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DAM BREACH ANALYSIS AND FLOOD INUNDATION MAPPING FOR GIDABO DAM

BY

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DAM BREACH ANALYSIS AND FLOOD INUNDATION MAPPING FOR GIDABO DAM

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(SPECIALIZATION: **HYDRAULIC ENGINEERING**)

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ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "Dam Breach Analysis and Flood Inundation Mapping for Gidabo Dam" submitted in partial fulfilment of the requirements for the degree of Master's with specialization in Hydraulic Engineering, the Graduate Program of the Department of Civil Engineering, and has been carried out by Firaol Befekadu, ID. No: PGHYD/200/07, under my supervision. Therefore, I recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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We, the undersigned, members of the Board of Examiners of the final open defence by have read and evaluated his thesis entitled "Dam Breach Analysis and Flood Inundation Mapping for Gidabo Dam", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering with specialization of Hydraulic Engineering.

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I hereby declare that this MSc thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

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LIST OF ABBREVIATIONS

ALOS PALSAR	Advanced Land Observing Satellite - Phased Array type L-band Synthetic Aperture Radar
BFF	Breach Formation Time
DEM	Digital Elevation Model
EAP	Emergency Action Plan
ECDSWC	Ethiopian Construction Design and Supervision Works Corporation
EMA	Ethiopian Mapping Agency
ERA	Ethiopian Road Authority
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory commission
GIS	Geographic Information System
HEC	Hydrologic Engineering Center
HEC-RAS	Hydrologic Engineering Center River Analysis System
HEC-GeoRAS	Hydrologic Engineering Center - Geographic River Analysis System
MoWIE	Mistry of Water, Irrigation and Energy
PMP	Probable Maximum Precipitation
PMF	Probable Maximum Flood
TIN	Triangulated Irregular Network
USACE	United State Army Corps of Engineering
WWDSE	Water Works Design and Supervision Enterprise

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ABSTRACT

This study presented the dam breach analysis and flood inundation mapping for Gidabo rock-fill embankment dam found in Southern part of Ethiopia. The geometrical data used for this study was extracted from the recently released ALOS PLASAR digital elevation model by Alaska Satellite Facility, which is having the spatial resolution of 12.5 by 12.5 meters. One dimensional unsteady flow simulation within HEC-RAS model was used to simulate dam breaching for both overtopping and piping failure scenarios. Dam breach parameters estimation was done for different empirical equations. Froehlich (2008) and Von Thun & Gillete methods was preferred since the results obtained for these methods are more approaches to the envelope curve developed for the historical dam failures for overtopping and piping failures, respectively. RAS-mapper and ArcGIS tools was used to present the maps of spatial distribution of flood extent, flood depth and flood velocity, flood inundation, flood hazard maps of the study area. The breach parameters estimated for both overtopping and piping failure scenarios was provided the reasonable values. The maximum breach discharge simulated for overtopping and piping failure was obtained as 15,945.18 m^3/s *and 14,904.18* m^3/s*, respectively. Since the failure were tested for hydrologic failure and for normal water level condition, the magnitude of flood and the spatial distributions are obviously different. Developed inundation maps from this study could be possibly help as guidance for dam owners to develop the emergency action plan and for future expansion of irrigation project infrastructures, and other developmental activities around the downstream of the dam.*

Keywords: *HEC-RAS, HEC-GeoRAS, DEM, Dam Breach, Inundation Mapping*

1. INTRODUCTION

1.1 Background

Dams are one of the most significant infrastructures that provide the benefits to flood control, irrigation, water supply, hydropower, navigation, and also used for recreation purposes. However, it is an evident of historical fact that failure of the dams have resulted in high casualties (Pandya et al., 2013). Hence, the prediction of dam breach is the necessary steps for forecasting and evaluation of flooding disaster and preparation of an emergency action plan (Shahapure, 2017). The development of an effective emergency action plans requires accurate prediction of inundation levels and the time of flood wave arrival at a given location (Xiong, 2011). Historically, failures of the dams that has been occurred during 1960 to 1970 have led to the inception of a new research field that deals with dam failures and its resulting consequences (Pandya et al., 2013).

The dam failures are comparatively rare; however, once they occur impose immense damage and loss of life and properties. Thus, being one of the most destructive failures, a wide range of dam breach analysis have been conducted (Sharma & Mujumdar, 2017). Modeling a dam breach is a critical step in helping to identify potential damage to structures and property, and most importantly, to prevent casualties (Asburry, 2009; Xiong, 2011). Basically, dam breach analyses include estimation of the dam breach parameters, estimation of the dam breach outflow hydrograph, routing of the dam breach hydrograph, and estimation of downstream inundation extent and severity (FERC, 2014).

Usually, dam breach prediction models are based on empirical data derived from a number of case studies, mostly from earth and rock fill embankment dam failures (FERC, 2014). The empirical equations relate the dam breach parameters to properties of the dam and reservoir such as height, dam type and its erodibility, volume impounded, and shape of the reservoir (FERC, 2014). Some of the most widely use statistical regression in empirical equations was developed by scholars such as MacDonald and Langridge-Monopolis (1984), USBR (1988), Von Thun and Gillette (1990), Dewey, Gillette (1993), and Froehlich (1995, 2008). Currently, breach modeling is further advanced with its prediction capability for breach outflow hydrograph and routing of the hydrograph downstream with the combination of physical models and empirical equations.

The term *breach modeling* is basically refers to the process of understanding and simulating the breach development in a dam (Pandya et al., 2013). It is mainly includes the estimation breach parameter, breach peak discharge and its hydrograph, breach flood routing, and the estimation of the hydraulic conditions at critical locations (Colorado, 2010). There are number of modeling tools has been developed ranging from simple to complex models. With advancements in modeling, many models like HEC-GeoRAS can interface with geographic information systems (GIS) tool to extract geometric data from digital terrain data to produce automated dam breach inundation zone delineations (Abdelbasset et al., 2015; FEMA, 2013).

In the dam breach analysis, there are three most commonly used approaches named as event-based approach, risk-based approach and tiered dam breach analysis approaches (FEMA, 2013). According to FEMA (2013), risk-based approaches become more acceptable method for dam safety and design purposes in the past two decades. Dam breach analysis can provides multipurpose from evaluating the already happened problems to establishing appropriate spillway design for the dams. Inundation maps produced from dam breach is also of great important for decision makers. In order to produce an accurate simulation results, the accuracy level of input data and/or parameters is more significant.

Within this study, it is aimed to use the combination of HEC-RAS, HEC-GeoRAS and ArcGIS modelling tools for the dam breach analysis and inundation mapping for Gidabo embankment dam. A number of literature review shown that, HEC-RAS one-dimensional unsteady flow simulation can provides more reliable results in dam breach study (Kumar et al., 2017). In order to produce appropriate accurate results, it may need detail surveying data to extract geometric data within HEC-GeoRAS, which is costly and time consuming task. Therefore, it is aimed to utilize digital elevation model (DEM) recently released online by Alaska Satellite Facility (ALOS PALSAR product which is having 12.5 x 12.5 m spatial resolution). The other data required executing the study like hydrological data and dam salient features can also be collected from the Ethiopian Construction Design and Supervision Works Corporation.

1.2 Statement of the Problem

Uncertainties related to design of capacity of dams are enormous. Among others, land use land cover, and climate change possibly alter flow patterns. This could lead to unprecedented inflows to the dam which may responsible to dam failure. Thus, it is imperative to investigate possible flow scenarios that may endanger lives and properties if dam failure occurs.

There are a number of dams which have been constructed and also under construction in our country. One of the good news is that, there is no failure of dam recorded yet in Ethiopia. However, Kesem Dam and Tendaho Dam are suspected to failure due to piping failure. Obviously, the dam breach modeling is the most significant task to identify the possible causes of failure, simulate the breaching process so that design parameters can be reviewed, mapping the area that shall be flooded in order to demarcate prepare emergency action plans. Generally, it is good practice to analyze probable consequences of damage before failure of the dam take place. And it is why this study is focused on dam breach analysis and inundation mapping for the Gidabo embankment dam.

1.3 Objectives of the Research

1.3.1 General Objective

The general objective of this research is to analyze the dam breach and develop inundation map for the Gidabo embankment dam using HEC-RAS and HEC-GeoRAS models.

1.3.2 Specific Objectives

The specific objectives of this research are:

- To assess dam breach parameters (breaching size and time, failure modes)
- To rout outflow hydrographs downstream of the dam
- To develop inundation map that provides areal extent of the flood

1.4 Research Questions

The research question should be answered after completions of simulations are:

- What are the key parameters that influence the estimation of dam breach parameters?
- What is the magnitude of the flood that would be produced at the downstream of the dam?
- Which part of the downstream area would be more prone to the flooding effect?

1.5 Significance of the Study

An attempt was made to predict dam breach flood inundation to identify possible flood prone area at the downstream of the dam in case of dam failure. The study enhances the scientific, real information, and identifies the problem of the study area to familiarize the community with the infrastructure to create the safe environment. Hence, the output of this study could be used to assist the dam owners (MoWIE) in order to develop emergency action plan, propose mitigation plan (structure or non-structural), future expansion of irrigation command area and etc.

1.6 Thesis Organizations

The thesis is organized in six chapters. Chapter one deals with the introduction, statement of the problem, research (objectives and questions), and the significance of the study as shown above. Chapter two describes the literature review on the history of dam breach, dam breach analysis and modeling, breach modelling with HEC-RAS, hazard classification systems and potential use of inundation mapping. Chapter three provide an overview study area and data sets used in this study. Chapter four is about methodology of the research adopted for this study using HEC-RAS and HEC-GeoRAS. Chapter five is describes result and discussions. Chapter six is includes conclusion and recommendation. Finally, references and appendices are also provided.

2. LITERATURE REVIEW

2.1 History of the Dam breach

Historically, several loss of life and property damage has been recorded over the world wide due to the failures of the dams. All types of the dams have experienced failure due to one or more types of events. However, by far the majority of the dam failures that occurred have been an embankment dams. Under this section, a review of historic dam failure occurred over the world during the past periods and the historical development of dam breach analysis will be discussed.

2.1.1 World Historic Dam Failures Recorded

Regardless of their design or construction, dams have increased forces applied to them during extreme events which increase the potential risk of failure. Failures may occur due to a variety of causes such as a significant hydrologic event, structural failure, seismic activity, operational error, or other deficiencies (Wahl, 1998). When the dam breach occurs, an uncontrolled release of water impounded behind the structure will cause flooding in the downstream area. Hence, the energy of the water that stored behind the dam is capable of causing rapid and unexpected flooding downstream that may resulting in loss of life and great property damage (Alvarez et al, 2017). The failure may occur in several forms, mainly as overtopping and piping modes for an embankment dam. Table 1 summarize the major failures of dams occurred on the world during the past period of time.

Table 1: Summarized worldwide major dam failures, Source: ("Dam Failure," n.d.)

Dam	Year	Location	Fatalities	Remarks
Marib Dam	575	Sheba, Yemen	unknown	Provoked the migration of up to 50,000 people from Yemen
Puentes Dam	1802	Lorca, Spain	608	1,800 houses and 40,000 trees destroyed
Bilberry Reservoir	1852	Holme valley, United Kingdom	81	Failed due to heavy rain
Dale Dike Reservoir	1864	South Yorkshire, United Kingdom	244	Defective construction, small leak in a wall, > 600 houses were damaged or
Mill River Dam	1874	Williamsburg, United States	139	Insufficient design, 600 million gallons of water were released
South Fork Dam	1889	Johnstown, United States	2209	Following heavy rainfall

Walnut Grove Dam	1890	Wickenburg, United States	100	Heavy snow and rain following public calls by the dam's chief engineer to strengthen the earthen structure
Austin Dam	1911	Austin, United States	78	Poor design, use of dynamite to remedy structural problems. Destroyed paper mill and much of the town of Austin
Tigra Dam	1917	Gwalior, India	1000	Failed due to water infiltrating through foundation. Possibly more fatalities
Gleno dam	1923	Province of Bergamo, Italy	356	Poor construction and design
St. Francis Dam	1935	Santa Clarita, United States	600	Geological instability of canyon wall
Dam of Sella Zerbino (2nd)	1942	Molare, Italy	111	Geological unstable base combined with flood
Mohne Dam	1943	Ruhr, Germany	1579	Destroyed bombing during operation Chastise in world war 2
Kurenivka Mudslide	1961	Kiev, Ukraine	1500	Caused by heavy rains
Panshet Dam	1961	Pune, India	1000	Dam burst due to pressure of accumulated rain water
Vajont dam	1963	Monte Toc, Italy	2000	The dam did not collapse but 110km/h landslide fill the reservoir, and water escaped over the top of the dam
Sempor Dam	1967	Central Java, Indonesia	2000	Flash floods overtopped the dam during construction
Banqiao ShimantanDam	1975	Zhumadian, China	171000	Extreme rainfall, 11 million people lost their homes, worst dam failure
Teton dam	1976	Idaho, USA	14	Due to piping, >1 billion $ property damage
Machchu-2 Dam	1979	Morbi, India	5000	Heavy rain and flooding beyond spillway capacity
Kantale Dam	1986	Kantale, Sri-Lanka	180	Poor maintenance, leakage and consequent failure
Koshi Barrage	2008	Koshi Zone, Nepal	250	Heavy rain, the flood affected over 2.3 million people
Situ Gintung Dam	2009	Tangerang, Indonesia	98	Poor maintenance and heavy monsoon rain
Germano Mine Tailings Dam	2015	Minas Gerais, Brazil	13	One village destroyed, 600 evacuated, 19 miss, Iron west sludge polluted Doce river
Patel Dam	2018	Solai, Kenya	47	Failed after several days of heavy rain
Xe-Pian Xe-Namnoy Dam	2018	Attapeu Province, Laos	36	Saddle dam collapsed during rainstorm, 6600 people homeless, 98 missing.
Brumadinho dam disaster	2019	Brumadinho, Minas Gerais, Brazil	217	Tailings dam suffered a catastrophic failure releasing $12x10^6$ m^3 of tailings slurry, 87 people missing

2.1.2 Historical Development of the Dam Breach Analysis

In order to reduce the potential damages owing to the dam breaches, several hydrologic and hydraulic modeling tools have been developed to simulate downstream water levels in response to a dam breach (Hoogestraat, 2011). Back to the history, the dam break study was started in France in 1850s. Thus, from 1850s to1950s there were no computers so the dam break study was focused on finding theoretical solutions and physical model test (You et al., 2012; Joshi, 2017).

However, since 1960s there have been numerous developments of physically-based and numerical dam breach models. For instance, the first breach model was proposed by Cristofano in 1965 (FEMA, 2013). From 1950s to 1990s the study was focused on the factors that responsible for dam break and flooding disaster at downstream of dam. From the period of 1990s to present, the study is focused on dam safety analysis with advanced technology from simple to very complex simulation 1D/2D model that can be automated by high processor computers.

The modern dam-safety analysis has been an evolving science since the failure of Teton Dam in Idaho, USA in 1976, there has been a significant evolution in the understanding of floods, dams and other critical infrastructures on which modern society & people's wellbeing rely (Joshi, 2017). In In 1977, the United State National Weather Service (NWS) has been developed the first DAMBRK model to analyze the dam breach process and route peak breach outflows to determine inundation depths downstream of the dam (FEMA, 2013). After this period to the mid-1990s, a series of regression equations were developed for the prediction dam breach parameters and peak discharge estimation from breached embankment dams (FEMA, 2013).

The statistical regression analyses such as MacDonald and Langridge-Monopolis (1984), USBR (1988), Von Thun and Gillette (1990), Dewey and Gillette (1993), and Froehlich (1995, 2008) were also developed for use with empirical methods (FEMA, 2013). Since this time, breach modeling process was further advanced with its prediction capability for the reservoir outflow hydrograph and for routing of hydrograph downstream with the integration of physical models and empirical equations. In July 1995 the U.S. Army Corps of Engineers was released the first version of HEC-RAS which designed by Mr. Gary W. Brunner and its updated every times. The current version of HEC-RAS have capable of performing one-dimensional steady flow hydraulics; one and two-dimensional unsteady flow river hydraulics and some others calculations (Brunner et al., 2016). There are so many studies have been carried out using HEC-RAS model for dam breach analysis. The present study can be also conducted with this model in combination with HEC-GeoRAS. According to the literature review by Kumar et al. (2017), unsteady flow simulation provides more reliable results in dam breach study.

Integration of HEC-RAS hydraulic modelling with geographic information systems (Abdelbasset et al., 2015) also provides the possibilities to automatically prepare the geometric data and to import it into the hydraulic model, like HEC-GeoRAS. In addition, data generated by the hydraulic model can be also exported back to the GIS to produce inundation maps and to perform further analysis (Derdous et al., 2015). However, recently released version HEC-RAS package have capability to produce visualization of inundation mapping using RAS-Mapper

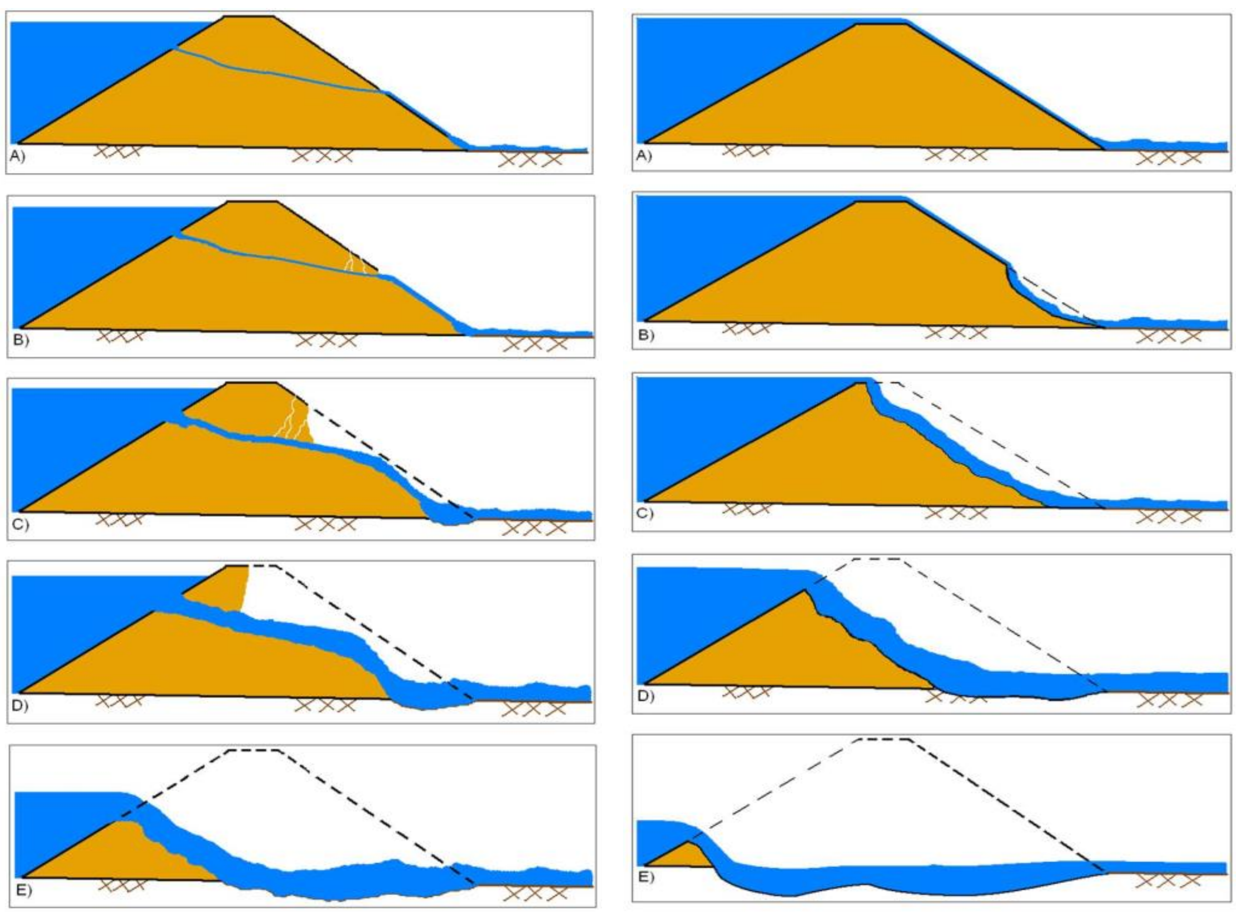
2.2 Embankment Dam Breach Scenarios

Several studies have been carried out over the worldwide in order to understand the main causes of embankment dam failures. The studies shown that the embankment dam's failures can be mainly occurs due to piping and overtopping failure and also sometimes because of structural failure (especially foundation defect). In dam breach studies the embankment dam breach scenarios are referred to two failure mechanisms called a "*sunny day (*fair weather*)*" and "*rainy day (*hydrologic failure*)*" failure conditions (FEMA, 2013).

2.2.1 Fair Weather (Sunny Day)

A fair weather breach is occurs under normal flow or dry weather (non-hydrologic) conditions where the water released causes the largest change in flows. The reservoir volume considered is as at its normal operating level. Sunny day breaches are generally associated with internal erosion (piping) of the embankment, embankment instability or potential damage following a major seismic event (Hall et al., 2016). Hence, fair weather breach is typically used to model piping failure modes for the embankment dam. This condition is examined by establishing an initial reservoir water level and starting a breach analysis without additional inflow from a storm event. Piping failures occurs when there is concentrated seepage that developed within embankment dam. When the water naturally seeping through the dam core, increases in velocity and quantity it could begin to erode the fine sediments out of the soil matrix.

Piping can also generate along conduits, outlet works, and abutments. If enough material eroded, then a direct piping connection may be established from the reservoir water to the dam face and initiate the failure of the dam by piping failure (FEMA, 2013). As the voids become larger and larger due to piping, erosion becomes more rapid and once the erosion reaches the reservoir, it can enlarge and cause catastrophic dam failure. The movement of water through the dam during such piping process is modeled as a pressurized orifice type flow. Typically, piping failure begins near the downstream toe of the dam and works its way toward the upper reservoir. Figure 1 (i) shown conceptually the stages of embankment dam breach condition during piping failure. The Teton Dam failure occurred in 1976, in USA is a famous example of a piping failure.



