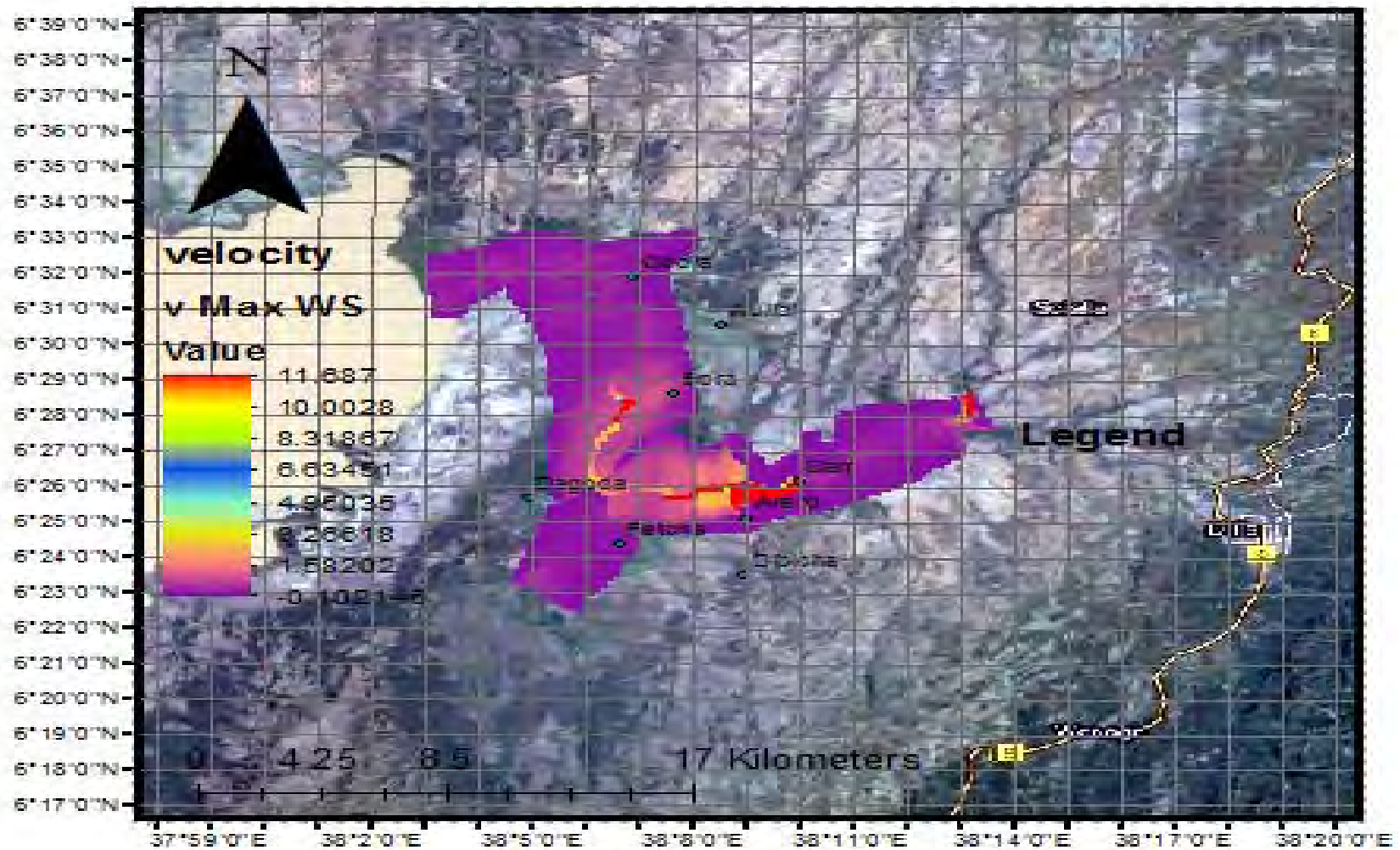


Hazard map of Piping failure by Xu and Zhang method

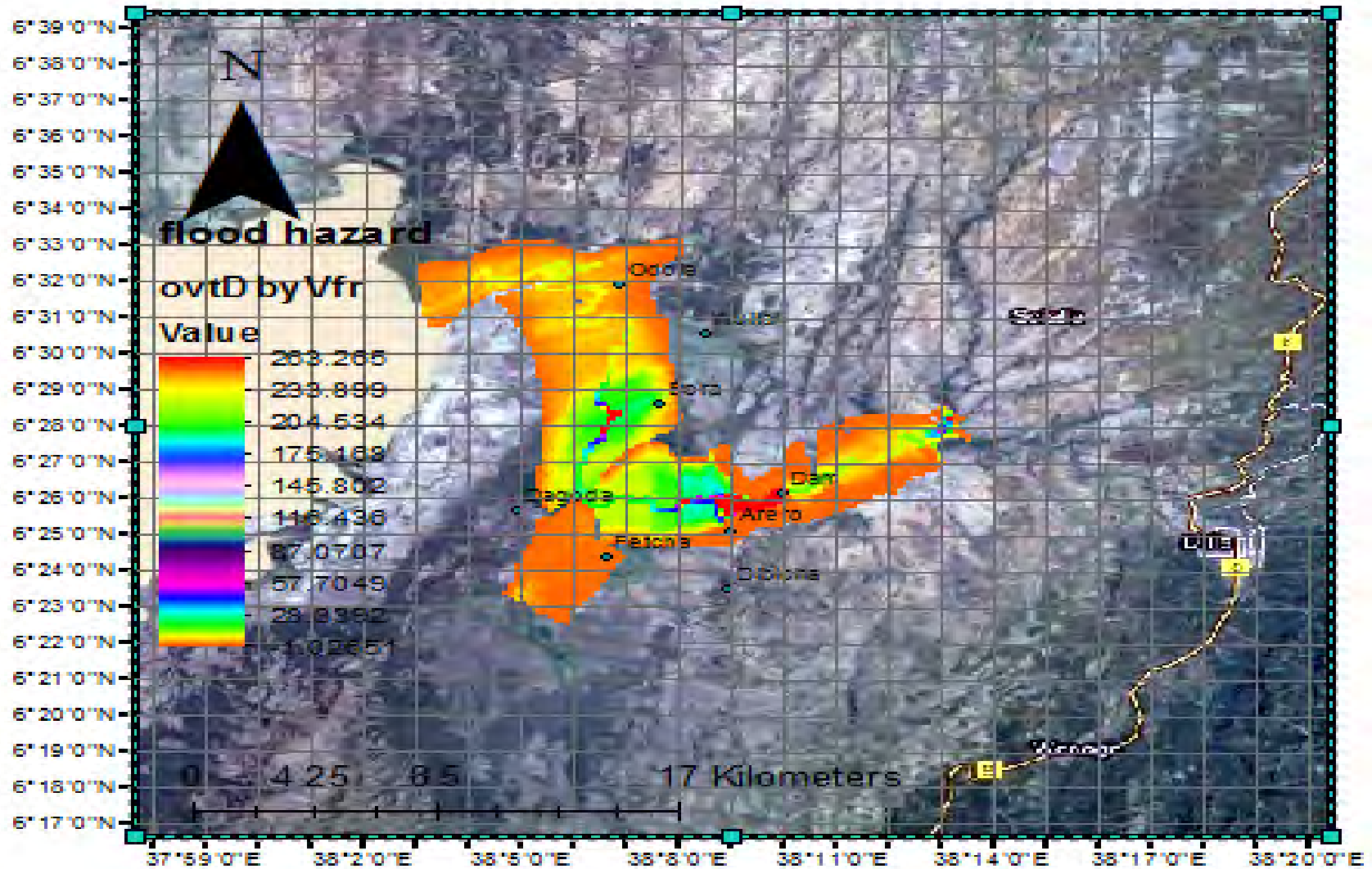