(i) Piping Failure (ii) Overtopping Failure

Figure 1: Stages of development of embankment dam breach (Source: USACE, 2014)

2.2.2 Hydrologic Failure (Rainy Day)

Hydrologic failure breach scenario of the embankment dam is a type of breach that superimposed upon extreme precipitation and runoff conditions where water released causes the largest incremental increase in flood damage. For such failure scenario the reservoir volume considered is at the top of the embankment, which can be significantly more than the volume at the normal operating level (Hall et al., 2016). Generally, rainy day breaches failure scenario is usually associated with an overtopping failure which may result from inadequate spillway and reservoir capacity to handle the resulting flooding event.

Overtopping failure occurs when the water surface elevation in the reservoir exceeds the height of the dam; water can then flow over the top crest of the dam, an abutment. A failure may also occur when a reservoir's outlet system is not functioning properly, thereby raising the water surface elevation of the dam. Overtopping of a dam as a result of flooding is the most common failure mode for embankment dams (FEMA, 2013).

Generally, overtopping failure of an embankment dam head cut erosion process will first starts on the downstream side of the dam and progresses upstream. The breach is considered to begin when erosion occurs across the width of the dam crest. While water is going over the dam crest, the dam crest acts like a broad-crested weir. The head cut begins to cut into the dam crest, the weir crest length will become shorter, and the appropriate weir coefficient will trend towards a sharp crested weir. The time for breach initiation that used in HEC-RAS model is shortly after what showed in figure 1 (ii) C (Brunner, 2014). When the head cut reaches the upstream side of the dam crest, a mass failure of the upstream crest may occur, and the hydraulic control section will act very much like a sharp-crested weir.

Figure 1 (ii) shown conceptual stages of embankment dam breach condition during overtopping failure. After the breach initiates at the top of the dam crest, it enlarges to its ultimate extent (Central Water Commission, 2018). The head cut will also continue to erode upstream side through the dam embankment, as well as erode down at the same time. According to most of literature review, majority of embankment dam failure was occurred due to overtopping failure. The Chaq-Chaq dam failure in 2006 is one of this type of failure example (Abdulrahman, 2014).

2.3 Dam Breach Analysis

Dam breach analysis is the process of studying a dam failure phenomenon and analyzing the resulting consequences at the downstream of the dam (Pandya et al., 2013). This generally deals with simulation of assumed failure modes using physical based, semi-physical based and non-physical based approaches for existing dams and analyzing the resulting consequences.

Dam breach analysis is usually conducted to determine the ultimate discharge from a hypothetical breach of a dam under different condition. Mainly, the outcomes from dam breach analysis is a breach hydrograph from dam failure with a flood wave immediately downstream of the dam, which is routed throughout the downstream river system (Asnaashari et al., 2014). This is used to determine the flood arrival time, peak flow, and the depth of flow at downstream locations.

In the context of risk informed decision making approach, dam breach analyses are needed for determining the potential consequences of a failure occurrence over a range of loading conditions. Dam breach analysis can also be used as part of a dam's remedial design process in the selection of alternatives options. The type of analysis as well as the level of accuracy expected can be determined the potential hazards and complexity of the downstream area being modeled. Depend on the dam hazard classification details of data required may be vary. For the conditions of highly hazarded classified area; more detail data will be needed to produce more accurate results.

According to FEMA (2013), dam breach analysis are used for multipurpose functions as listed out here below:

- ✓ Evaluating and establishing the hazard potential classification for a dam
- ✓ Estimating the potential for loss of life
- ✓ Evaluating dam safety risk and prioritizing dams within a dam safety portfolio
- ✓ Selecting the appropriate SDF or IDF for dam and spillway design
- ✓ Developing EAPs and exercise planning associated with dam safety permitting
- ✓ Developing breach inundation zone mapping for flood early warning systems, for flood evacuation planning, for mitigation plan and for risk communication d/s of the dams.

There are different approaches available for dam breach analysis. The most widely used dam breach analysis approaches are an event-based approach and risk-based approach. Details of each approach are discussed in consecutive sections.

2.3.1 Event-based Approach

The event-based approach has been traditionally the most widely used for the dam breach analysis. It is a deterministic method that based on specific rainfall events for the dam breach analysis and downstream inundation mapping. These events may include extreme rainfall and runoff events, which can be lead to natural floods of significant magnitude (Central Water Commission, 2018). For an event-based approach, both a non-hydrologic (fair weather failure/sunny day failure), and a specific hydrologic failure event, such as the probable maximum flood, are usually established based on a dam's downstream hazard potential classification (FEMA, 2013). Maximum flood for which a dam is to be designed for or to be evaluated is usually dependent on its existing hazard potential or size classification. Several hydrologic and non-hydrologic events are typically evaluated as part of an event-based dam safety analysis.

According to FEMA (2013), for hydrologic failure conditions, the magnitude of 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 the failure of the dam or for significant economic and environmental losses. Hazard potential classification of the dam is typically used to select the extreme hydrologic failure event. For a fair weather conditions, base flow is typically ignored because of the small discharge and volume as compared to that of a dam breach or the dam breach flow is two times greater than the base flow.

There are three most common initial water level elevations for fair weather breach analyses named normal pool elevation, invert of auxiliary spillway and top of the dam (FEMA, 2013). The advantage to using an event-based dam breach approach is that it is less complicated to perform and regulate, it is a direct approach, and the produces are more conservative breach inundation zone mapping when compared to a risk-based approach (FEMA, 2013). The hydrologic failures that cause dam breach are generally analyzed based on the IDF established by dam's hazard classification. The ranges of initial reservoir pool levels for a fair weather (non-hydrologic) analysis are summarized in table.

2.3.2 Risk-based Approach

Risk-based approach is sometimes also called consequence-based approach. FEMA (2013) indicate that risk-based approach is commonly used for dam design purposes to establish the SDF or IDF for a dam. For a risk-based approach, the downstream consequences for a range of hydrologic dam failure events are evaluated (FEMA, 2013). It sometimes occurs that two or more dams are constructed on a watercourse and the failure of an upstream dam may cause a sequential failure of a downstream dam. 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.

The benefit of the risk-based approach is that it may demonstrate, through an incremental damage assessment, that areas located downstream of a dam may be affected only marginally due to a dam breach, following a reduction in the IDF for a dam (FEMA, 2013). However, disadvantage of the risk-based approach is that by reducing the IDF to lesser magnitudes based on downstream consequences, the impact of future developments in the downstream of a dam is set aside from the present considerations. New development in the downstream zone that comes under inundation due to a dam breach may alter the consequences. This may result in the need for dam rehabilitation measures in the future to meet the demand of increasing the spillway capacity (Central Water Commission, 2018).

Inflow Design Flood and the Incremental Hazard Evaluation

FEMA (2013) define IDF 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.” 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. 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 (FEMA, 2013). The new FEMA guidance IDF for dam is shown in Table 2.

Table 2: Recommended IDF for Dams Using Prescriptive Approach (Source: FEMA, 2013)

Hazard Potential Classification	Definition of Hazard potential classification	Inflow Design Flood
High	Probable loss of life due to failure or miss-operation	PMF
Significant	No loss of human life but cause economic losses, environmental damage or disruption of lifeline facilities due to dam failure or miss-operation	0.1 % annual chance exceedance flood (1000-yr flood)
Low	No probable loss of human life and/or environmental losses due to dam failure or miss-operation	1 % annual chance exceedance flood (100-yr flood)

Loss of life / Population at Risk

Determination of probable loss of life is an important factor used in hazard potential classification systems and emergency action planning. Publication of (USBR, 1999) presents a risk-based method to estimate the number of fatalities that would result from dam failure. This method was developed using data from about 40 floods, many of which were caused by dam failure. These publications outline the following seven steps to complete an analysis for loss of life (Central Water Commission, 2018):

Step 1: Determine dam failure scenarios to evaluate

Step 2: Determine time categories for which loss of life estimates are needed

Step 3: Determine when dam failure warnings would be initiated

Step 4: Determine area flooded for each dam failure scenario

Step 5: Estimate the number of people at risk for each failure scenario and time category

Step 6: Apply empirically based equations or methods for estimating fatalities

Step 7: Evaluate uncertainty

The number of fatalities resulting from dam failure is most influenced by three factors: 1) the number of people occupying the dam failure floodplain, 2) the amount of warning provided to the people exposed to dangerous flooding, and 3) the severity of the flooding. Without exception, dam failures that have caused high fatality rates were those in which residences were destroyed and timely dam failure warnings were not issued (Central Water Commission, 2018).

For each failure scenario and time category, the population at risk must be calculated. Population at risk is defined as the number of people occupying the dam failure floodplain prior to the issuance of any warning. The method developed for estimating loss of life provides recommended fatality rates based on the flood severity, amount of warning time, and a measure of whether people understand the severity of the flooding (Central Water Commission, 2018). In generally dam breach is needs to be conducted for permanent solution to rescue downstream life and properties.

2.3.3 Tiered dam Breach Analysis

A tiered study approach was developed by the USDOI. This approach can be used to establish an initial dam hazard potential classification and to produce dam breach inundation zone mapping for EAPs. However, the tiered dam breach analysis structure is not appropriate for use in dam design (FEMA, 2013). The tiered approach is also 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. As the sophistication of the modeling increases, so does the level of effort, time, and cost necessary to conduct the analysis. Table 3 provides the guidance to determine the tier level for analysis for dam failure inundation modeling and mapping. This table is arranged similarly to some United State-developed tiered analysis structures, providing a logical combination of methods to perform an analysis. 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/reservoir with the capacity to store the flood waters (FEMA, 2013).

Table 3: Tiered Approach Dam Breach Inundation Mapping for used in EAPs (Source: FEMA, 2013)

Tier Level	Applicable to	Breach Parameter Prediction	Peak Breach Discharge Prediction	Downstream Routing of Breach Hydrograph
Tier1-Basic Level Screening and Sample analysis	✓ Low-hazard potential small size ✓ First level screening for significant- or high hazard dams	Empirical Equations	Simplified Models (SMPDBK, GeoDam-BREACH, or [TR]-66) or HECHMS	GeoDam-BREACH, SMPDBK, DSAT,1D HEC-RAS Steady State, or HEC-HMS Hydrologic Routing
Tier 2 – Intermediate	✓ Significant-hazard potential / intermediate size ✓ High-hazard dams with limited population at risk	Empirical Equations	HEC-HMS or HECRAS Unsteady Model	HEC-RAS (Steady or Unsteady Modeling) 1- D or 2-D models
Tier 3 – Advanced	✓ High-hazard potential / large size dams with Sufficient population at risk to justify advanced analyses	Empirical Equations, NWS BREACH, or WinDAM	HEC-RAS Unsteady Model	HEC-RAS Unsteady Model or 2-D models

FEMA (2013), recommend that 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, such as major diversion structures, split flows, or potential for a series of dam failures.

2.4 Dam Breach Parameters

Dam breach parameters are the key elements that are used for dam breach modeling concerned to the geometry (width, depth, shape) and time of failure of the breach formation (FEMA, 2013). Breach development and the resultant geometry have a fundamental control on the outflow hydrograph and the downstream movement of the flood wave (Wahl, 1998). However, breach simulation and breach parameter prediction contain the greatest uncertainty of all aspects of dam break analysis (Sidek, Ros, & Ahmad, 2011). Martin & Akkerman (2017), indicate that, defining breach parameters are challenging procedure.

The following definitions are the most commonly accepted for the evaluation and selection of dam breach parameters (FEMA, 2013). A figure 2 shows dimension of a dam breach corresponds to definitions here below.

- ✓ **Breach formation time (time-to-failure):** The duration of time between the first breaching of the upstream face of the dam (breach initiation) and when the breach has reached it full geometry.
- ✓ **Breach depth (breach height):** The breach depth is the vertical extent of the breach measured from a specific elevation to the invert of the dam breach.
- ✓ **Breach width:** The breach width is the average of the final breach width, typically measured at the vertical center of the breach.
- ✓ **Breach side slope factor:** The breach side slope is a measure of the angle of the breach sides represented as X horizontal to 1 vertical (XH: 1V).

The breach width is described as an average breach width (B_{ave}) in most equations, while HEC-RAS requires the breach bottom width (W_b) for input. The breach dimensions, as well as the breach formation time must be estimated outside of the HEC-RAS software and entered into the program. Many case studies have been performed on data from historic dam failures, leading to guidelines, regression equations, and computer modeling methodologies for the dam breach size and time (Brunner, 2014).

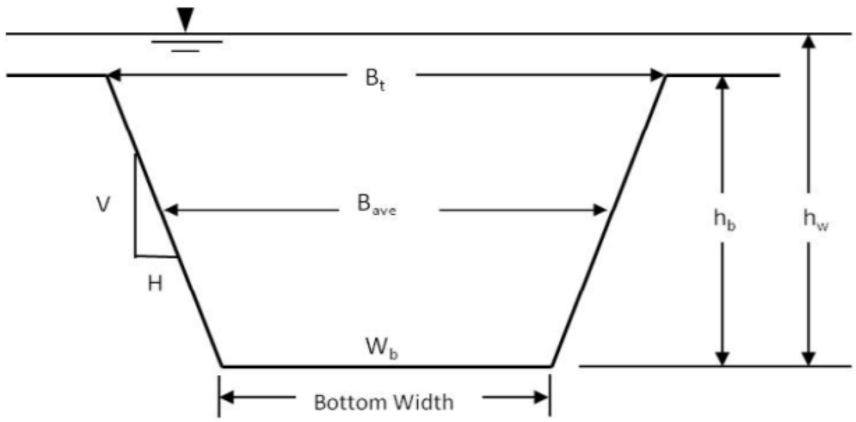


Figure 2: Schematic description of the dam breach parameters

where:

B_{ave} = average breach width, W_b = breach bottom width, h_b = breach height, and h_w = height of water

A dam breach usually occurs in two distinct phases starting with the breach initiation followed by the breach formation (FEMA, 2013).

- ✓ **Breach initiation:** During the breach initiation phase, flow through the dam is minor and the dam is not considered to have failed. It may be possible to prevent a dam breach during this phase if flow is controlled.
- ✓ **Breach formation:** Breach formation (defined above) begins when the flow through the dam has increased and progressed from the upstream face to the downstream face of the dam, is uncontrolled, and will result in the failure of the dam

More factors must be considered in selecting appropriate breach parameters including dam type, dam dimensions, and dam materials of construction and also other pertinent information such as historical records of seepage or foundation problems should also be considered (FEMA, 2013). Abdulrahman (2014) state that, the breach formation time is the most sensitive parameter in developing a hydraulic model for dam break problems and breach hydrograph development.

The simulation of the dam breach development and resulting outflow hydrograph in HEC-RAS requires parameters that define the ultimate geometry of the breach and the rate at which it formed in each breaching scenario overtopping or piping failure (Deo & Muchard, n.d.). Reasonable values for the breach size and development time are needed to make a reliable estimate of the outflow hydrographs and resulting downstream inundation, flood travel times, water velocities, etc. These parameters describing the breach have large uncertainty (Gee, 2008).

The estimation of dam breach location, dimensions and development time are significant in any assessment of a dam's potential risk. This is especially reliable in a risk assessment where dams will be ranked based on the potential for loss of life and property damage. The breach parameters will directly affect the estimation of the peak discharge coming out of the dam, as well as any possible warning time available to downstream locations. There are a number of methods available for estimating breach parameters for use in dam breach studies. Some of the most widely used methods are described in the next section.

2.5 Dam Breach Modelling

The breach modeling is basically refers to the process of understanding and simulating the breach development in a dam (Pandya et al., 2013). The critical elements involved in breach modeling are the estimation breach parameter (breach size/shape and time of failure), breach peak discharge and breach hydrograph estimation, breach flood routing, and the estimation of the hydraulic conditions at critical locations (Colorado, 2010).

A number of modeling tools has been developed to perform dam breach modeling, ranging from simple to complex models. With advancements in GIS-based modeling, many models can interface with digital terrain data to produce automated dam breach inundation zone delineations (FEMA, 2013). The most commonly used approaches for the dam breach parameters estimations are classified into three as non-physical, semi-physical, and physical models. These methods are briefly described in consequent sections.

2.5.1 Non-physical Based (Empirical) Methods

Empirical methods are used to predict time to failure and breach geometry, as well as to predict peak breach discharges. The empirical approach relies on statistical analysis of data obtained from documented failures. The most widely used and accepted empirically derived enveloping curves and/or equations for predicting breach parameters are: MacDonald and Langridge-Monopolis (1984), USBR (1988), Von Thun and Gillette (1990), and Froehlich (1995a, 1995b, 2008). These methods have reasonably good correlation when comparing predicted values to actual observed values (Colorado, 2010). The most basic statistical process generally involves plotting data for variables extracted from the dam failure are dataset such as volume of embankment removed, volume of water released, height of water behind the dam, and development time of the failure (Colorado, 2010).

A) Froehlich (2008):

Froehlich (1995a) utilized 63 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rock fill data sets to develop a set of equations to estimate average breach width, side slopes, and failure time (Brunner, 2014).

Froehlich (2008) updated the breach equation based on addition of new data by utilizing 74 data sets. In the application of these equations reported herein, the height of the breach is calculated by assuming that the breach goes from the top of the dam to the natural ground elevation at the centreline of the breach location. Froehlich method is more preferable since it is only dependent on volume of the reservoir, height of the breach and assumed side slope. The method also distinguishes between piping and overtopping failures using a variable coefficient termed the Failure Mode Factor, Ko. (Brunner, 2014). The data that Froehlich used for his regression analysis have the following ranges.

Height of the dams: 3.05 -92.96 meters,

Volume of water at breach time: 0.0139 – 660.0 $m^3 \times 10^6$

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

$$B_{ave} = 0.27\, K_o\, V_w^{0.32}\, h_b^{0.04} \quad \text{......Equation 1}$$

$$tf = 63.2\sqrt{\frac{V_w}{g h_b^2}} \quad \text{......Equation 2}$$

where: B_{ave}= average breach width (m)

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

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

h_b = height of the final breach (m)

g = gravitational acceleration (9.80665 m/sec^2)

t_f = breach formation time (seconds)

Froehlich's 2008 paper states that the average side slopes should be: 1H: 1V for overtopping failures and 0.7H: 1V for piping/seepage. 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; $Q = 0.607V_w^{0.295} h_w^{1.24}$ ……....Equation 3

B) MacDonald and Langridge-Monopolis method (1984):

MacDonald and Langridge Monopolis utilized 42 data sets of embankment dam to develop a relationship called "Breach Formation Factor (BFF)". BFF is the product of the volume of water coming out of the dam and the height of water above the dam (Brunner, 2014). They estimated the quantity of eroded embankment materials V_{eroded} (m^3) for earth and rock dams as:

$V_{eroded} = 0.0261(V_{out}*h_w)^{0.769}$: for earth-fill ………………………….Equation 4

$V_{eroded} = 0.00348(V_{out}*h_w)^{0.852}$: for rock-fill ……..……………………Equation 5

$tf = 0.0179(V_{eroded})^{0.364}$

$$W_b = \frac{V_{eroded} - h_b^2(CZ_b + h_b Z_b.\frac{Z_3}{3})}{h_b(c + h_b.\frac{Z_3}{2})}$$ ……………………………..…….Equation 6

$Q = 3.85(V_d h_d)^{0.411}$ ……………………...………………………...….Equation 7

where:

V_w = volume of water discharged through breach (m^3)

V_d = volume of water above breach invert at time of failure (m^3)

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

V_{out}= Volume of water that passes through the breach (m^3)

tf = breach formation time (hours)

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

h_d= depth of water above the breach invert at time of breach (m)

h_b = height from the top of the dam to bottom of breach (m)

W_b = bottom width of the breach (m)

C = Crest width of the top of the dam (m)

$Z_3 = Z_1+Z_2$

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

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

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

C) Von Thun and Gillette (1990):

They have used 57 dams from both the Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) to develop their methodology. The method is proposed to use breach side slopes of 1.0H: 1.0V, except for a dam constructed from cohesive soils which should be in the range of 0.5H: 1V to 0.33H: 1V, (Brunner, 2014). The equations developed by Von Thun and Gillette are given below as:

B_{ave} (m) $=2.5h_w+C_b$ ……………………………………………….Equation 8

tf (hr) = $0.015*h_w$ for highly erodible dam ………………………….Equation 9

tf (hr) = $0.0209*h_w + 0.25$, for erosion resistant dam ……………….Equation 10

where: B_{ave} = average breach width (m)

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

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

Table 4: Function of reservoir size (C_b) values

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

Peak Flow Equations and Envelope Curve

Several scholars have developed peak flow regression equations from historic dam failure data and developed envelope curve. Once a breach hydrograph is computed in HEC-RAS, the computed peak flow from the physical model can be compared to these regression equations as a test for reasonableness. However, one should use great caution when comparing results from these equations to model predictions (Brunner, 2014). Some of peak flow equations summary from historic dam failures are given below in metric form.

- ✓ USBR (1982): $Q = 19.1\,(h_w)^{1.85}$ (envelope equation) ………….…Equation 11
- ✓ MacDonald and Langridge-Monopolis (1984):

  $Q = 1.154\,(V_w h_w)^{0.412}$ ………………………................…Equation 12

  $Q = 3.85\,(V_w h_w)^{0.411}$ (envelope equation) ……………….…Equation 13
- ✓ Froehlich (1995b): $Q = 0.607 V_w^{0.295} h_w^{1.24}$

where: Q = peak breach outflow (cubic meters per second)

h_w = depth of water above the breach invert at time of breach (meters)

V_w = volume of water above the breach invert at time of failure (meters)

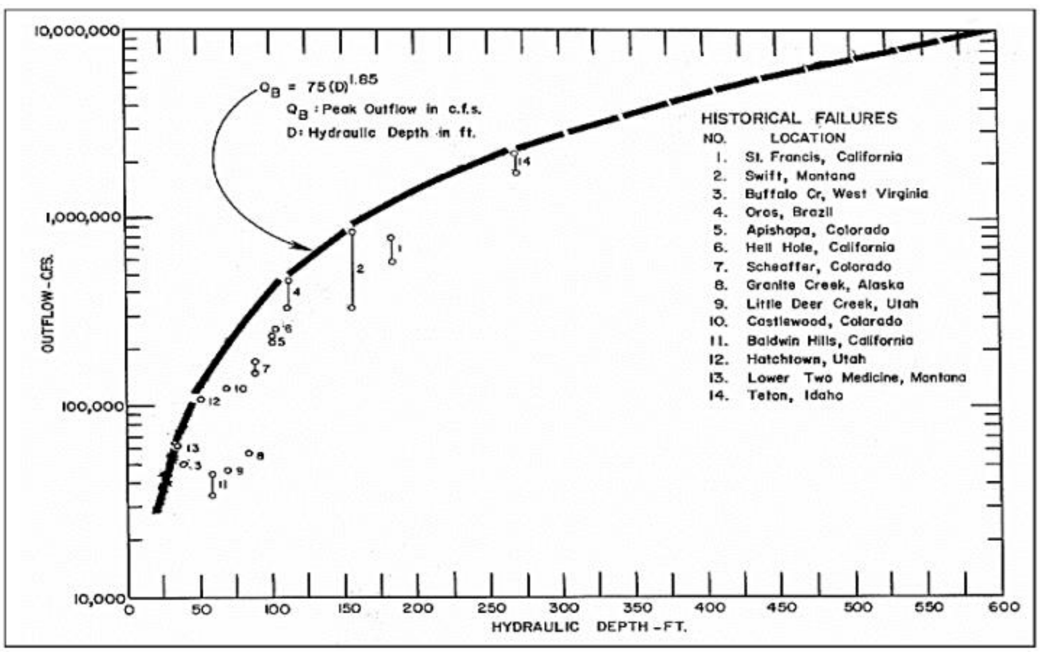


Figure 3: Envelope Curve based on 14 dams breached (Source: USACE, 2014)

2.5.2 Semi-physical Based (Parametric) Models

These models are developed based on simplified assumptions of physical the processes with an intention to reduce the large uncertainties associated within non-physically based models and to simplify the complex computational procedure in physically based models. These models require the user to provide information on 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. These models may provide a more accurate flood hydrograph as compared to empirical equations; however, they are vulnerable to large degree of uncertainty due to the input data set. Some examples of the parametric computer models that can estimate the peak discharge and breach hydrographs from dam breaches parameters (breach geometry and breach development time) are HEC-1, HEC-HMS and HEC-RAS. They can also use to calculate flood routing of the hydrograph downstream, and, in the case of HEC-RAS, can be used to estimate the hydraulic conditions at critical downstream locations (Colorado, 2010). The description of parametric model for hydrology and hydraulics are given here below.