Appendix B- Flood velocity, depth and hazardness maps due to overtopping when analyzed by different methods



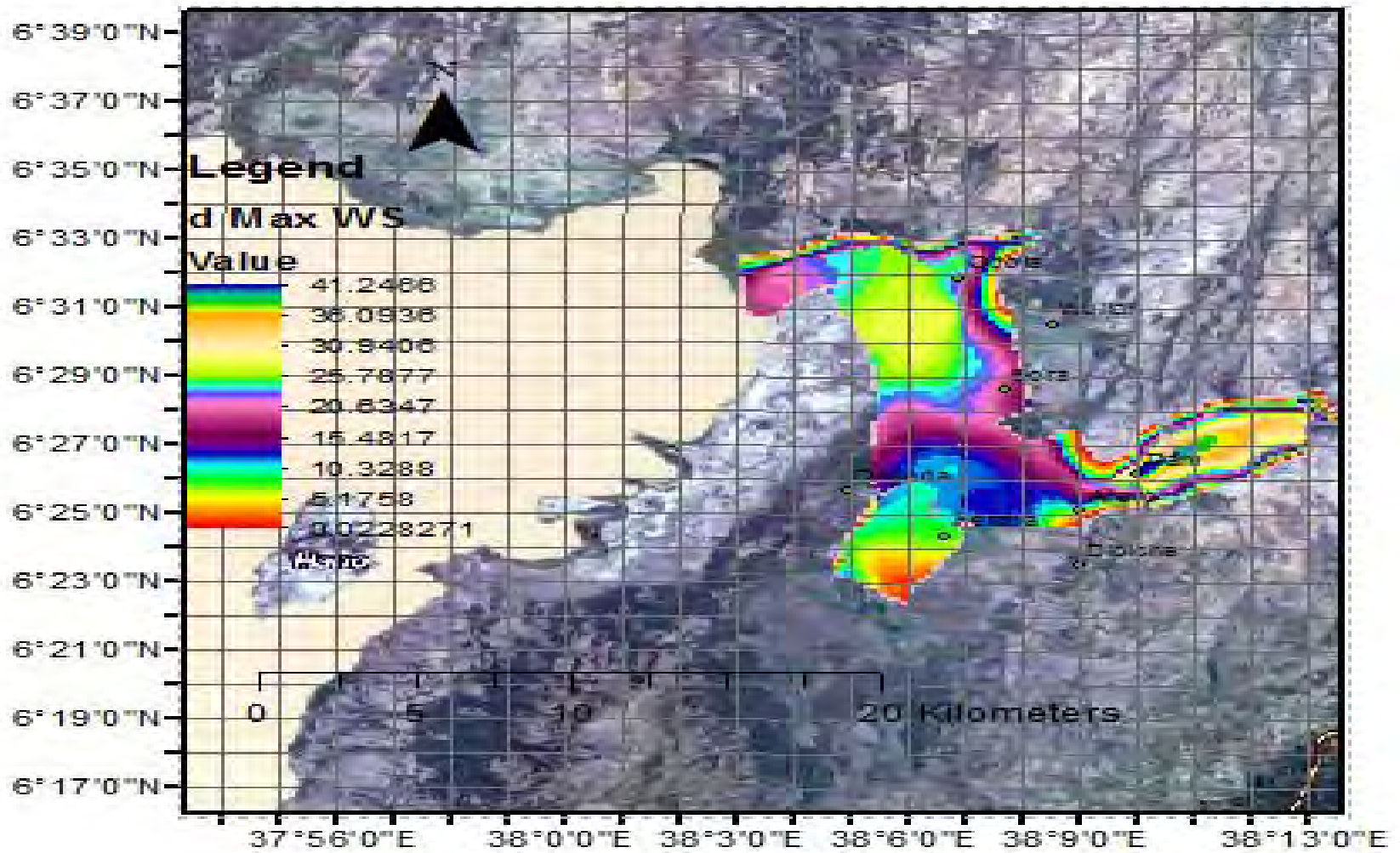
Depth map of overtopping failure by Frohlich 2008 method



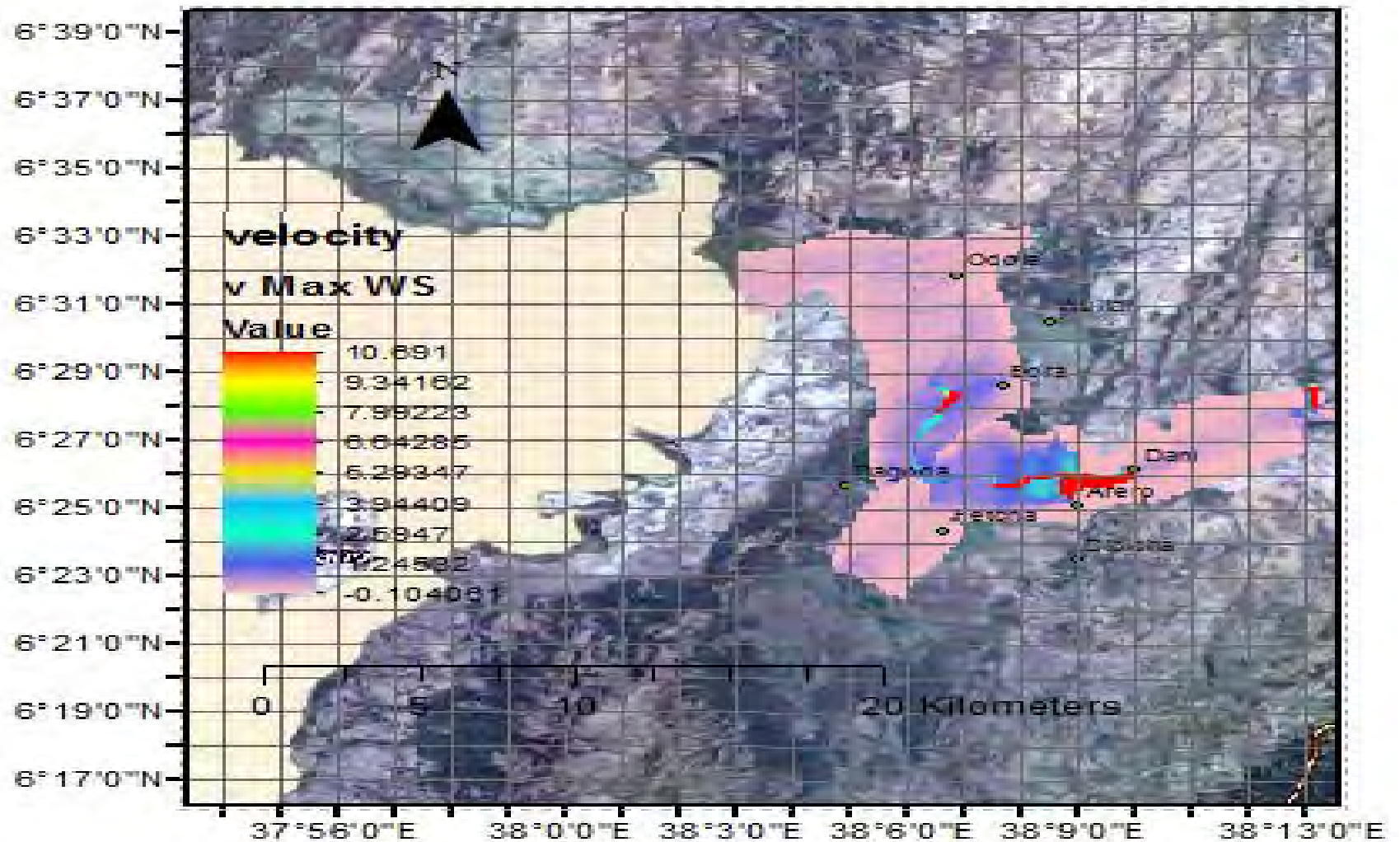
Velocity map of overtopping failure by Frohlich 2008 method



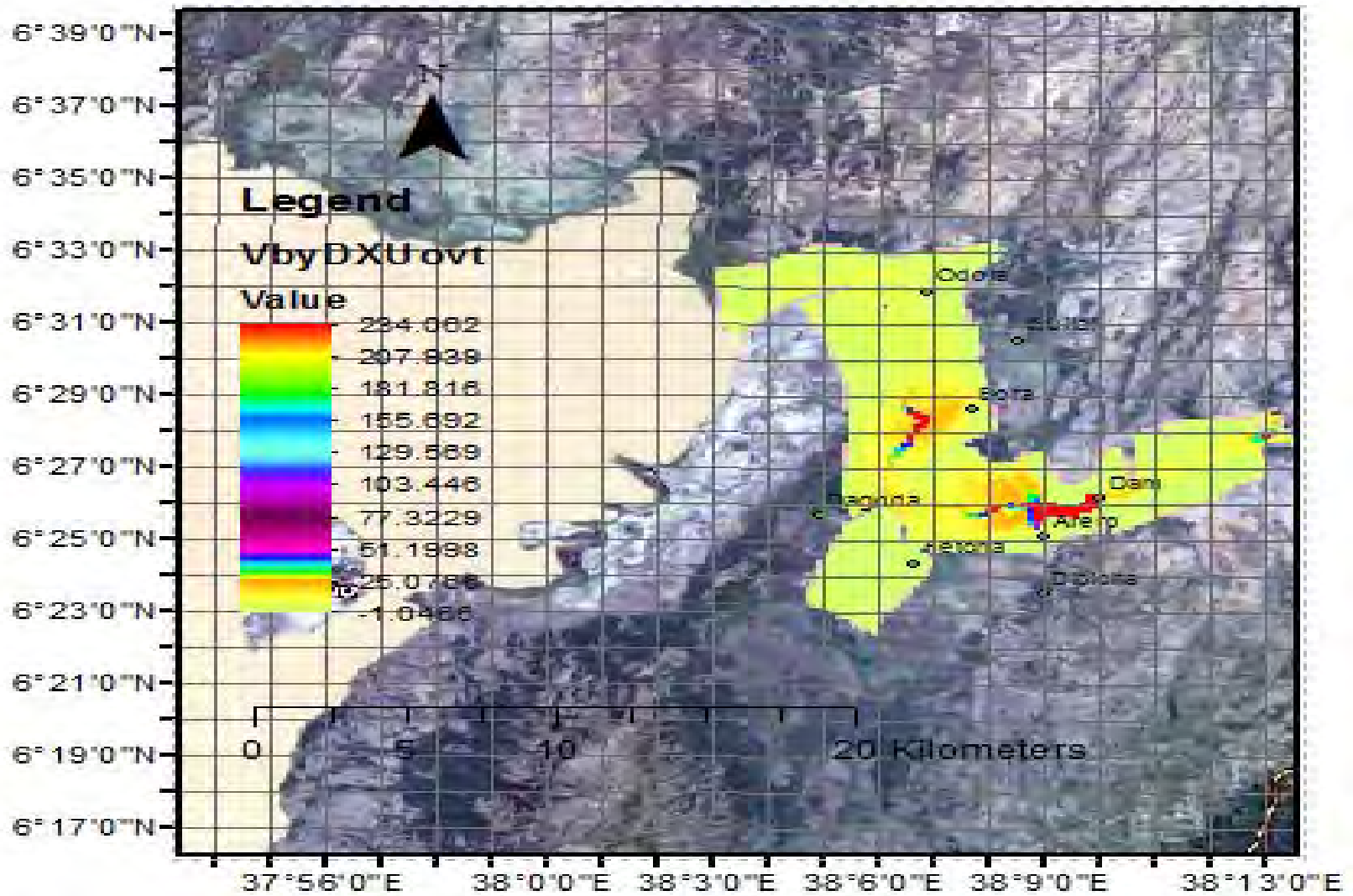
Hazard map of overtopping failure