I) Hydrologic Models

The hydrologic routing can employs the continuity equation and empirical or analytical relationship between storage within the reach and discharge at the end. In the absence of significant backwater effects, the hydrologic routing models can offer the advantages of simplicity, ease of computational efficiency (Colorado, 2010). Hydrologic routing models provide attenuated flow hydrographs at locations of interest, but do not provide useful information on water surface elevations or flow velocities. HEC-1 and HEC-HMS are the most widely used hydrologic models for dam safety analysis, and both contain a parametric dam breach routine that calculates the breach hydrograph (Colorado, 2010).

II) Hydraulic Models

Hydraulic models, in general, are more physically based than hydrologic models since they only have one parameter (the roughness coefficient) to calibrate. Full unsteady flow equations have the capability to simulate the widest range of flow situations and channel characteristics. The basic data requirements for hydraulic routing techniques include; flow data, channel geometry, roughness coefficients, and internal boundary conditions (Colorado, 2010). There are two conditions in hydraulic modeling called steady and unsteady flow analysis. In unsteady flow, time dependent changes in flow rate are analyzed explicitly as a variable, while steady flow analysis models neglect time all together. Steady flow analysis can determine a water surface elevation and flow velocity at a given cross section for a given flow using Manning's equation under the assumption of gradually varied flow conditions (Colorado, 2010). Unsteady flow analysis can be used to evaluate the downstream attenuation of the flood wave, providing a more accurate estimate of flood magnitude and velocity at critical locations.

2.5.3 Physical Based Models

Physical models 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. A physically-based model also referred as process model utilizes the generally accepted relationships based on physical principles to establish the framework of a model (Colorado, 2010). Some of these models are BREACH, BRDAM, BEED, FLOW SIM1, FLOW SIM2, etc.

The benefits of physically based models are; the modelling 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 and uncertainties within individual processes or parameters may be included within the model. While, their disadvantages are; they required very high computing power device/computer and the model runtimes become quite long as the simulation of processes becomes more complex.

Since 1960s, there have been numerous developments of physically-based, numerical dam breach models. Currently, the NWS BREACH model is a well-known and commonly applied physically based model developed by a Federal agency of United State. The NWS BREACH model was developed to be more realistically simulate breaches initiated by overtopping or piping in an embankment dam (FEMA, 2013).

The BREACH dam breach model predicts the development of a breach and the resulting outflow hydrograph using an erosion model based on principles of hydraulics, sediment transport and soil mechanics (FEMA, 2013). Physical models are relatively simple concept, but it can be very complex when the input is changing with time. In the case of dam breach analysis, both the input and physical constraints are changing with time as the dam erodes and the reservoir evacuates, which it means that become complex.

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 those methods are comparative analysis and experimental method.

- **Comparative Analysis:**
  In case 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 analysis approach. However, according to (Wahl, 1998), this method is highly not reliable for large dams because of lack of accurate and comprehensive case study data on large dam.
- **Experimental method:**
  To better understand the processes involved in 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 in every time.

2.6 Dam Breach Modelling with HEC-RAS and HEC-GeoRAS

A HEC-RAS modeling tool has been developed by the Hydrologic Engineering Centre of the US Army Corps of Engineers and the first version was released in 1995. Currently, the program has capable of performing 1D steady flow and 2D steady and unsteady flow computation, movable-boundary flow for sediment transport analysis, and water quality analysis. So many studies have been conducted on dam breach analysis using HEC-RAS model since the past decades. The present study will be also conducted with this model with the combination of HEC-GeoRAS.

The review by Kumar et al. (2017) shown that HEC-RAS unsteady state simulation have been perform well and the application of this model can save calibration time and eliminate the requirement to run a second model to determine corresponding flood stage simulation.

From their literature review, they conclude that HEC-RAS is efficient tool which provides more reliable results in dam breach study and the results obtained from the simulation model can be used to prepare the inundation mapping in GIS platform through HEC-GeoRAS tool.

Unsteady flow component of the HEC-RAS modeling simulates one-dimensional unsteady flow and can perform subcritical, supercritical or mixed flow regime computations. HEC-RAS one-dimensional unsteady flow hydraulic model has a capability to of simulating highly dynamic flood events such as a dam breach (Asburry, 2009). For two dimensional hydraulic models, are based on integration over the flow depth to obtain depth averaged velocity values and are solved using an appropriate numerical approach such as a finite element model (Pandya et al., 2013).

The governing equations for unsteady flow are the momentum equations derived from the full equations of motion (St. Venant equations) and conservation of mass (continuity). Upstream boundary conditions are typically consisting of an inflow hydrograph from the upstream watershed into a defined reservoir. For the case of dam breach analysis, the reservoir outflow is dynamically routed downstream. Dynamic wave (unsteady flow) routing is the most accurate modelling technique to capture pool elevations and outflows of long narrow reservoirs (Brunner, 2014).

The overtopping and piping failure modes are include into the HEC-RAS model. In HEC-RAS, overtopping failures start at the top of the dam while a piping failure can start at a specified location and grow up to the maximum specified extents. Dam breach parameters, such as breach width, depth, side slopes, and development time are estimated external to the model. The values for the breach size and development time are needed to produce a reliable estimate of the outflow hydrographs and resulting downstream inundation areas (FEMA, 2013).

HEC-RAS model has capable of perform inundation mapping of water surface profile results directly using the RAS Mapper or the external HEC-GeoRAS extension on ArcGIS tool. Using the HEC-RAS geometry and computed water surface profiles, RAS Mapper creates an inundation depth and floodplain boundary dataset. HEC-GeoRAS is an extension of GIS tools that prepare the geometric date for import into HEC-RAS and generate the flood inundation data from the HEC-RAS output. Through the combination of the GIS extension, HEC-GeoRAS and HEC-RAS, one can use an available digital elevation model to build the geometric file for a

HECRAS model (Sharma & Mujumdar, 2017). Additionally, the GeoRAS extension has simple import and export capabilities to smooth the transition between creating the geometric file, running the model and displaying the results in GIS platform.

HEC-RAS model can be used to model the reservoir using fully dynamic wave routing. If it is not possible to perform full dynamic wave routing through the reservoir, or if the presumed difference between level pool routing and dynamic routing is small, then level pool routing can be performed with HEC-RAS. Figure 4 and 5 shows reservoir routing mechanisms (Brunner, 2014). This figure indicates the routing mechanisms for level pool and fully dynamic routing. When developing unsteady state flow for dam breach, we should have to consider cross section spacing and hydraulic properties, computational time step, drop in bed profiles in addition to the others common input data types/parameters.

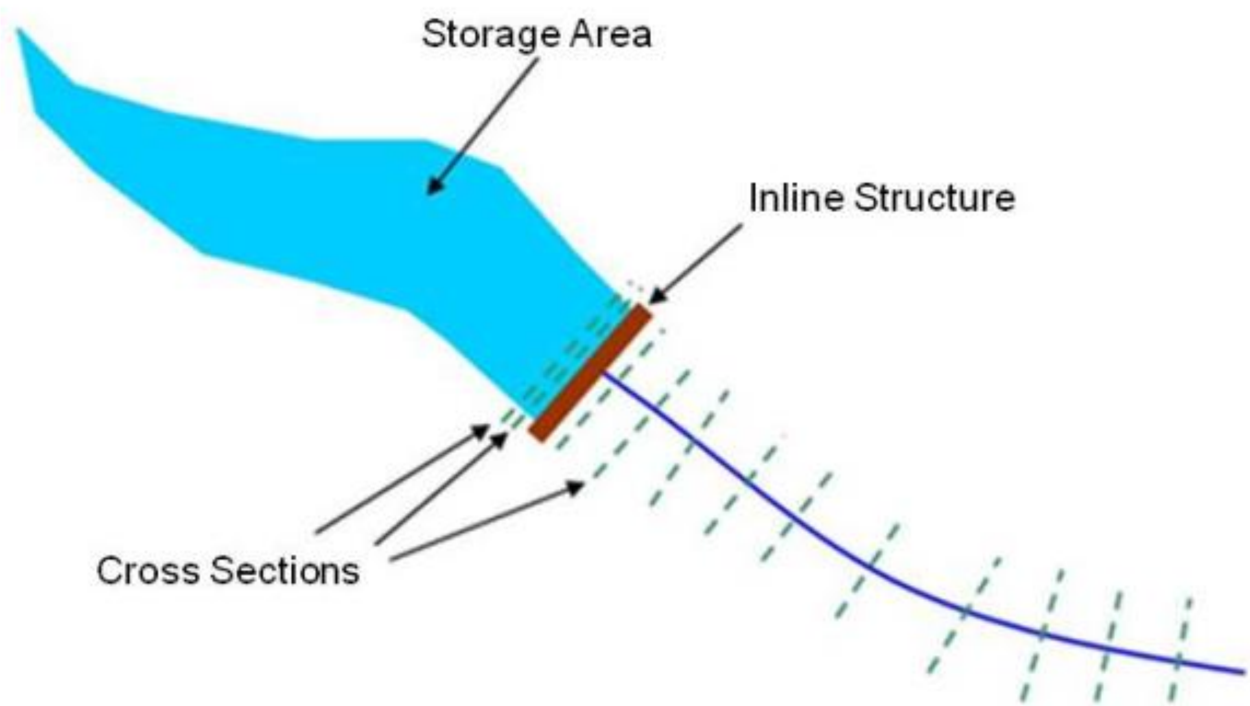


Figure 4: Storage area and cross section layout for level pool routing (Source: USACE, 2014)

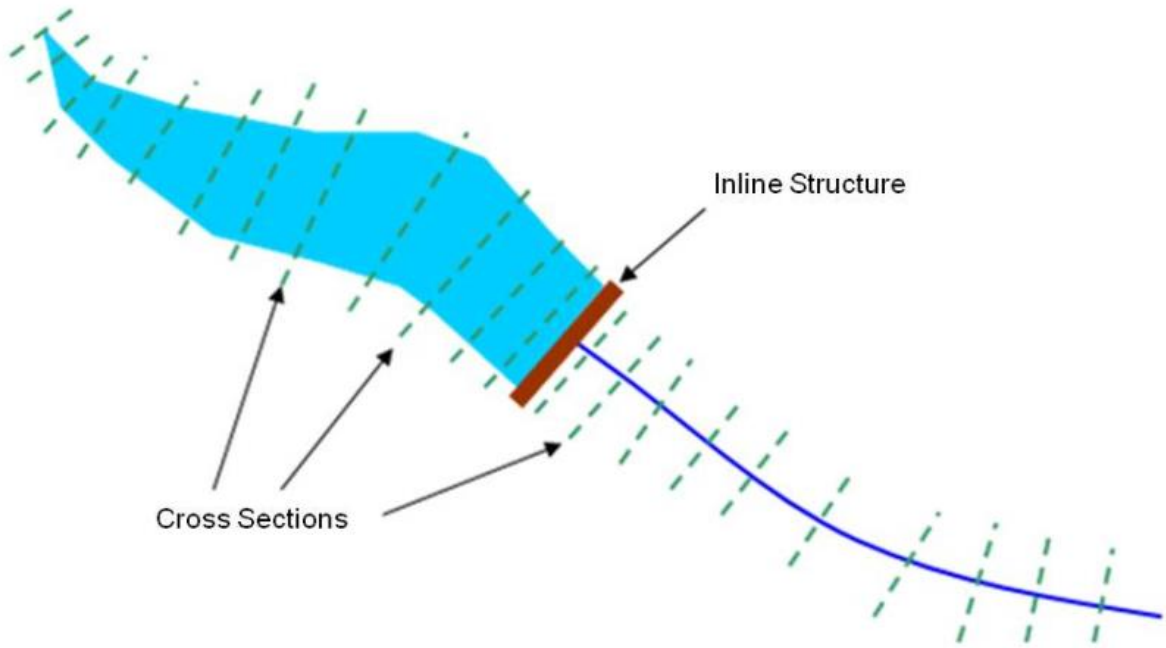


Figure 5: Storage area and cross section layout for one dimensional fully dynamic routing (Source: USACE, 2014)

2.7 Dam Hazard Potential Classification Systems

There are a number of guidelines has been developed by different agencies for dam hazard potential classifications. For instance, it is developed by FERC, FEMA, USBR, USACE, NRCS, BIS, etc. An organization can develop their own classification systems based on own interest. However, according to the literature review the dam hazard potential classification developed by FEMA and USACE are the most widely used classification systems.

The dam hazard classification system shown on Table: 5 were developed in 1997 by USACE for civil works project. The categories are based upon project performance. The direct loss of life is based upon potential for loss of life on inundation mapping of the area downstream of the project. The lifeline losses are based upon indirect threats to life caused by the interruption of lifeline services due to project failure, or operation. The property losses is based upon direct economic effect on the value of property damage to project facilities and downstream property and the environmental losses is based up on environmental impact downstream caused by the incremental flood wave produced by the project failure (FEMA, 2013).

Table 5: USACE Dam Hazard Potential Classification System for Civil Works Projects

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

FEMA guidance recommends that the dam hazard potential rating be based on consideration of the effects of a failure or miss-operation during both normal and flood flow conditions. Furtherly, FEMA recommends that the hazard potential should be based on the worst-case probable scenario of failure or miss-operation of the dam (FEMA, 2013). The following classification in table 6 is developed by FEMA in 2004a.

Table 6: FEMA Hazard Potential Classification System

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

- **Low Hazard Dam:** is 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.

- **Significant Hazard Dam**: is for which significant damage is expected to occur, but no loss of human life is expected from failure of the dam. It 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.

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

2.8 Flood Inundation Mapping

Flood inundation maps are tools for visualization of flood information for decision-makers and the public in general. These maps form the basis for developing different flood risk scenarios based on land use, environmental conditions and social and economic conditions (Central Water Commission, 2018). Analyzing the failure of a dam for developing an inundation map can be viewed as a two-step process. First the estimation of the dam breach outflows, and second, routing the breach outflows downstream to determine the resulting flood inundation extents (NCDENR, 2015).

The inundation map provides a description of the areal extent of flooding which would be produced by the dam break flood. It should also identify zones of high velocity flow and depict inundation for representative cross-sections of the channel (Dam Safety Guidelines, 2007). An inundation mapping shows a continuous line of inundation identifying the area potentially at risk in event of dam failure. It starts at the dam and continues downstream to a point where the breach flood no longer poses a risk to life and property damage, such as large river or reservoir with the capacity of storing the flood waters (NCDENR, 2015).

With compared to other types of flood, floods caused by dam failure are characterized by instant occurrence, vast quantity of flow, and powerful impulsive force (Mao et al., 2017). Inundation mapping of regions downstream the dam is needed to better understand the consequence of dam breach outflows for effective emergency managements (Yusuf et al., 2009). Dam breach inundation maps have a variety of advantages. In general, the flood mapping generated from dam breach has a potential uses for the following items (Central Water Commission, 2018):

- **Emergency Action Plans:** EAP is a formal document that identifies potential emergency conditions at a dam and specifies preplanned actions which are required to be followed for minimizing damage to properties and loss of life. For example, development of EAPs for all high and significant hazard potential dams is critical to reducing the risks of loss of life and property damage that may happen due to dam failures.

- **Disaster Response:** this includes the actions, which are to be taken during and in the immediate aftermath of a flood incident to save and sustain lives, meet basic human needs, and reduce the loss of property and the damage to critical infrastructure.

- **Hazard Mitigation Planning:** mitigation is the proactive effort to decrease the loss of life and property by reducing the effect of disasters. This is achieved through identification of potential hazards and the risks they pose in any given areas, and risk analysis of mitigation alternatives. The result is selection of proactive measures, both structural and nonstructural, which will help to reduce economic losses and potential loss of life when implemented.

- **Dam Failure Consequence Assessment:** it includes the identification and quantification of the probable consequences of a dam failure. Hazard mitigation planning focusses on specific projects to reduce flood risk. Consequence assessment focuses on the economic and social impacts of a probable disaster, and the organizational and government actions needed after a dam breach in order to respond and recover.

The factors that influence the estimation of inundation that occurs downstream flood travel time, depth of inundation, velocity of flow and the flood hazard are includes pre-existing flow in the river, channel and overbank characteristic (slope, geometry, and roughness), debris content, change in channel during winter/summer, and topography (Mccann, 2016).

3. DESCRIPTION OF THE STUDY AREA AND DATA SETS

3.1 Location of Study Area

Gidabo Embankment Dam is situated in the Abaya-Chamo sub-basin of the Rift Valley Lakes Basin, on the lower courses of the major rivers of the Gidabo basin in the southern part of Ethiopia. It is within the two administrative regions named Oromia and SNNPR. To be more specific, it falls in Abaya district of Borena zone of Oromia region and Dale district of Sidama zone of SNNPRS, near to Dilla town to east of Lake Abaya. Geographically lies approximately between 6°20' and 6° 25' N Latitude and 38° 05' and 38°10'E Longitude. See Figure: 6 below.

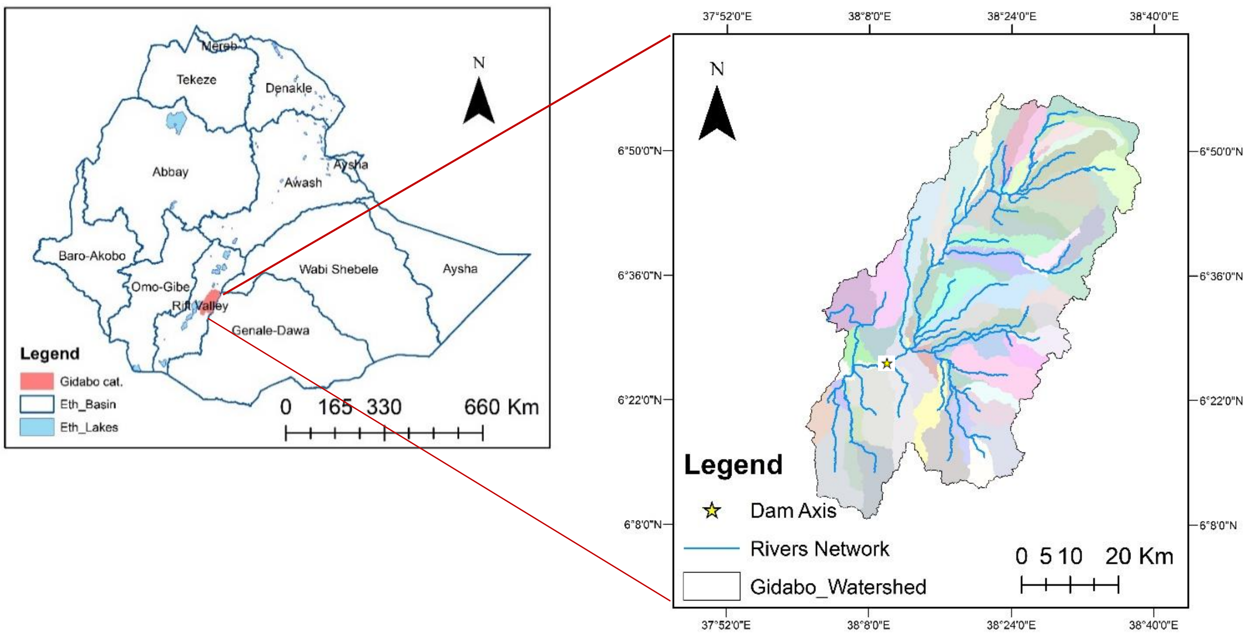


Figure 6: Gidabo Embankment Dam location

Gidabo embankment dam project is intended to use for the irrigation purpose. It is aimed to irrigate 13,425 hectare of command area on both sides of the river course. The dam is constructed as earth-rockfill dam with pertinent structures and impounding 63 million cubic meter of water. The area downstream of the dam is characterized by a flat terrain of relatively the same elevation.

According to the study documents from Federal Water Works Design and Supervision Enterprise, the mean monthly rainfall data in the project area varies from 23 mm in December to 117 mm in April month. Regarding to the land use land cover of the study area, the downstream part of the dam is largely forms of deltaic and swampy lands. Tall savannah grasses are common features of the area. When dried-up, the margins of swampy and marshy lands in this area are used for grazing by pastoralist. The middle part of the catchment is occupied with small and scattered bushes and grasses. The upper part of the catchment is covered with thick bushes and shrubs. The upstream of the dam is highly degraded and easily erodible. Hence, selection of channel side slope and the manning’s roughness coefficient that will be used within the model is demand great attention.

3.2 Data Sets Used

The data sets used within this study are described in subsequent sections. Data collections from all respective body were carried out earlier to this research work. However, the data requires extensive pre-processing steps, mainly the rainfall data. In such case, the rainfall data have been already processed by Ethiopian Construction Design and Supervision Works Corporation (ECDSWC) which formerly called as the Federal Water Works Design and Supervision Enterprise (WWDSE) were used.

3.2.1 Hydrological Data

In HEC-RAS modelling, flow hydrograph can be used as either upstream or downstream boundary condition. Therefore, the requirement of hydrological data is to produce inflow design flood as an input data for boundary condition either as hydrograph or as peak discharge called probable maximum flood (PMF). In order to determine this PMF, a Probable Maximum Precipitation (PMP) is initially calculated using the observed daily rainfall data from stations located within the catchment and stations nearby to the watershed. Then, PMP is transformed into PMF inflow hydrograph using SCS unit hydrograph. For this study, the PMF developed by (WWDSE, 2008) was used. The maximum peak flood is about 3507.82 m^3/s, (see Figure: 7). The Capacity-Area-Elevation curve was also used to determine volume of water at a given elevations.

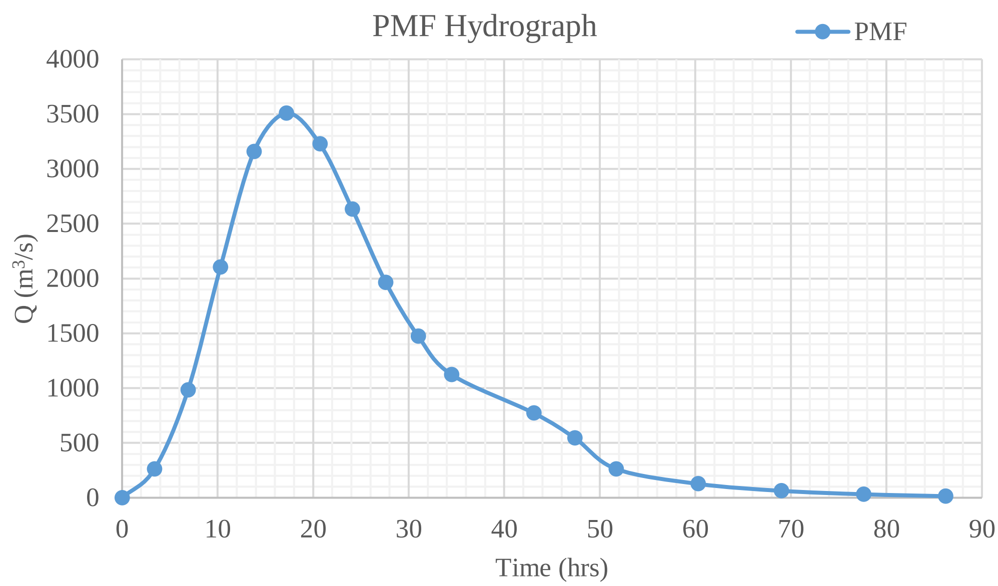


Figure 7: Gidabo flood hydrograph corresponds to PMF at dam site (Source: WWDSE, 2008)

3.2.2 Digital Elevation Model

Digital elevation model is very crucial data being used for determining elevations at any point. The cross-section of river at various points along the river is an important parameter in any dam breach analysis (Chandrabose & Nair, 2014). The generation of cross-sections data by field survey is a very laborious, and time consuming and also costly expensive. Hence, an alternative method for arriving cross-sections was investigated.

In this study, the cross-sectional data is aimed to be derived from ALOS PALSAR product digital elevation model recently released online by Alaska Satellite Facility. The DEM have spatial resolution of 12.5 x 12.5 meter, which is more detail than the previous SRTM release. It is freely available from https://www.asf.alaska.edu/ page and granted after only one time registration. After downloaded the Zipped folder contains different format of file, DEM was selected. A number of DEM tiles was downloaded to cover the whole area of interest, then it was mosaicked and subset the study area. Finally, HEC-GeoHMS was used to delineate Gidabo catchment as shown on Figure: 8 below.

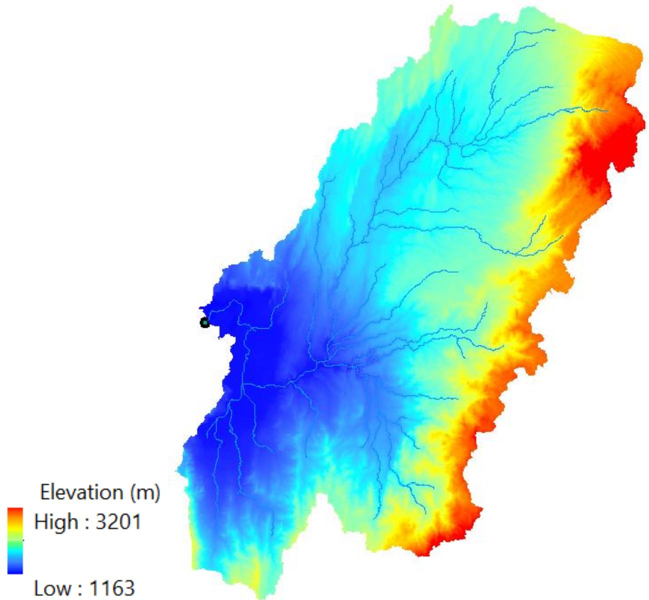


Figure 8: Gidabo catchment delineated using HEC-GeoHMS

3.2.3 Dam Salient Features

Dam physical characteristics are used to estimate breach parameters. Dam salient features such as width, height, upstream and downstream slope and storage capacity are the most important element for dam breach parameters estimation. Gidabo Embankment Dam salient features are given below.

Table 7: Gidabo Dam Salient Features (Source: WWDSE)

Items	Description
Dam type	Rock fill dam
Dam height above the bed level	25.8 m
Dam maximum bed width	210.38 m
Dam crest width	8 m
Dam crest length	335.52 m
Dam crest elevation	1225.8 m a.s.l.
Normal water level (NWL)	1219.5 m a.s.l.
Maximum water level (MWL)	1223.8 m a.s.l.
Upstream face slope	3H : 1V
Downstream face slope	3H : 1V
Reservoir capacity (NWL)	62.3 M m^3

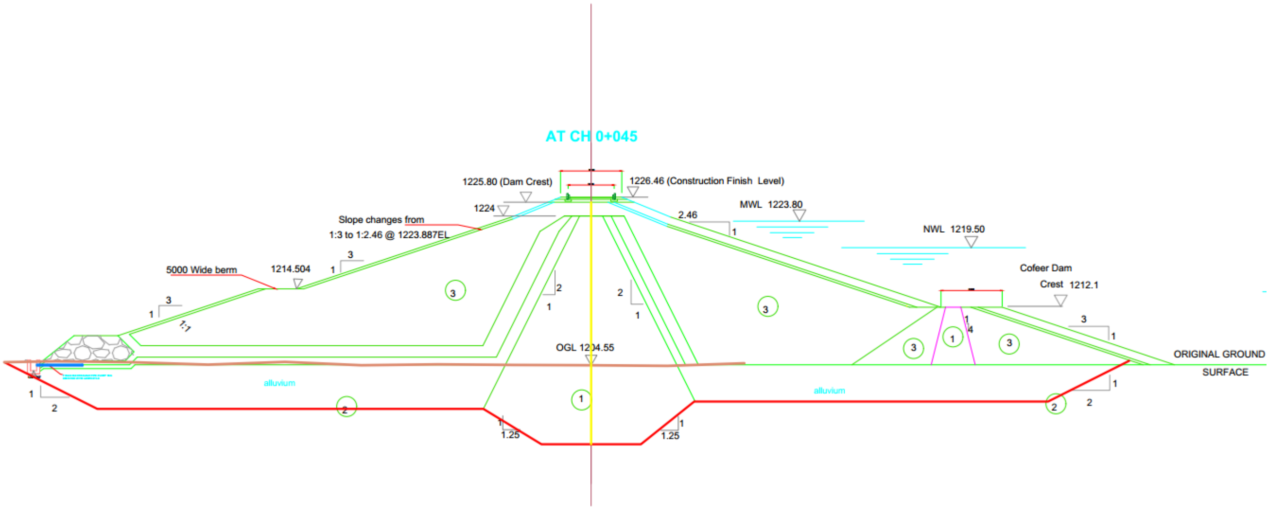


Figure 9: Gidabo Embankment Dam Cross-sectional view (Source: WWDSE, 2008)

Figure 10: Gidabo Embankment Dam top view (Source: ECDSWC, 2019)

3.3 Description of Selected Models

Basically, selection of the modeling tools should be based on the type of information desired from modelling effort, the specific conditions need to be modeled, level of accuracy and reliability. Furthermore, in our case the selection of the model were made based on the literature reviews and experience of the previous studies has been conducted on dam breach analysis and inundation mapping. For this study recent version of HEC-RAS and HEC-GeoRAS (an extension of ArcGIS) were selected. The details description of these models is given below.

a) HEC-RAS

River Analysis System (RAS) is the model package that was developed by the US Army Corps of Engineers Hydrologic Engineering Center (HEC) and has wide range of application in hydraulics. HEC-RAS has the capability of modeling breaching process of a dam and predict the catastrophic outflow hydrographs. It supports both overtopping and piping failure modes with the failure trigger being a target water surface, water surface and duration, or specific time.

To model a dam failure in RAS, we can enter the failure mode, breach size, and breach time. The breach size is defined by a trapezoid and the duration over which the breach occurs. Lastly, RAS allows the user to customize the progression of the breach over the full formation time (Cameron and Brunner, 2008). HEC-RAS has a very easy to use graphical users‟ interface (GUI) and also provides a highly efficient file management, data entry and editing, hydraulic analyses, and tabulation and graphical displays of input and output data (Brunner, 2014).

In HEC-RAS the 1D, stream flow profile follows the basic principle of physical laws such as conservation of mass and principle of conservation of momentum. These laws are expressed mathematically and referred as continuity and momentum equations. In unsteady flow, time dependent changes in flow rate are analyzed explicitly as a variable, while steady flow analysis models neglect time all together (Brunner, 2014). HEC-RAS unsteady flow analysis can be used to evaluate the downstream attenuation of the flood wave, providing a more accurate estimate of flood magnitude and velocity at critical locations.

b) HEC-GeoRAS

Geographic River Analysis System (GeoRAS) is an ArcGIS extension developed by the US Army Corps of Engineers Hydrologic Engineering Center (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. Within this model we can extract geometric data from digital elevation model either in TIN or GRID format. HEC-GeoRAS used twice in processing. The initial task executed in the HEC-GeoRAS to extract information for hydraulic modelling is called *Pre-Processing*. After unsteady flow simulation is done in HEC-RAS, results can be exported for post-processing in the HEC-GeoRAS extension of GIS. The user can read the HEC-RAS results into the HEC-GeoRAS and perform the flood inundation mapping (Brunner, 2014). Having exported the HEC-RAS output into HEC-GeoRAS, flood inundation maps in terms of water surface extent, depth and velocity are produced and this final process is known as *Post-Processing*. A summary of RAS Layers is provided in Table: 8.

Table 8: Summary of HEC-GeoRAS Layers and Corresponding results for HEC-RAS

RAS Layer	Description
Stream centerline	Used to identify the connectivity of the river network and assign river stations to computation points
Cross-sectional cut lines	Used to extract elevation transects from the DEM at specified locations and other cross-sectional properties
Bank lines	Used in conjunction with the cut lines to identify the main channel from overbank areas
Flow path centerlines	Used to identify the center of mass of flow in the main channel & overbanks to compute the downstream reach lengths between XS
Inline Structures	Used to extract the weir profile from the DEM for inline structures (i.e. dams)

ArcGIS is well suited to assist in performing dam failure analysis (Ackerman & Brunner, 2008) in combination with HEC-GeoRAS. RAS flood routing models provide GIS extensions which facilitate further analysis of their results on GIS platform. HEC-GeoRAS is an effective tool specifically designed to be used in ArcGIS software to facilitate import/export of data and results between HEC-RAS and ArcGIS. Such extensions are highly useful in dam break analysis as they provide an easy way to prepare inundation maps (Pandya et al., 2013). Hence, it will be utilized to generate visually informative flood inundation maps.

4. METHODOLOGY

The methodology used in this study is described briefly in subsequent sections.

4.1 Breach Modelling and Inundation Mapping Procedures

Data organization and analysis followed within this study as per model requirement is summarized here under. Figure: 11 shown the overall methodology were formulated in order to achieve the pre-defined research objectives.

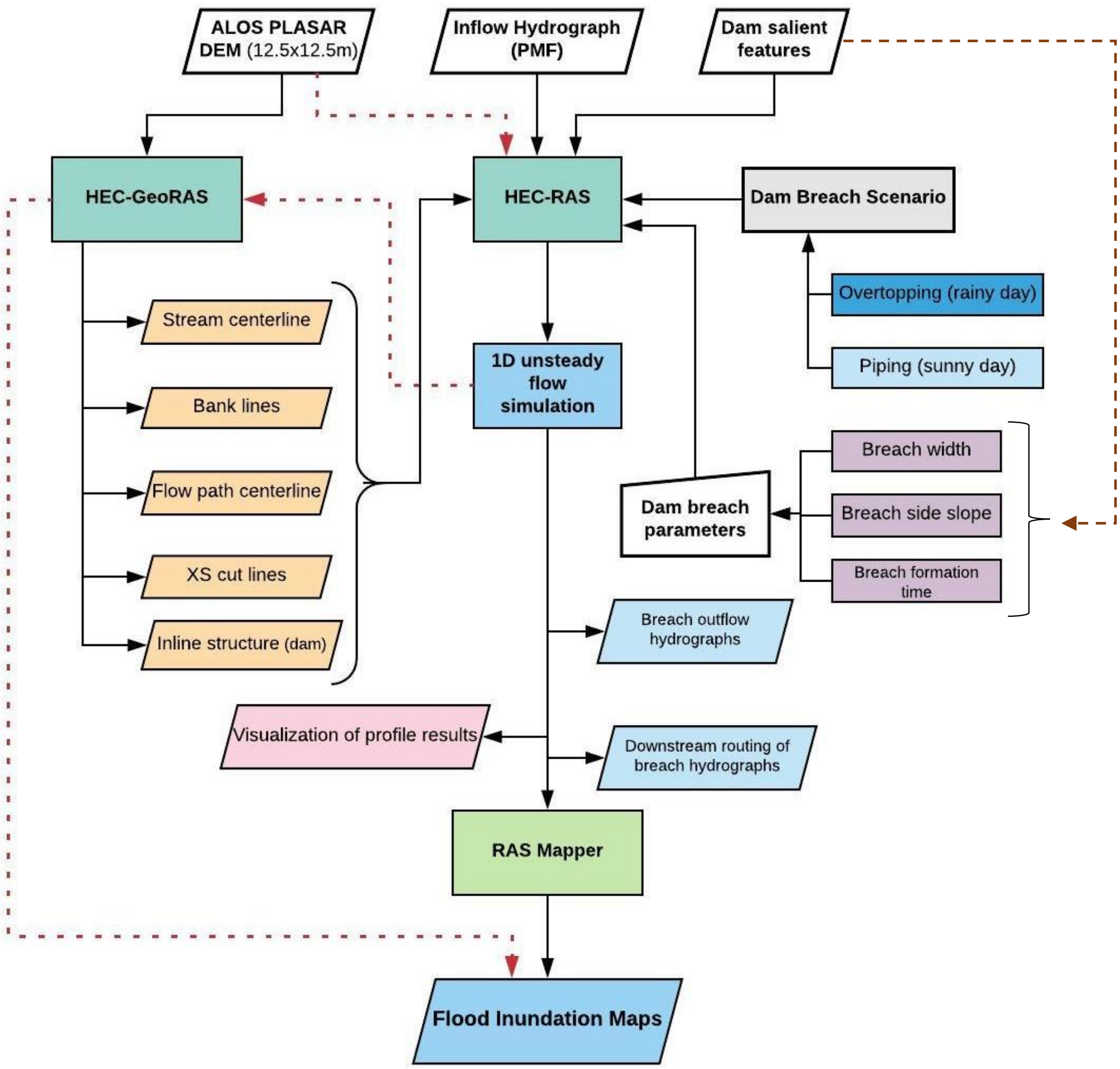


Figure 11: Methodology followed to perform the dam breach analysis and inundation mapping

The above flowchart summarized the procedures that were employed for dam breach analysis and inundation within this study. The red dotted lines indicate optionality of procedure. Prior to the methodology part, an extensive literature review were carried out in order to develop a robust approach. With the selected models, the first task is to extract the river geometrical data from digital elevation model using HEC-GeoRAS, the tools which used as a bridge between the ArcGIS and HEC-RAS. Then, extracted geometrical files can be imported into HEC-RAS. Great attention was paid to select the representative Manning's roughness coefficient that can affect the flow resistance within a channel and over the floodplain. In order to set boundary conditions of the model, inflow hydrograph (the probable maximum flood) was entered to the model as an upstream boundary condition. For the downstream boundary condition; normal depth with specified slope friction value was used.

Based on physical characteristics of the dam (dam salient features), dam breach parameters were estimated using empirical equations manually or within in the model. The selection of breaching scenarios as overtopping or piping failure also performed. Once the model setup done with all data mentioned data, then one dimensional unsteady flow simulation for each breaching scenarios was run. Output data was obtained at every time rate set initially before run. Finally, after unsteady flow analysis the output from HEC-RAS which contains cross-sections with water surface elevation attached to them was imported into HEC-GeoRAS in ArcGIS for post-processing and is used to prepare a flood inundation maps.

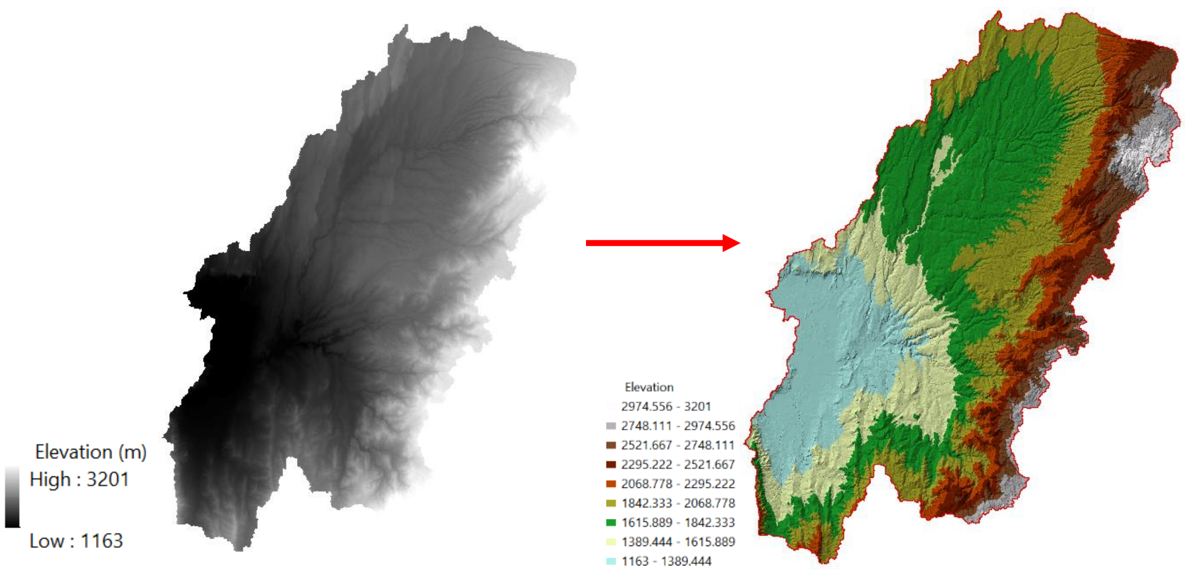
4.2 HEC-GeoRAS Model Development

HEC-GeoRAS model contains a set of tools specifically designed to process geospatial data to support hydraulic model development and analysis. As it was explain in chapter three under the section of model selection, HEC-GeoRAS was used for pre-processing when the initial task was to generate geometric data for HEC-RAS input and also for the post-processing to prepare flood inundation map. Under this section it is referred to only pre-processing steps.

4.2.1 Defining River Geometry

The geospatial data used for extraction of river geometry for hydraulic computation in HEC-RAS, can be prepared either by conducting detail survey or extracted from digital elevation model. For this study, the second option was selected. This is because of the fact that surveying data collection from field is time consuming task, some areas impossible to access (marshy land), and it is costly. Hence, the DEM which having spatial resolution of 12.5m by 12.5m (ALOS-PALSAR product) that obtained from Alaska Satellite Facility online can be used.

In order to extract the river geometry, DEM raster format was converted into Triangular Irregular Network (TIN) format. For this study area, 5 km downstream of the dam along the river, it is quite flat and very difficult to digitize the river profile from TIN (figure12 b) format with only visual inspection to generate geometric data. Hence, Gidabo River catchment was delineated using DEM raster by "HEC-GeoHMS" and used to locate stream positions in order to assist river digitizing. Topo Map from EMA were also used as a guidance when deal with river cross section. It is shown on figure 13. Absolute detection of river cross section from satellite image is really difficult. Usually high resolution data are required for hydraulic analysis. However, even if there is highest resolution data is available, it may not perfectly appropriate for most of hydraulic applications, but may be suitable for dam breach scenarios (Ackerman & Brunner, 2008).



(a) (b)

Figure 12: Gidabo catchment DEM (a) and TIN (b) for extraction of geometric data

The main geometric data can be generated within HEC-GeoRAS are stream centerline, bank lines, flow path centerline, cross section cut lines and defining inline structure (dam). Digitizing these elements from TIN was executed manually. However, regarding cross sectional cut lines the HEC-GeoRAS model itself facilitate automatic generation with user specified interval distance and width. It should be perpendicular to the flow and not to cross each other. Hence, it was edited one by one where it is required.

The Cross Sectional Cut Lines layer is the most important data constructed across the flood plain to capture the profile of the land surface. The Stream Centerline layer is used to identify the connectivity of river system from upstream to downstream direction. Inline structure layer were also used to identify dam. Once the RAS layers have been created, GeoRAS tools and menus are available to assign and populate attribute data. Finally, the data saved to the HEC-RAS geospatial data exchange format and imported into HEC-RAS model. The geometry of the Gidabo River generated HEC-GeoRAS was exported to HEC-RAS as shown in figure 13 below.