by Froehlich 2008 method



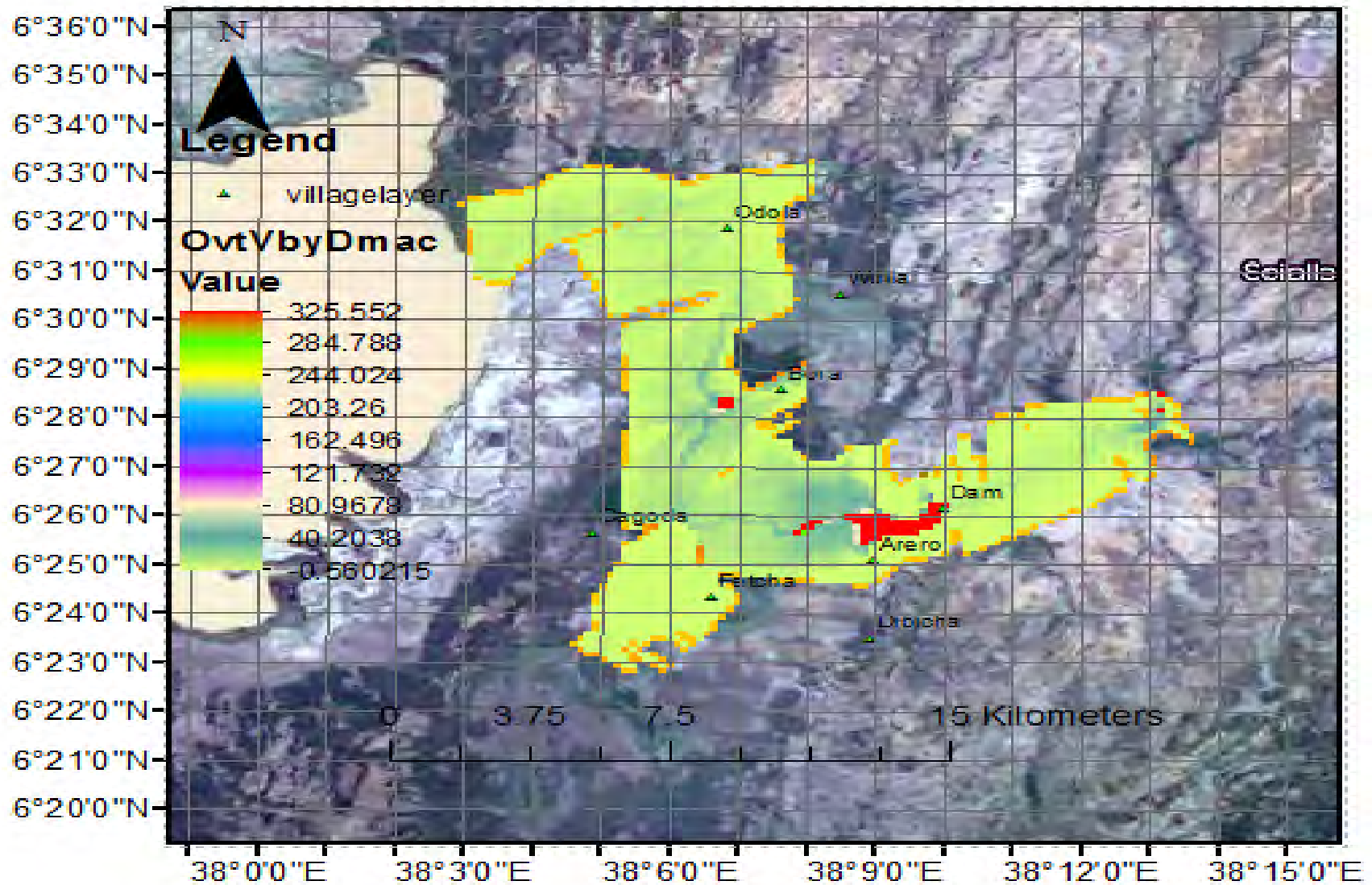
Depth map of overtopping failure by Xu and Zhang method