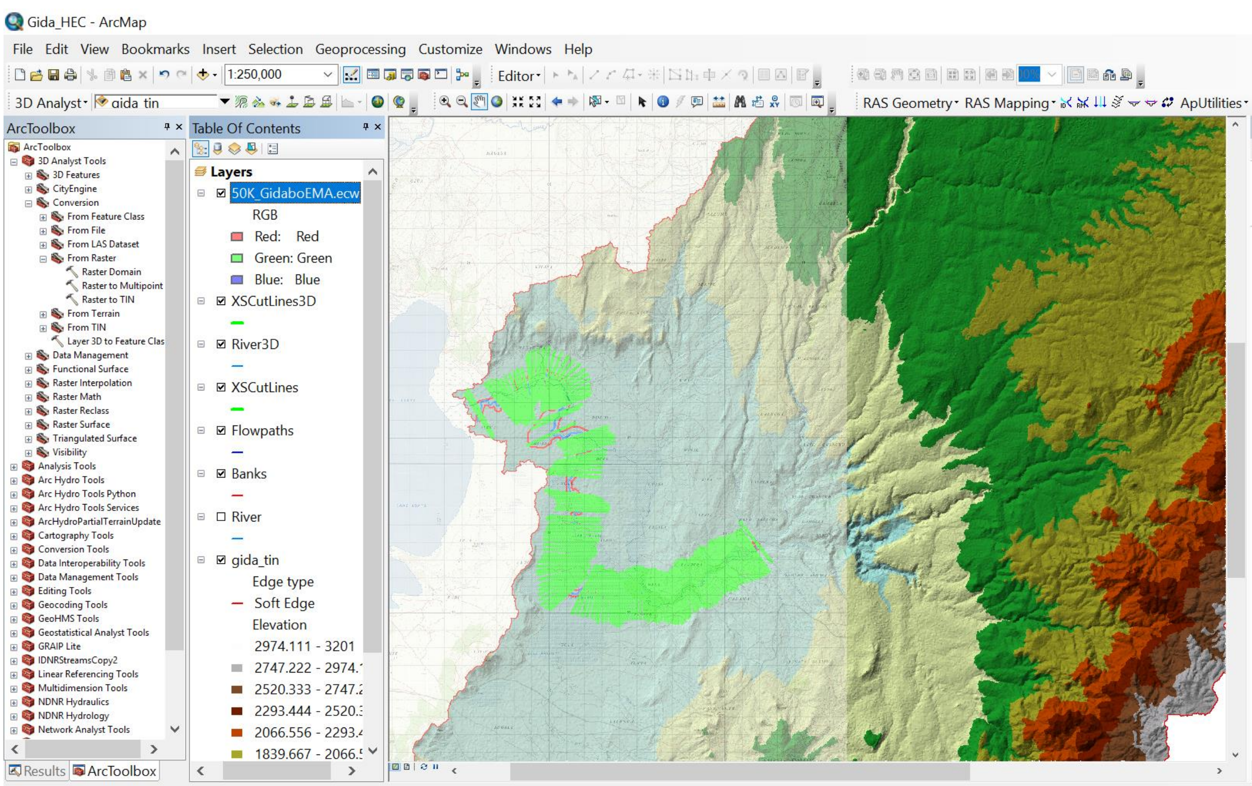


Figure 13: Geometric data extracted from the DEM with EMA map background

4.3 HEC-RAS Model Development

HEC-RAS model was designated with the capability to execute dam breaching analysis for both overtopping and piping failure scenarios. For this study, the dam failure was investigated for each failure mechanisms. The model development requires an accurate representation of geometrical data, initial and boundary conditions and estimation of breach parameters. For this study, one dimensional unsteady flow simulation was carried out. Particular information's required for HEC-RAS model setup for one dimensional unsteady flow condition are described in the subsequent sections.

4.3.1 Importing Geometric Data

After the river geometry has been extracted from DEM within HEC-GeoRAS, they need to be imported into HEC-RAS model. Then, the imported geometric data can be used to perform dam breach analysis with an integration of others input parameters/data. In HEC-RAS, the cross-section spacing can be interpolated further into a minimum spacing if needed. Once all the important geometric data created in RAS Layers (stream centreline, bank lines, flow path centrelines, XS cut lines) in HEC-GeoRAS, the setup should be imported into HEC-RAS model with its all required geometrical information. The Manning's equation and Inline structure can be adjusted from HEC-GeoRAS or in HEC-RAS.

For this study every required geometric data was created within HEC-RAS 5.06 version. Because, this new version is very handy and report the possible errors could be occurred during creating geometric data before use in the model run. However, since the research tittle for this study was formulated before the release of this new version to use the integration of HEC-GeoRAS and HEC-RAS, all geometric data were converted to shapefile and used within HEC-GeoRAS. The final simulation for one dimensional unsteady flow was executed with 5.01 versions. The figure 14 shown the geometric data on HEC-RAS with the background of Google Satellite image and terrain map created within RAS-Mapper.

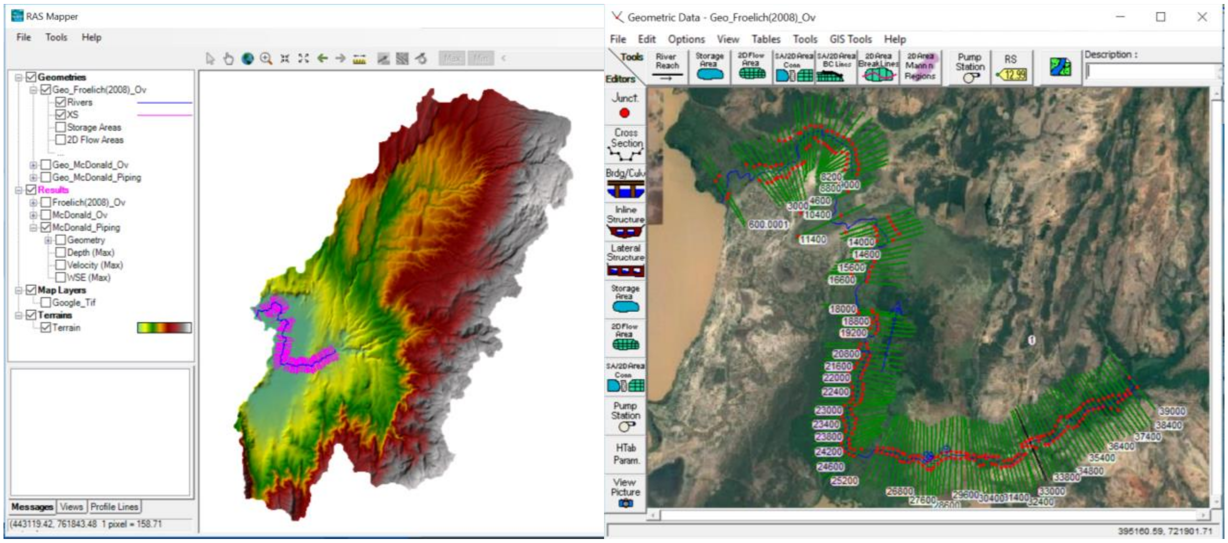


Figure 14: Geometric data with terrain background (left) and Google Satellite background (right)

4.3.2 Estimation of Dam Breach Parameters

Estimating the dam breach parameters is one of the most important things that have to be done before dam breach analysis is simulated. Determination of breach parameters are executed outside the model manually and entered to the model. Dam physical characteristic were used for breach parameter estimations. Within this study three empirical equations developed by Von Thun and Gillete, MacDonald and Langridge-Monopolis (1984) and Froehlich (2008) are used to estimate breach parameters. Detail descriptions of these equations were given under literature review. Both overtopping and piping failure was assumed for this study. Thus, breach parameters are estimated for both overtopping and piping and are used as an input for HEC-RAS. This breach parameters are breach width, breach side slope and breach formation time which used during unsteady flow analysis with other input data. Results of dam breach parameter calculations for both overtopping and piping failure mode using the Von Thun and Gillete, MacDonald and Langridge-Monopolis (1984) and Froehlich (2008) methods are described within the next chapter.

4.3.3 Establishing Boundary and Initial Conditions

For unsteady flow simulation, boundary conditions at both upstream and downstream ends of the model are needed. The selection of boundary conditions depends on the dam breach study's purpose, their locations relative to the areas of interest, and level of sensitivity based on the degree of confidence required. There are several types of boundary conditions available to the user within HEC-RAS interface like flow hydrograph, stage hydrograph, normal depth and etc.

Upstream boundary conditions are typically input as discharge hydrograph. Hence, in this study the probable maximum flood (PMF) hydrograph were considered for upstream boundary condition for the breach simulation. The downstream boundary conditions were set as normal depth with a friction slope of 0.002 values. Initial conditions also need to be defined in addition to boundary conditions. Within HEC-RAS there is an option to enter initial conditions under unsteady flow dialog boxes. To initiate the model for simulation a considerable amount of minimum flow and initial flow are set on the model.

4.3.4 Establishing Manning's Roughness Coefficient

The Manning's roughness coefficient (n) value is used to describe the resistance to the flow due to channel roughness caused by bed-forms (sand or gravel), bank vegetation and obstructions, bend effects, and etc. In an unsteady flow river routing simulation, results were often very sensitive to the Manning's coefficient values. The selection of the Manning coefficient is aimed to reflect the influence of bank and bed materials, channel obstructions, irregularity of the river banks. It is uncommon to think that a natural channel has single value of "n" for all occasions. Direct determination of the roughness coefficient is almost impossible in the study of natural river flows. Various factors affecting the values of roughness coefficients were presented by Chow (1988) and he provided suggestion on determination of representative roughness coefficient.

Manning's roughness coefficients for this study were selected based on standard values given by Chow (1988). An assessment of the land use land cover helped to provide relatively acceptable values of roughness coefficient. Based on suggested standard value by Chow (1988) and review of the characteristics of the study area, roughness values of 0.038 and 0.060 were used for the main channel and overbank (right/left) sides respectively.

4.4 Breach Simulation with HEC-RAS

Once the model set up done with all data mentioned earlier for 1D unsteady flow conditions under different breaching scenarios, then the HEC-RAS model was run to simulate the corresponding results. Simulation date and time duration was set before running the model. Output data obtained at every time rate set initially before run. The computational interval used the unsteady flow calculations was probably one of the most important parameters entered into the model. According to the Brunner (2016), a general rule of thumb is to use a computational interval that is equal to or less than the time of the rise of hydrograph divided by 20. The model simulation result is shown in next chapter.

4.4.1 Governing Equations for Solution of 1D Unsteady Flow

The two physical laws which govern the flow of water in stream for HEC-RAS computer program called the principle of conservation of mass (continuity), and the principle of momentum. These laws are expressed mathematically in the form of partial differential equations. The Saint-Venant equations are solved numerically in hydraulic routing to determine flow characteristics. Considering the elementary control volume shown in figure 15, distance x is measured along the channel. At the midpoint of the control volume the flow and total flow area are denoted Q(x,t) and A_T, respectively. The total flow area is the sum of active area A and off-channel storage area S.

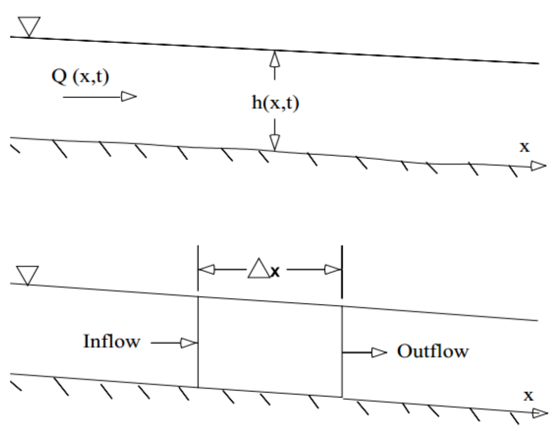


Figure 15: Elementary Control Volume for derivation of continuity and momentum equations

Continuity Equation describes conservation of mass for the one-dimensional system with the addition of a storage term, S. The continuity equation can be written as:

$$\frac{\partial A}{\partial t}+\frac{\partial S}{\partial t}+\frac{\partial Q}{\partial x}-q_1=0$$

……………………….......................….Equation 14

Where: x = distance along the channel, t = time, Q = flow, A = cross-sectional area, S = storage from non-conveying portions of cross-section, q_1 = lateral inflow

HEC-RAS unsteady flow engine combines the properties of the left and right overbank into a single compartment called the floodplain.

Momentum equation states that the rate of change in momentum is equal to the external force acting on the system. Conservation of momentum is expressed by Newton's second law. For a single channel momentum equation is written as:

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

…………………………..........….Equation 15

Where: g = acceleration of gravity, S_f = friction slope, V = velocity

The HEC-RAS computer program uses equations that describe 1-D unsteady flow in open channels, the **Saint-Venant** equations consist of the Continuity equation and the Momentum equation. The solution of these equations defines the propagation of a flood wave with respect to distance along the channel and time.

4.4.2 Breach Outflow Hydrographs

The main aim of dam breach modeling is mostly concerned with the determination of outflow hydrograph at time of failure. The volume represented by the hydrograph is the storage volume of the reservoir released during the breach. Factors that affect the shape of the hydrograph are include size and shape of the breach, breach formation time, depth of water at the dam, volume of stored water, surface area of reservoir, and shape of the reservoir.

4.4.3 Breach Outflow Hydrographs Routing Downstream

Routing breach outflow hydrographs was used to compute the discharge, water surface elevation, and velocity throughout the river reach. Dam breach flood hydrograph is a dynamic and unsteady situation with which HEC RAS use for analysis. Therefore, the preferred approach is to utilize a fully developed Unsteady State flow routing model. Basically, there are two routing methods know as hydrologic and hydraulic routing.

The hydrologic routing involved the balancing of inflow, outflow, and storage-discharge relation through use of the continuity equation. This application of hydrologic routing is used for reservoir routing. The reservoir component initiated by receiving upstream inflows and routed these inflows through a reservoir using storage routing methods. The hydrologic routing employs the continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the end.

Dam breach outflow hydrographs is used as inputs to the river routing through the immediate downstream reaches of dam site. Naturally, dam breach outflow hydrographs are highly unsteady flows that require a full unsteady flow routing method. In order to fully define an unsteady hydrograph, hydraulic routing employs the continuity equation and both energy and momentum balances to calculate open channel flow profiles. These equations are often referred as the St. Venant equations or the dynamic wave equations. The implicit formulation of this equation is well-suited from the standpoint of accuracy for formulating unsteady flows in a natural channel.

Routing of the dam breach discharge hydrograph is a required step in the development of flood inundation mapping for Emergency Action Plans. Routing of the breach hydrograph is performed to evaluate the attenuated or reduced peak discharge at critical locations downstream of the dam. In addition to calculate the attenuation, determining the flood wave arrival time and the depth or velocity of flow at some critical locations are also very important parts of the analysis. Hence, routing of hydrograph at different distance downstream of the dam was employed.

4.5 Developing the Flood Inundation Maps

Dam breach flood inundation map is a graphic display that can be used to indicate areas that may be flooded as a result of dam failure. The map develop may be used by a wide range of end-users for planning and as a response tool to determine the effects of dam failure in downstream areas. To prepare inundation map, the results generated within HEC-RAS is exported to HEC-GeoRAS on ArcGIS. The geo-referenced cross sections were imported and water surface elevations attached to the cross sections are used to create a continuous water surface. The water surface was then compared with the terrain model and the floodplain is identified where the water surface is higher than the terrain. RAS-mapper produces inundation maps for flood extent, velocity and depth that extend over the flood plain. Flood hazard map was also generated from depth and velocity maps product using the range defined according to FEMA 2018 guideline.

5. RESULTS AND DISCUSSION

Under this chapter the results and discussion based on the methodology have been followed on the previous section, which driven towards the objectives of the research is presented. Simulation of dam breach and inundation mapping are the essential mechanisms to characterize and identify the potential consequences due to the hypothetical dam failures on the downstream of the dam. The important points in dam breach analysis study are to estimate the breach parameters appropriately and the corresponding peak discharge which could possible inundates downstream part of the dam. For this study, the dam was tested for both overtopping and piping scenarios. The outputs from HEC-RAS one dimensional unsteady flow simulation for this study are discussed in the subsequent sections.

5.1 Dam Breach Parameters Results

The estimation dam breach parameters is one of the most important task that have to be done before running the model. Breach parameters can be calculated manually or by inserting the required physical characteristic of the dam such as top of dam elevation, pool elevation at failure, breach bottom elevation, pool volume at failure, slope of u/s and d/s face, dam crest width, and failure mode (overtopping/piping) in to the model. Table 9 summarize estimated dam breach parameters: Breach Bottom Width (W_b), Breach Formation Time (t_f) and Side Slope (SS) H:V using three empirical equation discussed under Section 2.5.

Table 9: Estimated Dam Breach Parameters

Methods	**Overtopping**			**Piping**		
	W_b (m)	SS (H:V)	t_f (hr)	W_b (m)	SS (H:V)	t_f (hr)
MacDonald et al	310.31	0.5	2.4	78.04	0.5	1.56
Froehlich (2008)	160.22	1	3.2	78.02	0.7	1.71
Von Thun & Gillete	106.05	0.5	0.78	91.03	0.5	0.64

The computer programs that build in HEC-RAS model for that empirical equation is assign the breach parameters value automatically once we click on select button. The values calculated manually by hand are the same with that of the model yield.

Estimation of breach parameter is significant on study the dam breach analysis. Because breach parameters like bottom width and breach formation time are the most governing parameters in the determination of the peak discharge and shape of outflow hydrographs. For instance, if the breach formation time is very small, then the time to peak is very short and also if the breach bottom width is narrower, the peak discharge is relative higher and vice versa.

MacDonald et al. and Froehlich (2008) have their own importance regarding breaching scenarios. MacDonald has differentiated the equation for earth fill dam and rock fill dam while Froehlich (2008) differentiate its equation for piping and overtopping. The breach parameters from both equations are more or less similar for piping failure. Hence, the peak discharge and peak time for these two equations are almost the same when seen from simulated hydrographs. However, there is some significant difference in the case of overtopping failure mode. For the case of Von Thun & Gillete, the equation underestimate the breach parameters in all case except for the breach width for piping when compared with the others two equations. This is condition affected the hydrograph generated for Von Thun and Gillete so far.

5.2 Dam Breach Simulation Results

One dimensional unsteady flow simulation is the basic part of dam breach analysis where flood hydrographs from the dam to the downstream part can be generated. With all the necessary input data and with no or very small significant errors adjusted within tolerance window, HEC-RAS can perform unsteady flow analysis. After entering geometrical data, boundary conditions for both upstream and downstream cross sections and initial conditions in to HEC-RAS unsteady flow simulation can be initiated for the specified period of time.

If the HEC-RAS setup have a significant problems, simulation can be stopped totally and indicate the red color line instead of blue. However, if there is simple problems within adjusted tolerable interval in the model prior to running, simulation will be completed with warning messages. For this study, simulation of steady flow and unsteady flow without inline structure (dam) were executed before final run in order to understand how the model is behave. After looking to the warning messages from several run and adjust one by one, the final simulation run was done. Simulation time window is set as February 2039 by guessing failure may occur after 20 years from now. For this study, six simulations run were performed totally.

Gidabo Dam was tested for both rainy-day (overtopping) and for sunny-day (piping) scenarios within this study. For the case of hydrologic failure (overtopping), upstream boundary condition was set as the probable maximum flood (PMF). While normal depths with the friction slope value of 0.002 was used as the downstream boundary condition for both overtopping and piping failure. It is also possible to adjust the HEC-RAS model set up used for overtopping failure by changing only the breaching mode (scenario). However, the assumption was making, dam failure may occur under normal condition without additional discharge due to piping occurred within the body of the dam. Hence, piping failure was checked for normal water level condition for this study.

Considering the overtopping failure, designed spillway may accommodate surplus discharge for a while during the event and may absent the failure of the dam. But in the case of piping it is totally different and impossible to stop the failure once the piping exceed the permissible amount. For this study, the normal water level is at 1219.5 m a.s.l. which is the same with the spillway crest level that designed for 1 in 10000 year return period. According to the spillway design report from WWDSE, 1 in 10,000 year return period is become half of the probable maximum flood (0.5PMF). Thus, Gidabo Dam is classified as low hazard and intermediate size category dam according dam classification guidelines. Detailed simulation results from HEC-RAS model for each failure scenarios are given under the next sections.

5.2.1 Breach Outflow Hydrograph

Dam breach out flow hydrographs from the model for both overtopping and piping failure are presented hereunder.

i) Outflow hydrographs from overtopping

Based on input parameters and boundary conditions set for overtopping failure mode for each empirical equation, the hydrographs was generated as follow. In compare to piping failure, the peak discharge simulated for overtopping is large. This indicates that the amount of water during PMF event is much larger than of the normal reservoir condition. As it tried to explain on previous section, the shape of the hydrograph can be determined by breaching parameters. For each the empirical equations, the hydrograph attain the second peak after18:00 hrs.

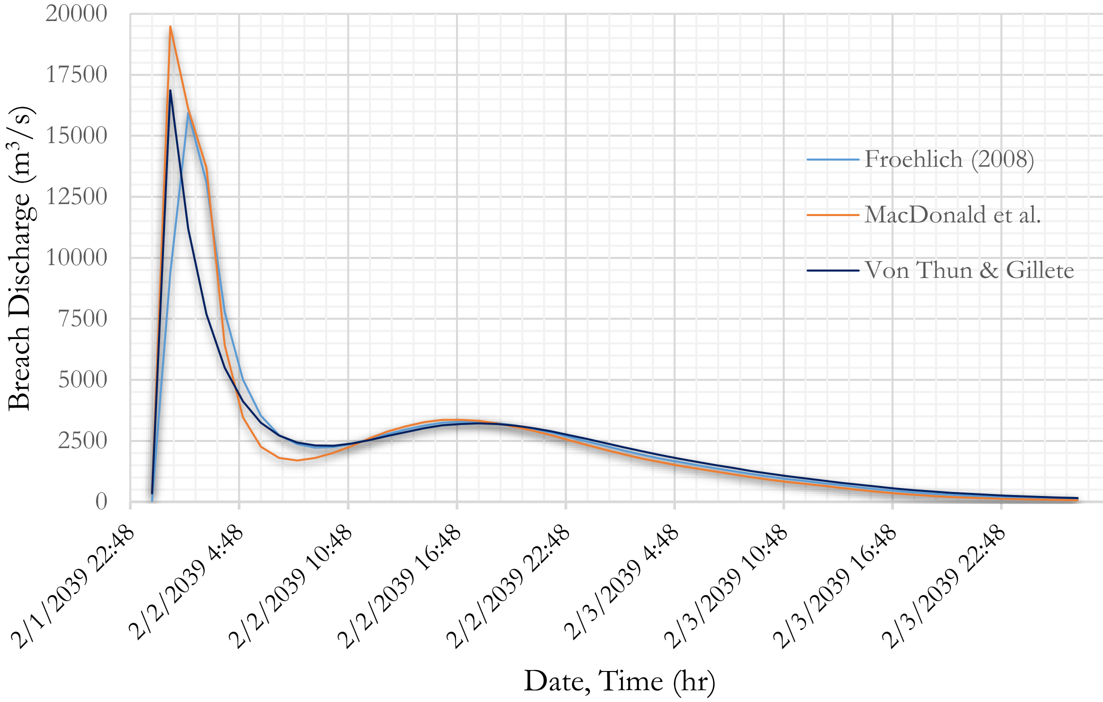


Figure 16: Breach outflow hydrograph for overtopping failure

ii) Outflow hydrographs from piping

Depend on input parameters & boundary conditions set for piping failure mode for each empirical equations, the hydrographs was generated as follow. In compare to the overtopping failure, the peak discharge simulated for piping is smaller. This indicates that the amount of water at normal reservoir condition is smaller than the discharge for PMF event. The same to the overtopping condition, hydrograph for piping failure also attain another smaller peak. The time to peak for the piping failure is a bit shifted downstream. This indicates the behavior of piping failure. Piping failure is not occurred quickly in compare to the overtopping condition. The piping failure start from downstream part to upstream gradually. From the flood inundation map, we can also observe that the velocity of piping failure is smaller in compare to the overtopping failure mechanisms.

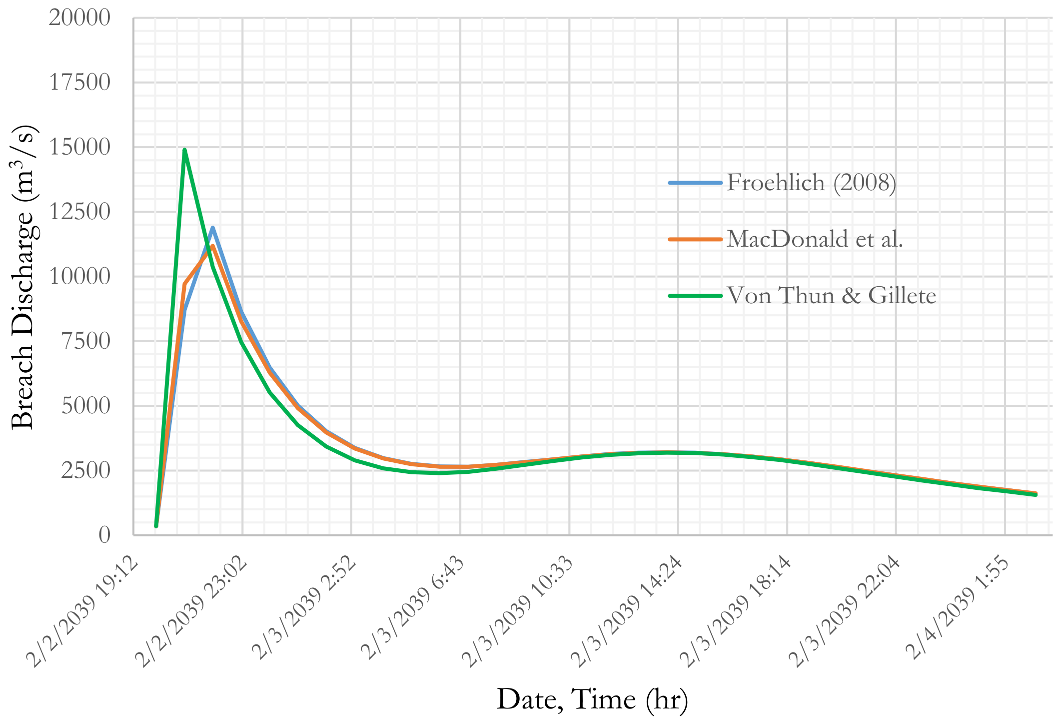


Figure 17: Breach outflow hydrograph for piping failure using Froehlich (2008)

5.2.2 Breach Outflow Hydrograph Routing Downstream

Flood routing approach is used to indicate the spatial and temporal variations of the flood wave throughout the river reach at downstream. When flood waves travel downstream there is attenuation and delay on the peak of flood. As the peak of the hydrograph decreases and the base time of the hydrograph increase.

The shape of outflow hydrograph generated downstream depends upon the bed slope, channel geometry and roughness, length of channel reach, and initial and boundary flow conditions. Within HEC-RAS, it is possible to obtain the hydrograph at each cross section. Usually, the peak of the hydrograph decrease from upstream to downstream. For this study, the peak of discharge simulated at the dam location and at different chainage downstream the dam. It is obvious that there is a decrease in the peak discharge and delay of time to attain the peak as we go downstream. It is also possible to extract such information at principal location downstream of the dam. Figure 18 and 19 below shows the corresponding hydrographs at the arbitrary cross-section for the overtopping and piping failure cases, respectively downstream the dam.

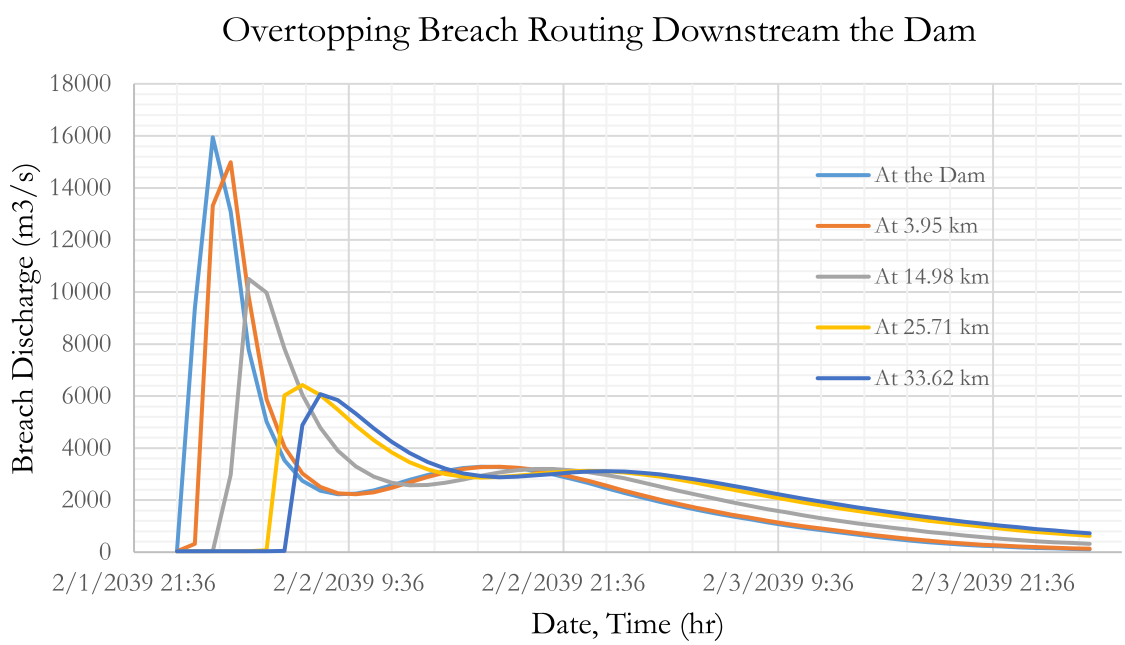


Figure 18: Water Surface elevation mapping for overtopping failure using Froehlich (2008)

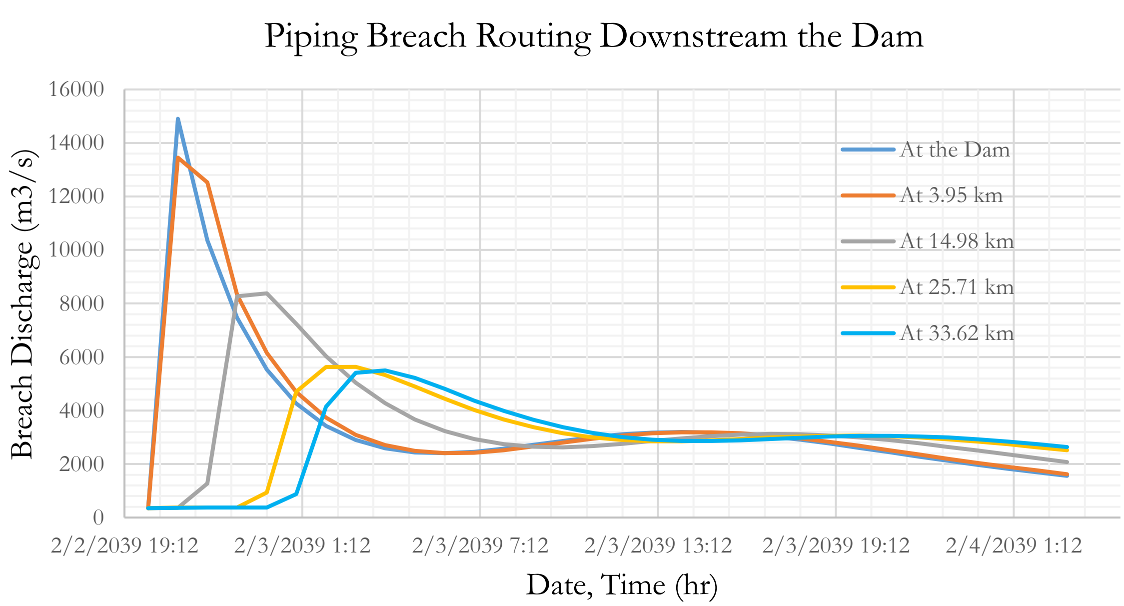
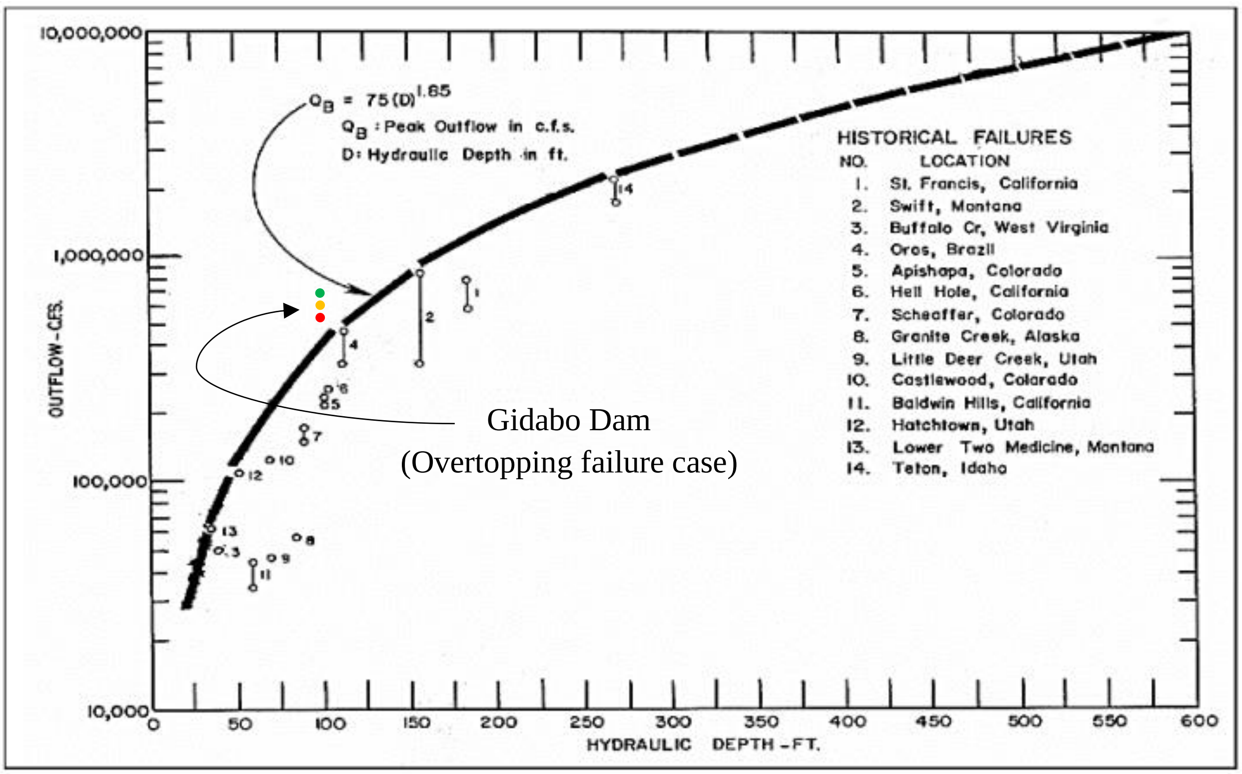


Figure 19: Depth of flood inundation mapping for piping failure using Von Thun & Gillete

5.2.3 Peak Flow Envelope Curve

In order to test the reasonableness of simulated peak discharge from HEC-RAS model, we have to compare it with the historic dam failure for which regression equation was developed for. Several scholars have been developed peak flow regression equations from historic dam failure data and developed envelope curve. Hence, the peak discharges was converted in to cubic foot per second with the corresponding depth of water above the breach invert at time of breach for each breaching scenarios. Figure 20 and figure 21 have shown the range of comparisons for both overtopping and piping failure for the Gidabo Embankment dam.

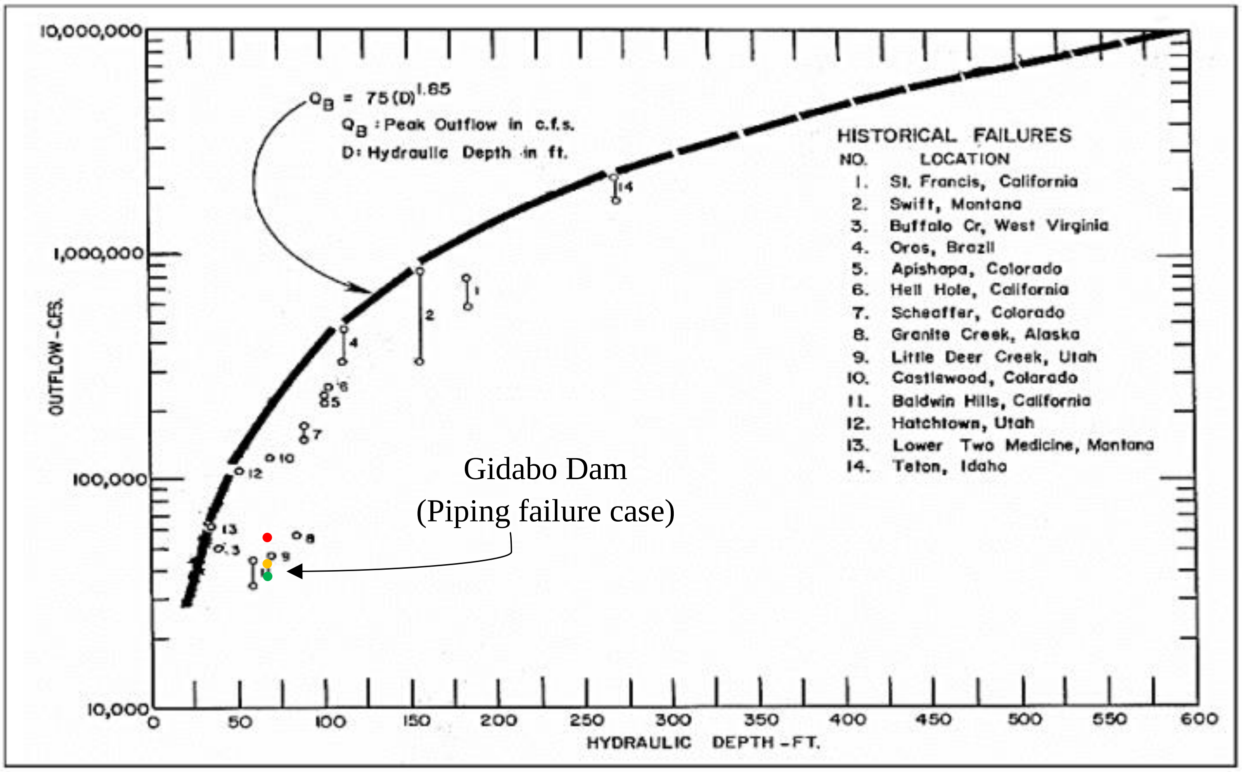
a) For overtopping failure with hydraulic depth of 89.8 ft.



- MacDonald et al Q_{peak} = 687350.2 cfs
- Froehlich (2008) Q_{peak} = 563098.7 cfs
- Von Thun & Gillete Q_{peak} = 595692.7 cfs

Figure 20: Envelope curve comparison for Gidabo dam for overtopping failure

b) For piping failure with hydraulic depth of 64 ft.



- MacDonald et al Q_{peak} = 395069.3 cfs
- Froehlich (2008) Q_{peak} = 419896.3 cfs
- Von Thun & Gillete Q_{peak} = 526343.6 cfs

Figure 21: Envelope curve comparison for Gidabo dam for piping failure

For the case of overtopping failure mode, Froehlich (2008) was predicted the results that closer to the envelope curve as compared to the others two methods. More or less the result peak discharges are close to each other's. This indicates that the simulation results are reasonable for our study. For the case of piping failure, the obtained results are a bit far from envelope curve. However, the results are approximately similar to the failure of historical dam for Granite Creek, Little Deer Creek and Baldwin hills dams for empirical equations of Von Thun & Gillete, Froehlich (2008) and MacDonald et al respectively. Thus, the simulation for the piping is also reasonable.

5.2.4 Cross Sectional, 3D perspective, and Longitudinal Plot Profiles

Downstream of the dam after about five km, the area is quite flat. Hence, to capture the flood over the floodplain cross sectional distance of 3 km were used with 200 m intervals. About 8.5 km downstream of the dam along the river at left side, there is potentially possible area to be inundated as observed from terrain map and topographic map of the area which need to extent the river cross sections beyond three kilometres. However, for this study we did not able to capture the flood at those positions since the constructed cross section are for three kilometres only. Figure 22 provide the cross sectional profile of station 35600, which is closer to dam location from upstream side. In compare to most of the downstream, this station have u-shaped valley which makes it suitable for the construction embankment dam.

Figure 23 also provide the XYZ perspective or (3D) profile plot and longitudinal profile of the Gidabo River respectively. The 3D water surface profile is used to understand the pattern of river meandering and extent of spreading water surface over the floodplain. While the longitudinal profile helps to understand the drop of head of water surface following bed surface elevation. As we observed from figure 23 the area study areas below dam site is very flat and have small vertical difference as we go down. This issue was affected determination of appropriate floodplain to capture all extent of the flood for this study.

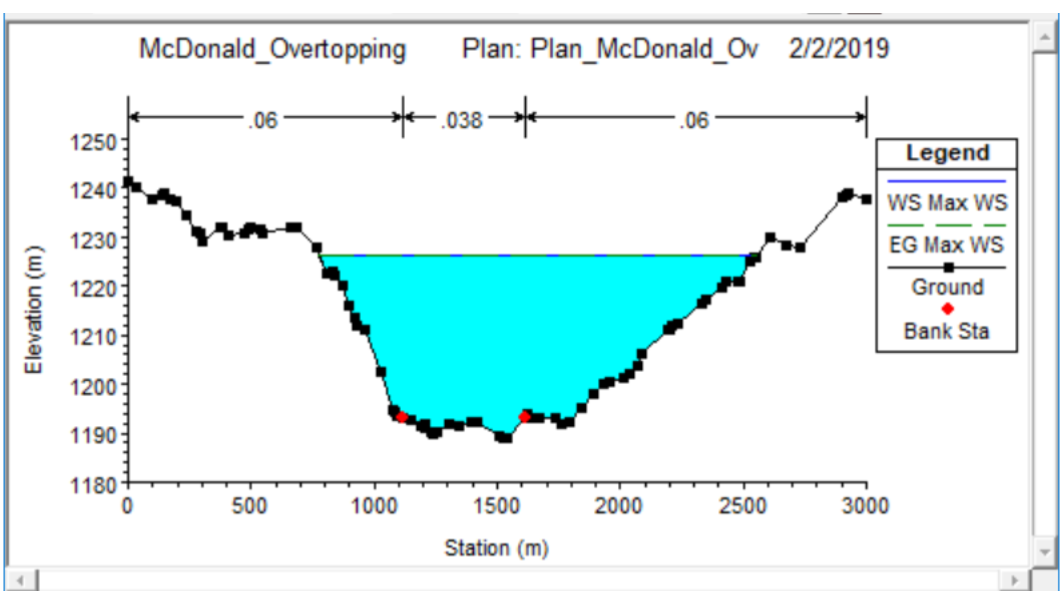
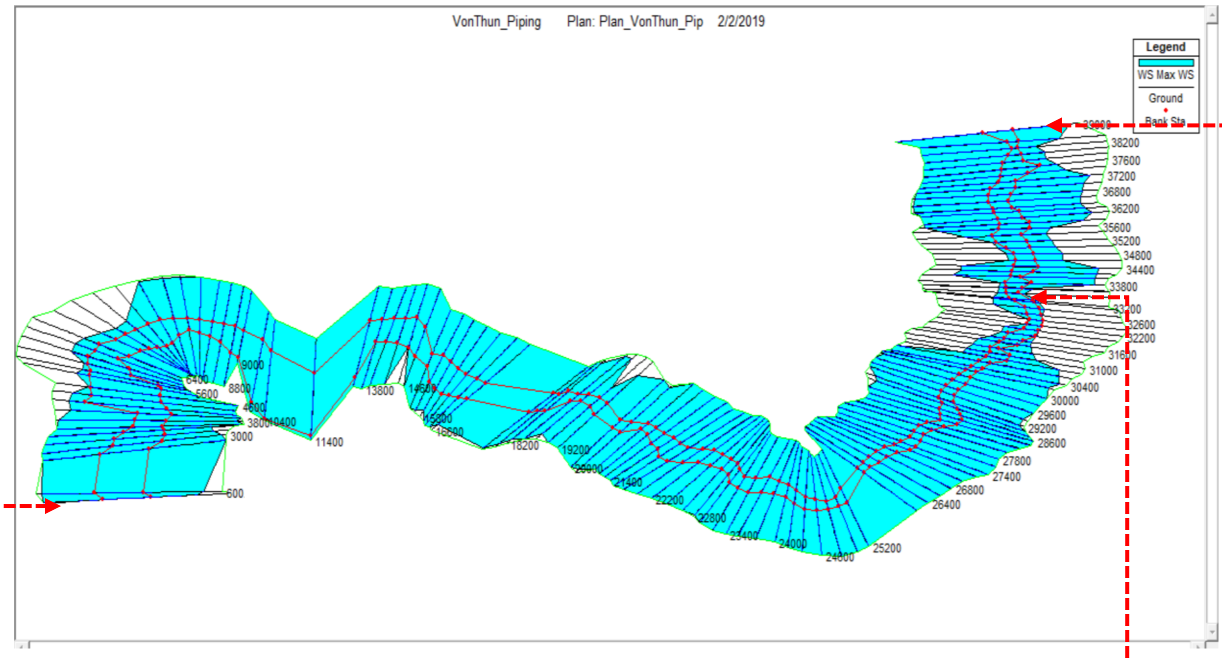