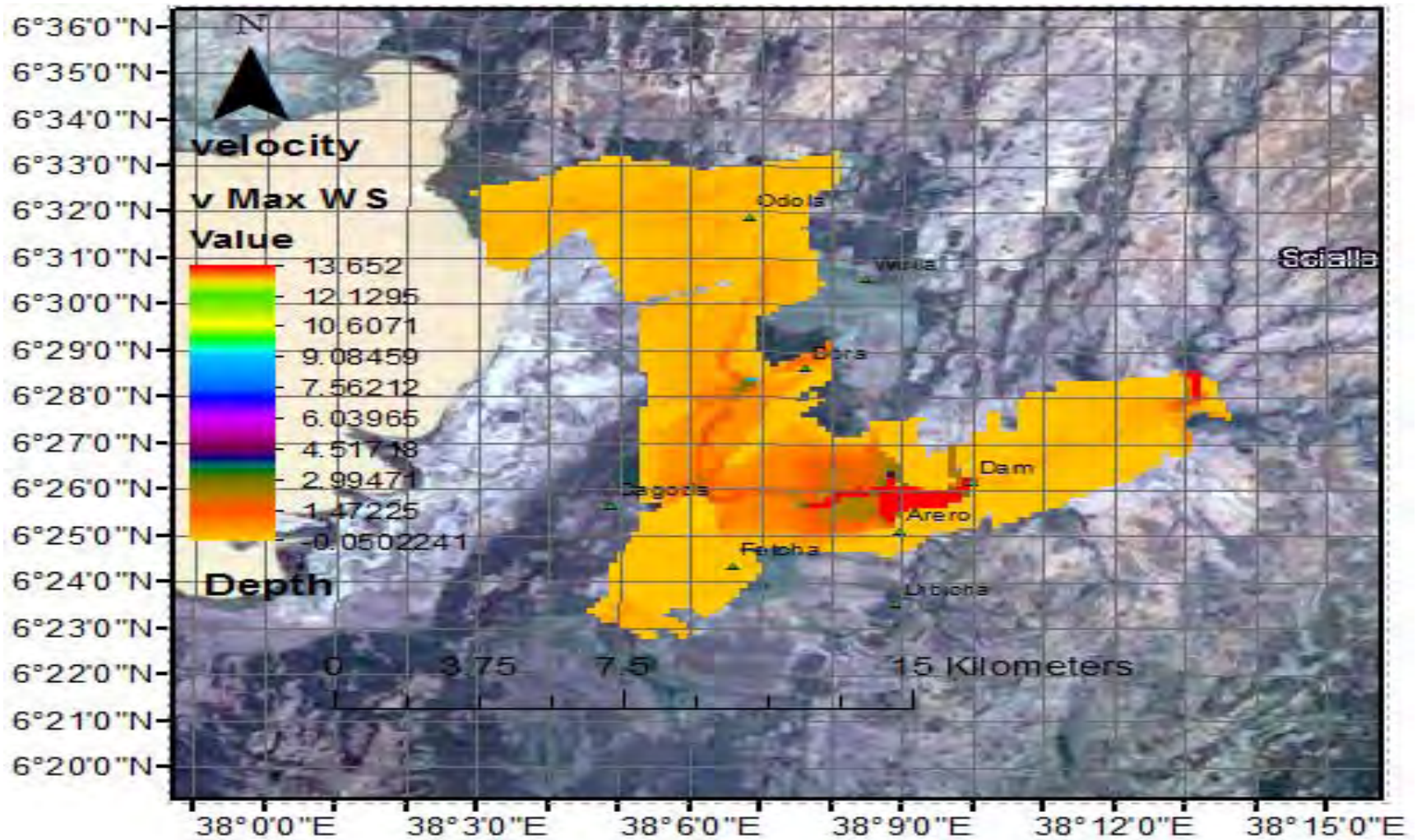
Velocity map of overtopping failure by Xu and Zhang method



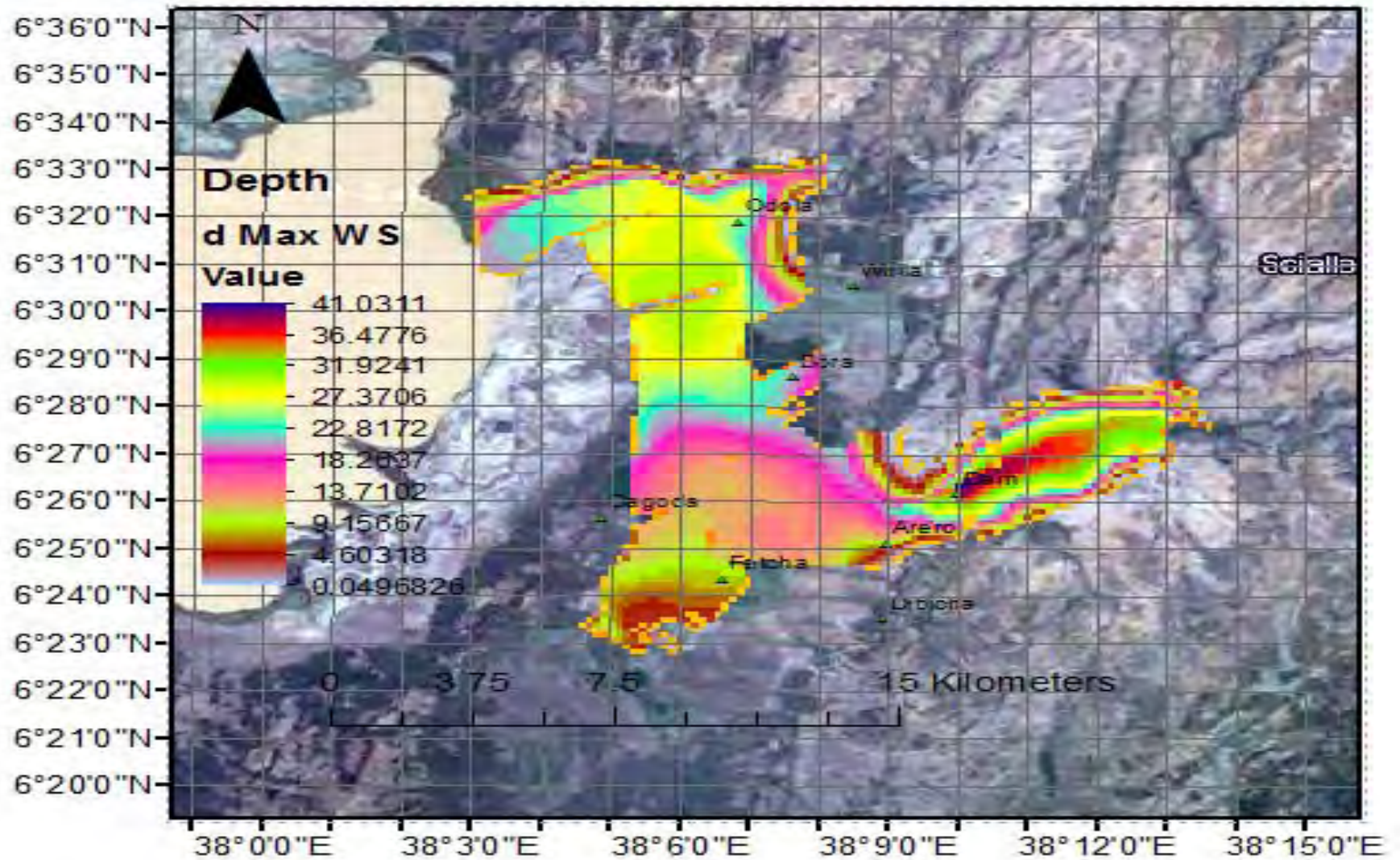
Flood hazard map of overtopping failure by Xu and Zhang method



Flood hazard map of overtopping failure by Macdonald et al method



Velocity map of overtopping failure by Macdonald et al method



Depth map of overtopping failure by Macdonald et al method