Figure 22: Cross sectional view of bed profile at upstream side near to the dam



3D water surface profile of Gidabo River

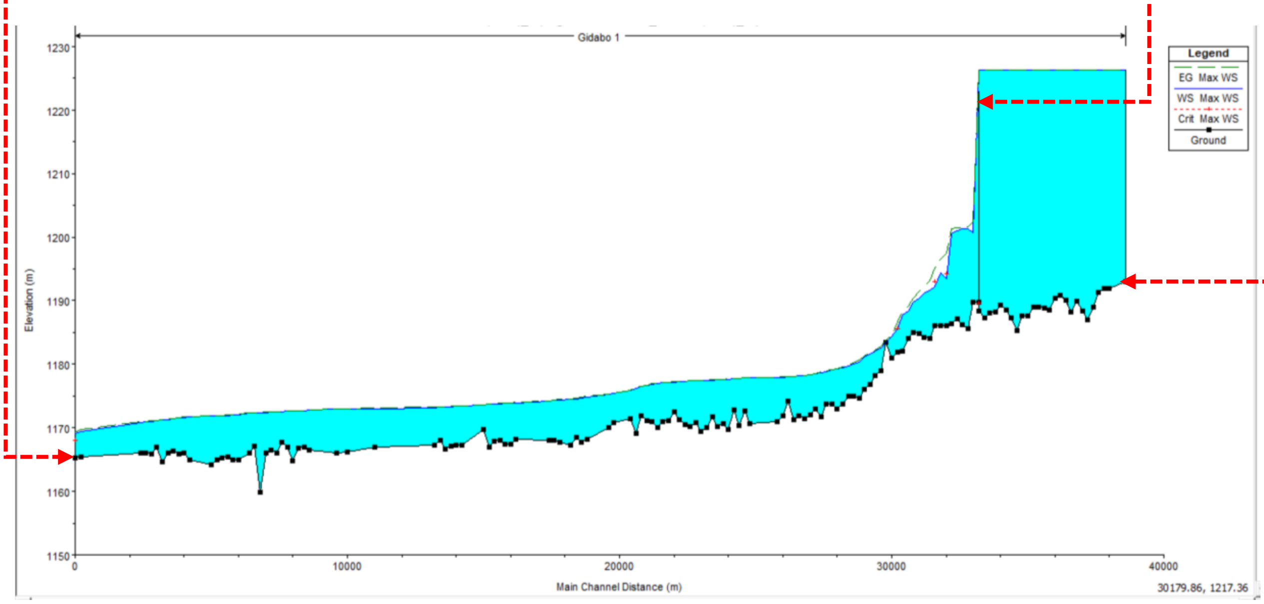


Figure 23: Longitudinal cross section for Gidabo main channel

5.3 Flood Inundation and Flood Hazard Mapping

Flood inundation maps are used as graphical tools for different purpose as discussed under literature review. Hence, the preparation an accurate flood information like flood extent, depth and velocity are most significant. The development of effective emergency action plans requires an accurate prediction of inundation levels, time of flood arrival and spatial extent of the flood. In other side, inaccurate flood inundation map may result on extra cost or also loss of life and property damage. Therefore, flood inundation mapping are demand great attentions. For Gidabo dam breach analysis flood inundation map was developed for both overtopping and piping failure mechanisms.

Since the peak outflow hydrograph that obtained from HEC-RAS using breach parameters for set Froehlich (2008) method were more closed to the envelope curve, the flood from Froehlich (2008) were used for overtopping failure. In the same manner, the flood obtained from Von Thun and Gillete were used for the piping failure flood inundation mapping. RAS-mapper was used for flood inundation mapping to indicate the aerial extent, depth of flood and also velocity map of the flood. Hence, RAS-mapper were used for flood mapping with the background of image from google satellite and terrain map (refer appendix). From velocity map we can observe that, there is very high velocity downstream of the dam for overtopping failure mode compare to the velocity map for piping failure. This is because of the amount of discharge and time of breach formation. Hereunder on the figure 24, the flood inundation boundary which generated by overtopping failure was displayed.

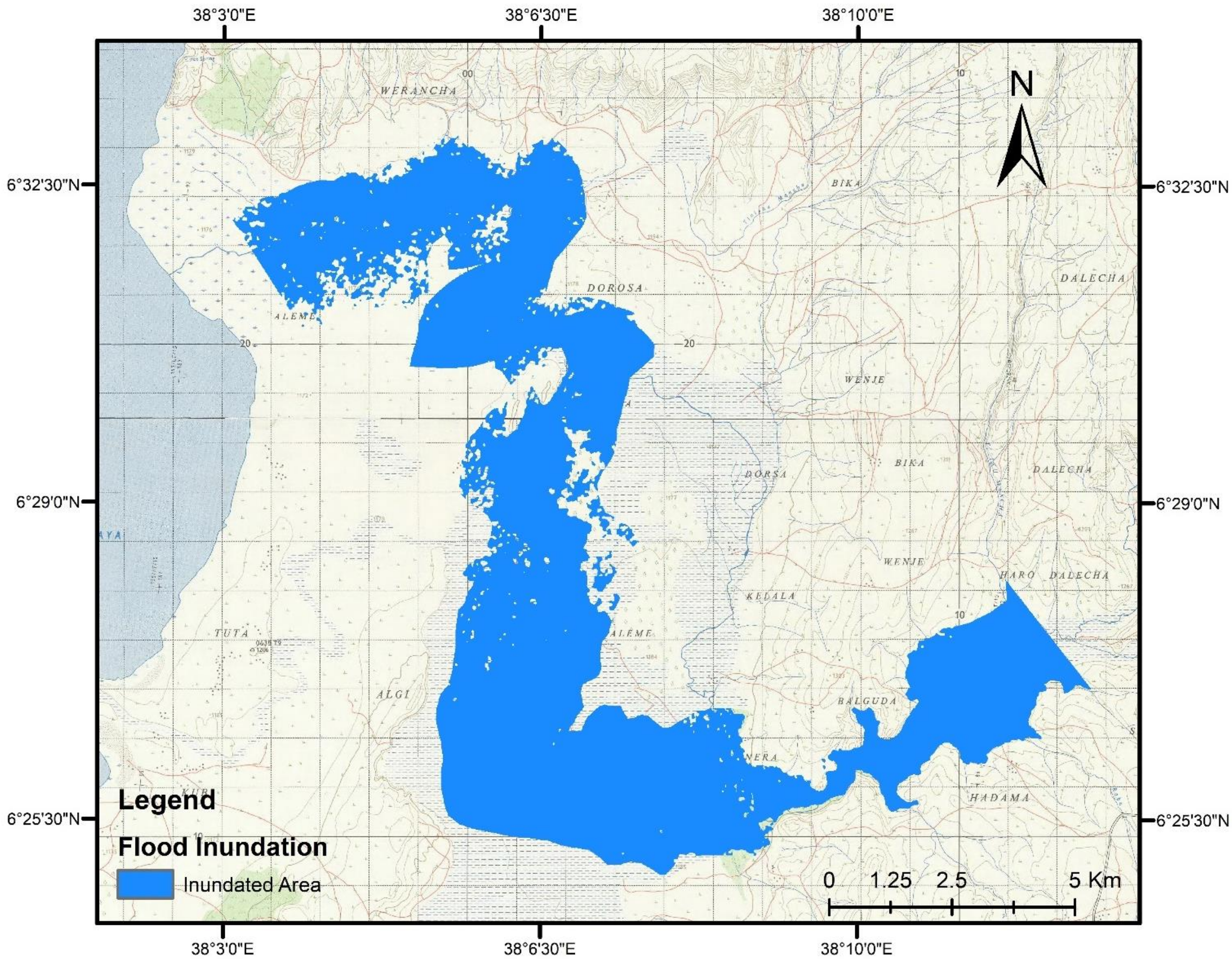


Figure 24: Flood inundation mapping for overtopping failure using Froehlich (2008) method

A flood hazard is an indication of the possible source of danger due to flooding effect. However, does not imply any risk unless persons or objects that are vulnerable to damage are exposed to it. Usually, the hazard to be mapped includes the flood inundation areas, water depths and velocities, and arrival times of flood waves. There are different guidelines available nowadays to categorized flood hazard. Hazard regime may be defined differently for children, adults, vehicles and building. The following category on table 10 was adopted from FEMA, 2018 guideline and also from (Central Water Commission, (2018). The flood severity map represents the combined effect of depth & velocity, and most often categorized as Low, Medium, High, Very High and Extreme Hazard.

Table 10: Flood severity categories based on flood depth and velocity (Source: FEMA, 2018)

Flood Severity Category	Depth * Velocity Range (m^2/Sec)
Low	< 0.2
Medium	0.2 - 0.5
High	0.5 - 1.5
Very High	1.5 - 2.5
Extreme	> 2.5

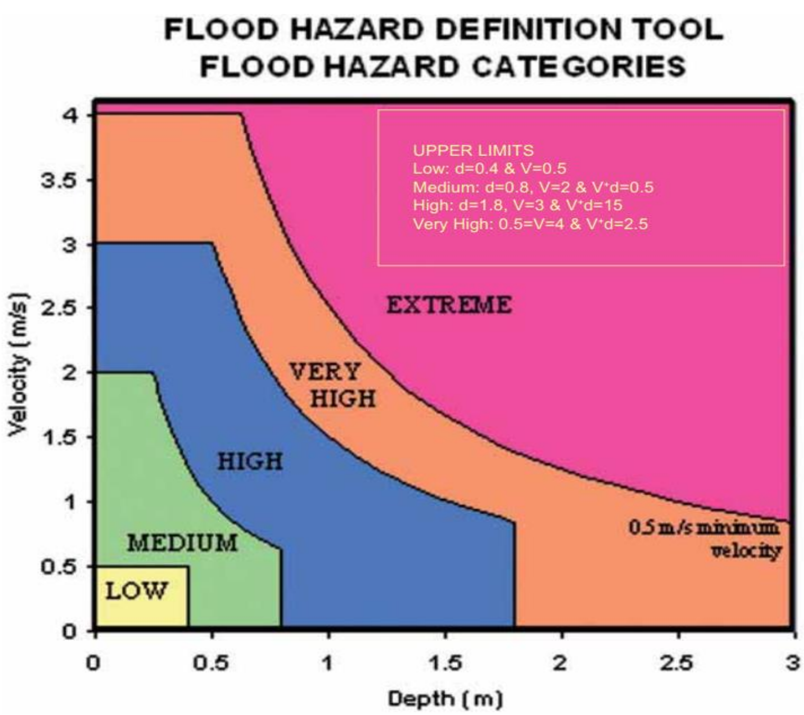


Figure 25: Flood hazard categories (Source: FEMA, 2018)

Figure 26 and 27 below indicate the flood severity map developed for the case of overtopping and piping failure respectively as the product of flood depth and flood velocity. As we can observe from the maps most of the areas are categorized under extreme condition. This indicates that either the depth or the velocity floods are more. When they multiplied; yield greater than the values indicated on the table 10 given above. The flood risk would be depend on the live or properties liable to the damages. In order to generate flood risk map, we need to conduct socio-economic study, which beyond the scope if this study. Hence, the flood risk maps were not communicated here.

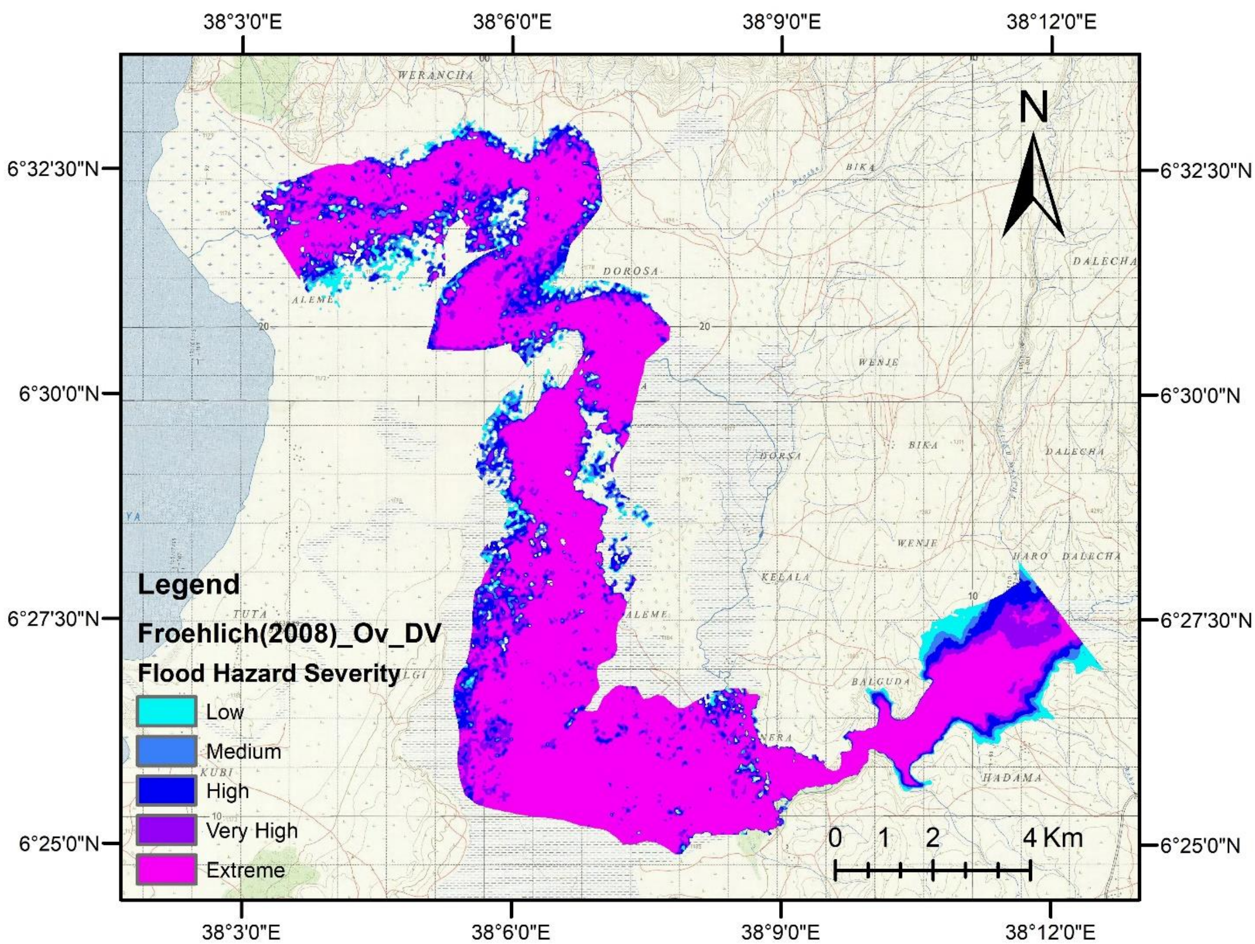


Figure 26: Flood hazard map for the overtopping failure using Froehlich (2008)

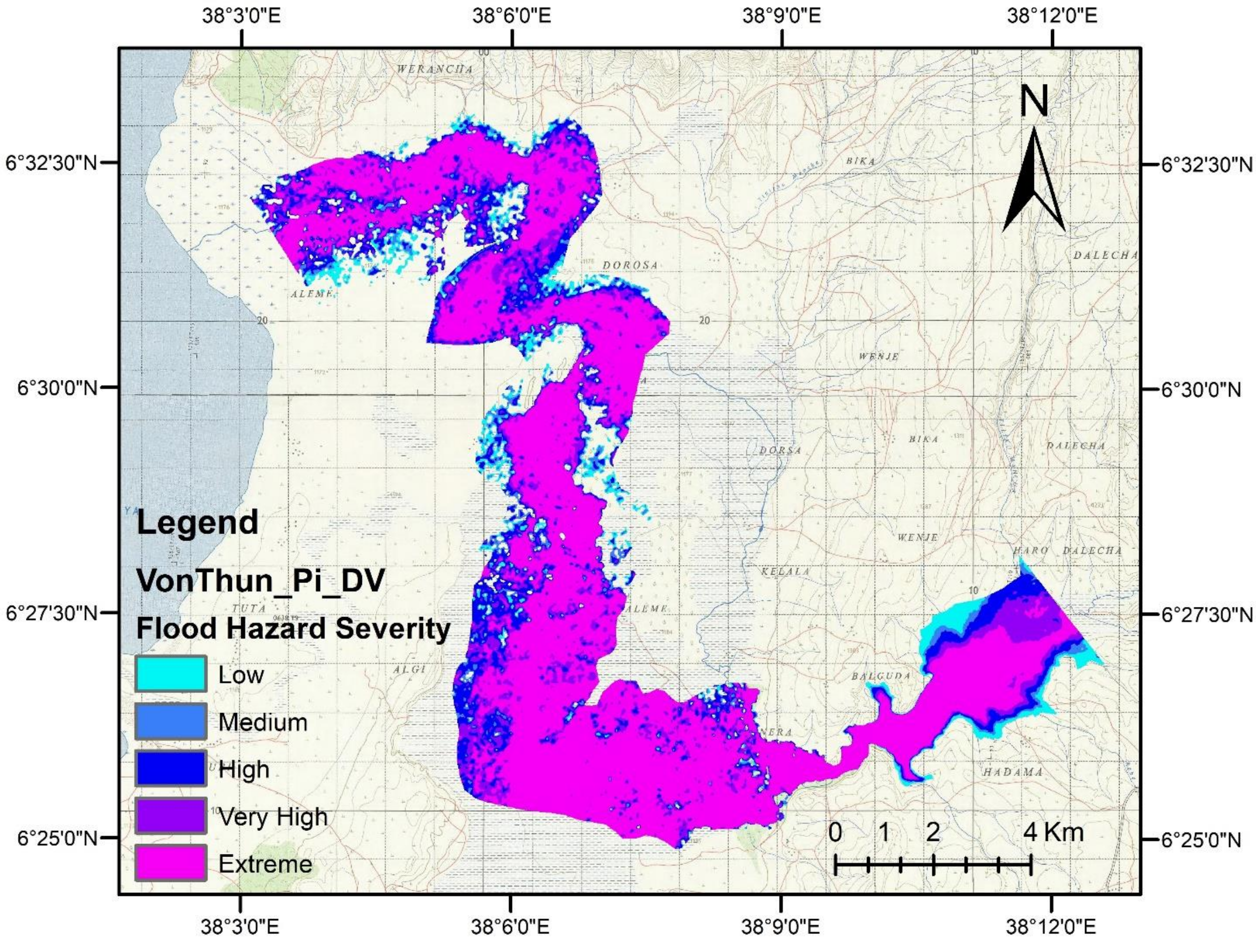


Figure 27: Flood hazard map for the piping failure using Von Thun and Gillete

6. CONCLUSION AND RECOMMENDATION

6.1 Conclusion

Dam breach analysis and inundation mapping are of the most significant study in need since the water stored behind the dam has capable of resulting on high devastation at the time of breach. There are various assumptions and techniques available for dam breach analysis studies. In the present study, an attempt was made to simulate dam breach and development of flood inundation mapping for Gidabo embankment dam which found in the Southern part of Ethiopia, using HEC-RAS model. The geometric data used within HEC-RAS was extracted from high resolution digital elevation model from ALOS PLASAR product which having 12.5 x 12.5 m spatial resolution using HEC-GeoRAS. The dam was tested for both overtopping and piping failure scenarios. Overtopping (*rainy-day*) failure due to the probable maximum flood and piping (*sunny-day*) failure at normal pool level was analysed. Different set of breaching parameters has been utilized from empirical equation developed from historical dam failures.

From one dimensional unsteady flow simulation in HEC-RAS model, it is recognized that the breaching parameters set for Froehlich (2008) was yield the result more approaches to the envelope curve developed from 14 historic dam failure with the peak of 15,945.14 m^3/s. Hence, flood inundation map obtained from this method was developed. In the same manner, Von Thun and Gillete was used for flood inundation map for piping failure with the peak discharge of 14,904.39 m^3/s. Towards the research objectives, assessment of breach parameters has been done. However, in order to state which key parameters are highly influence the dam breach, the sensitivity analysis need to be performed by tuning the values of each parameters by keeping one parameter constant and changing others. Obviously, it is consuming much more time and did not performed for this study. However, from the experience of modellers and literature review, it is seen that manning's roughness is one of the sensitive parameters for dam breach peak discharge in addition to breach width, and time of formation of failures. Routing of hydrograph at the end of the downstream was also done to indicate the arrivals of flood wave. Finally, the flood inundation map describes extent of the flood, depth of the flood and velocity of the flood magnitude was developed. These maps could be possibly used for dam owners to develop emergency action plans.

6.2 Recommendation

Since dam breaches have high potential damage in compare to the normal flood events, it needs great attention in order to safe loss of life and property damage. Development of appropriate flood inundation mapping requires an accurate dam breach analysis study supported by detail available data like cross sectional data and manning's roughness data. In current study, there was shortage of the cross-sectional data. Even if high resolution DEM were used, the extraction of geometric data was challenging since the areas are more flatter. This is because as the terrain become flatter and flatter the accuracy of the DEM could be decrease so far. Left side the downstream of the dam, there is an area potentially could be flooded. Since I consider it as small part of the land parcel, did not included in the simulation process. However, in order to develop fully dynamic one dimensional simulation, it needs to be considered as the part of the study. Therefore, I recommend that further study should be consider it as one tributary in addition to the main reach and execute the simulations.

Breach models have different level of accuracy and computational algorithms. Therefore, it is better to re-check the model output with the different dam breach and flood model having high level of accuracy and strong computational algorithms. In order to develop safe and economical emergency action plans, it is best practice to simulate dam breach with different breach scenarios to see the different range of hazard. Finally, I would like to recommend that the dam owner and concerned body have to pay special attention to the dam breach monitoring and develop the best strategies to communicate EAP with the stakeholders in case the dam is suspected to the fail.

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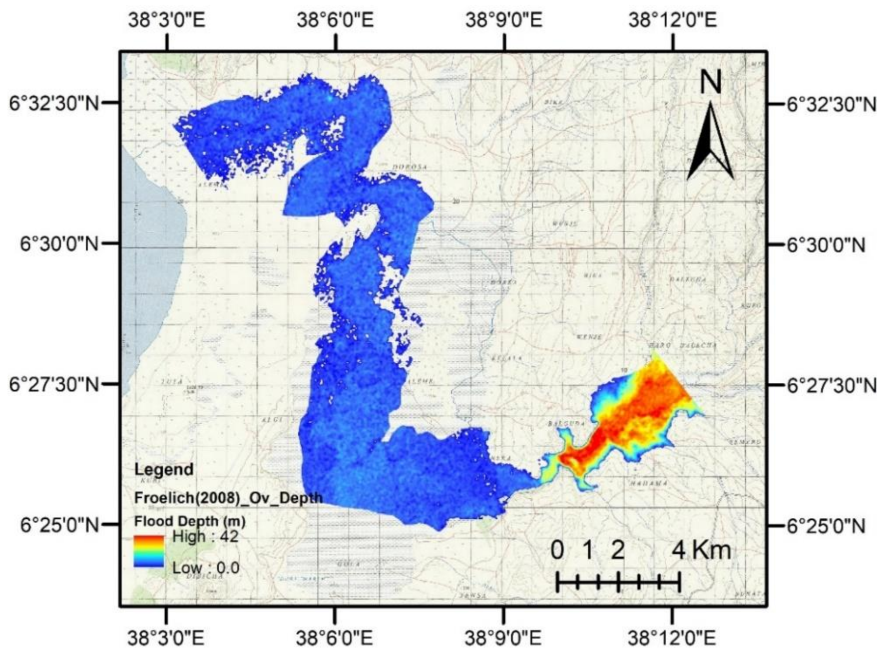
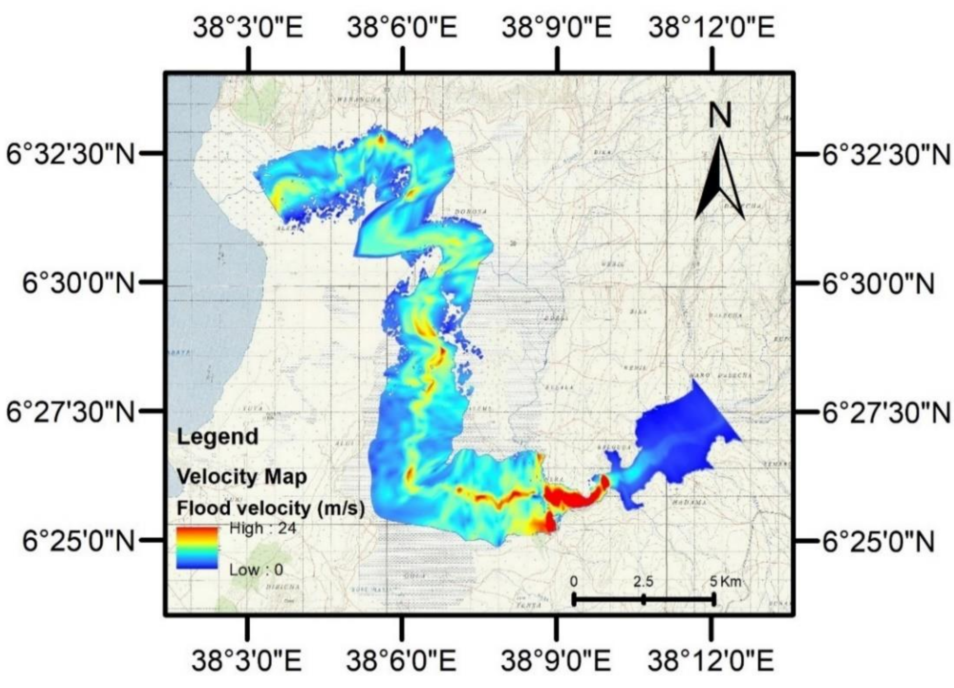
APPENDICES

Flood Inundation Maps

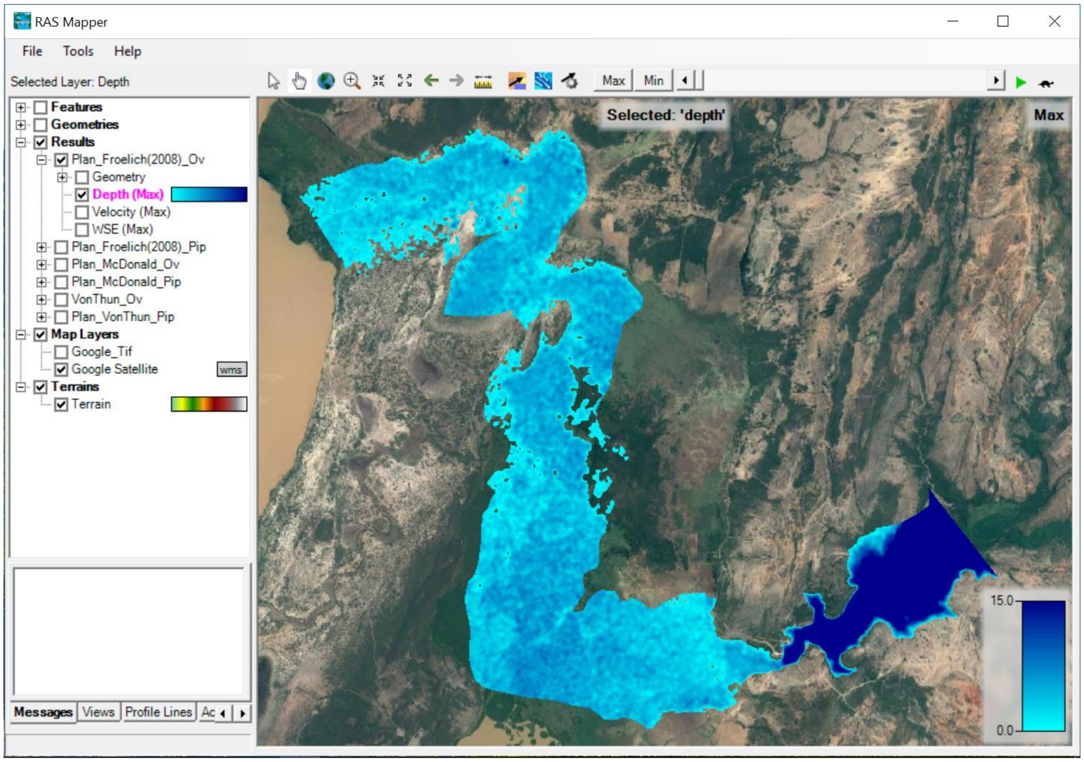
Depth of flood inundation map for overtopping failure using Froehlich (2008) method



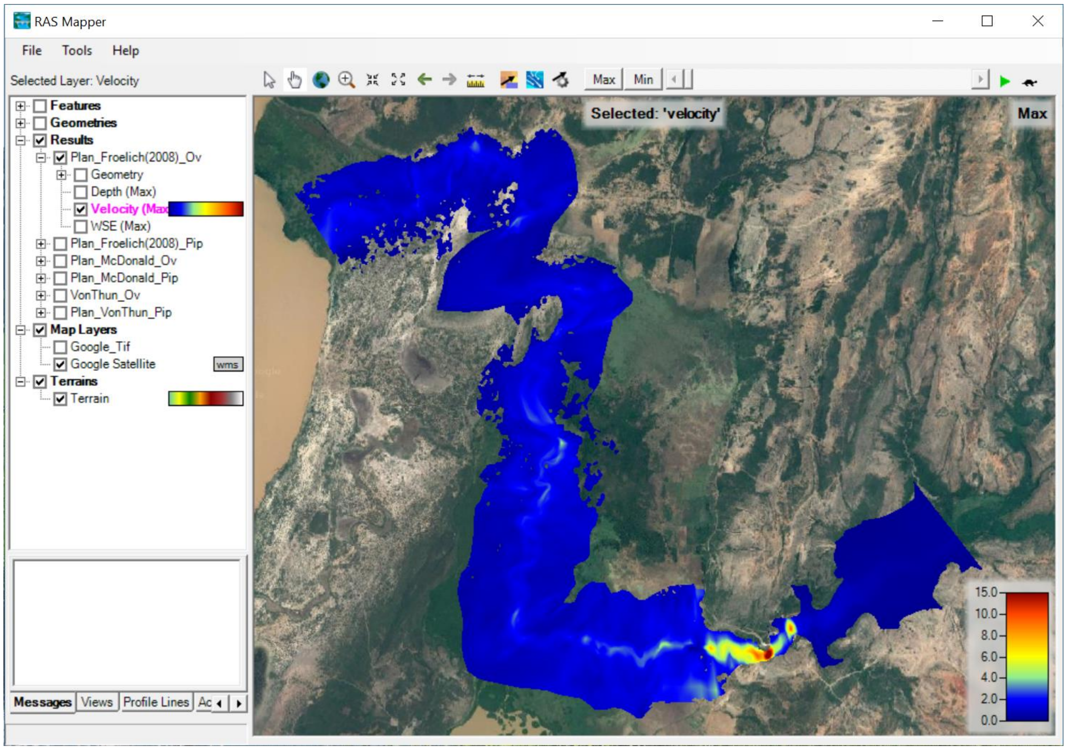
Flood velocity map for overtopping failure using Froehlich (2008) method



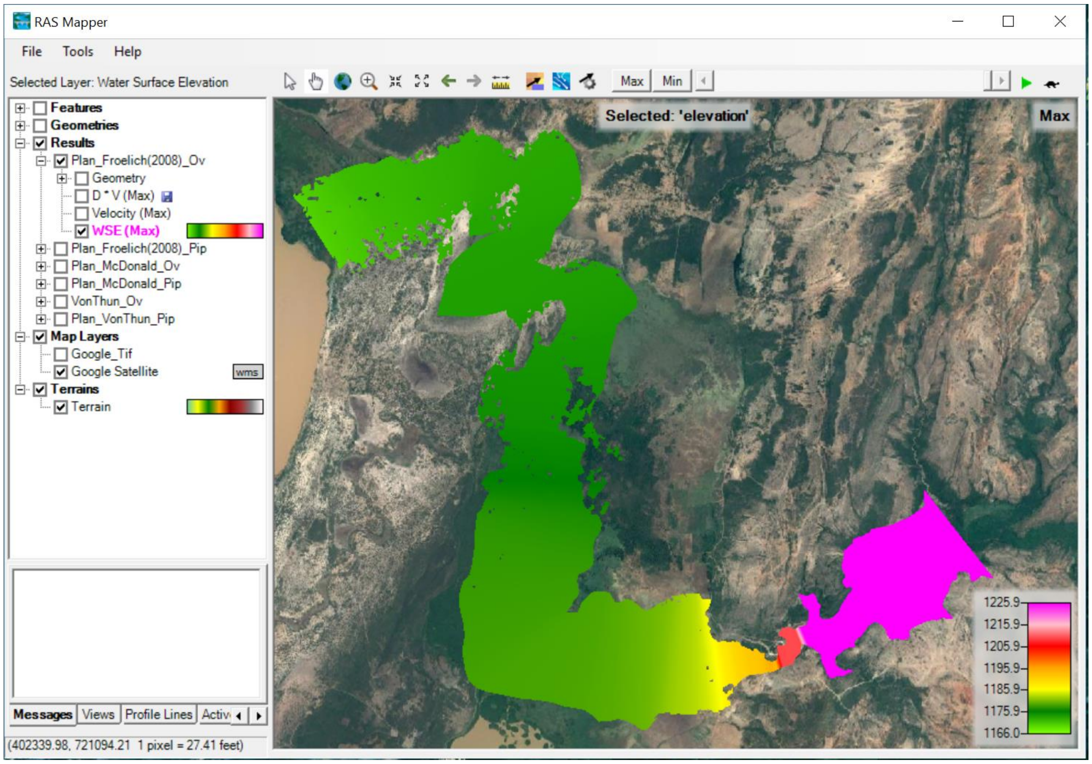
Depth of flood inundation map for overtopping failure using Froehlich (2008) method



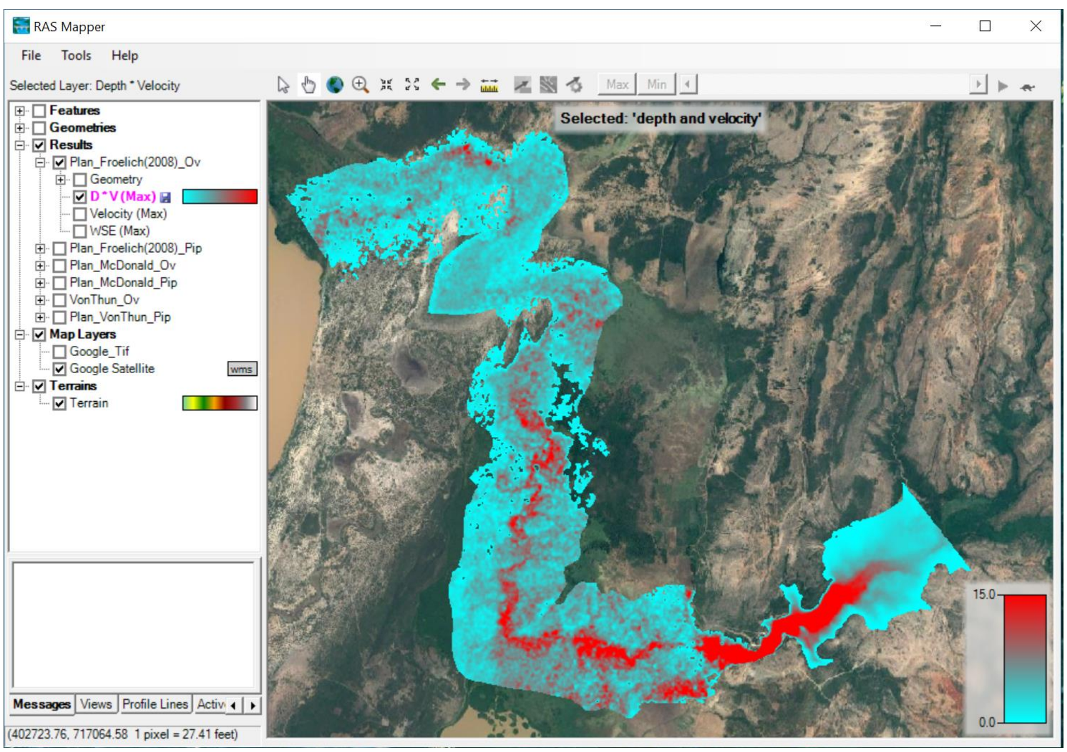
Flood velocity map for overtopping failure using Froehlich (2008) method



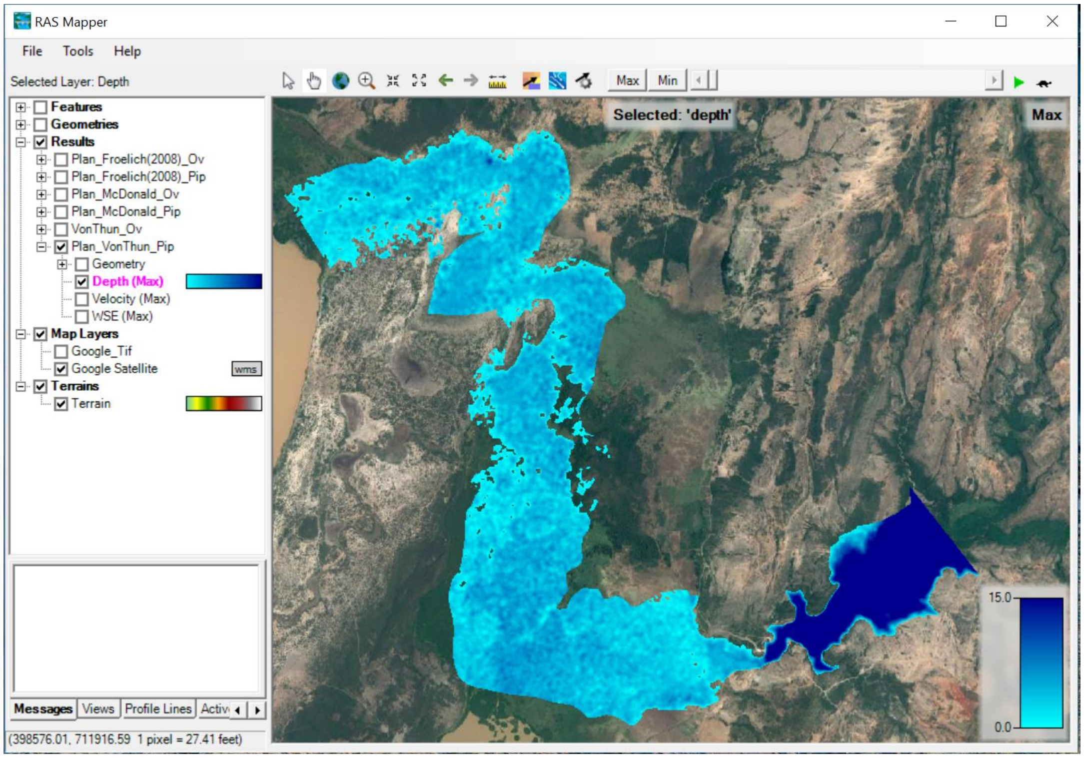
Water Surface Elevation map for overtopping failure using Froehlich (2008) method



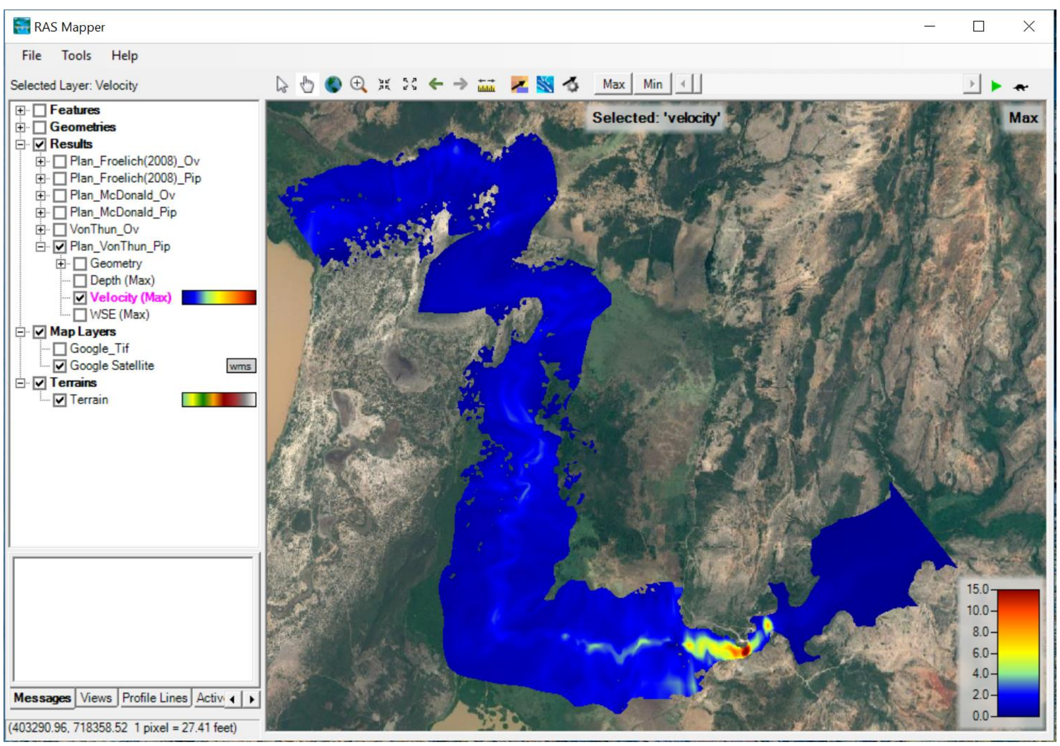
(Depth * Velocity) map for overtopping failure using Froehlich (2008) method



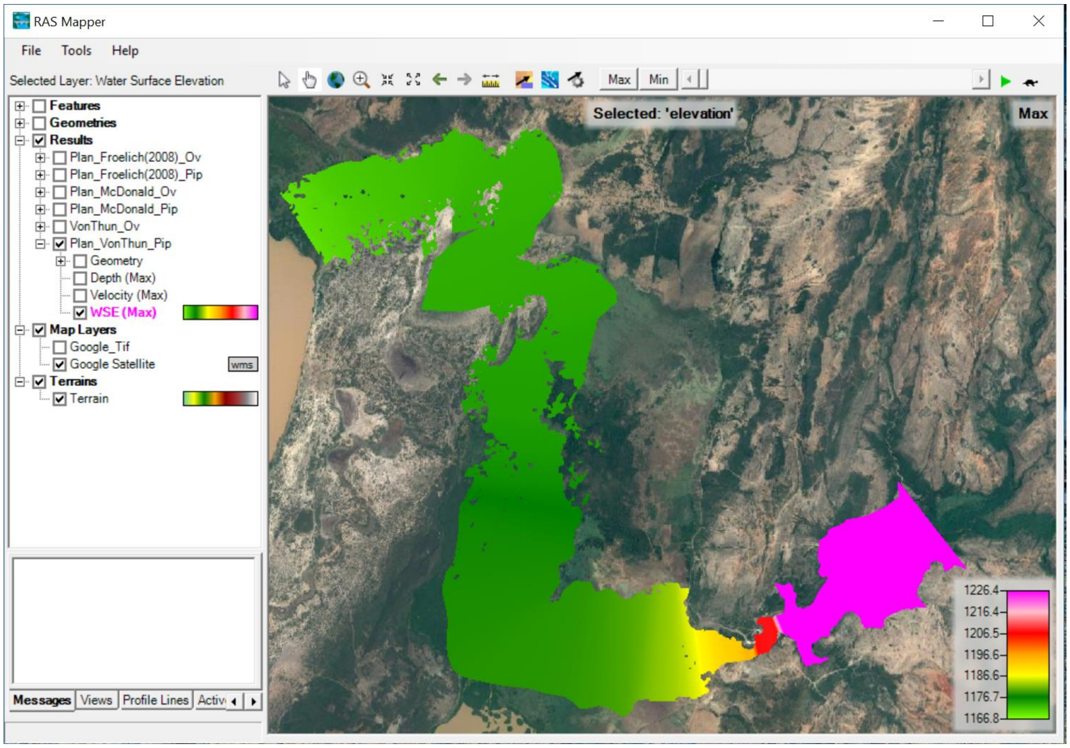
Depth of flood inundation map for piping failure using Von Thun and Gillete method



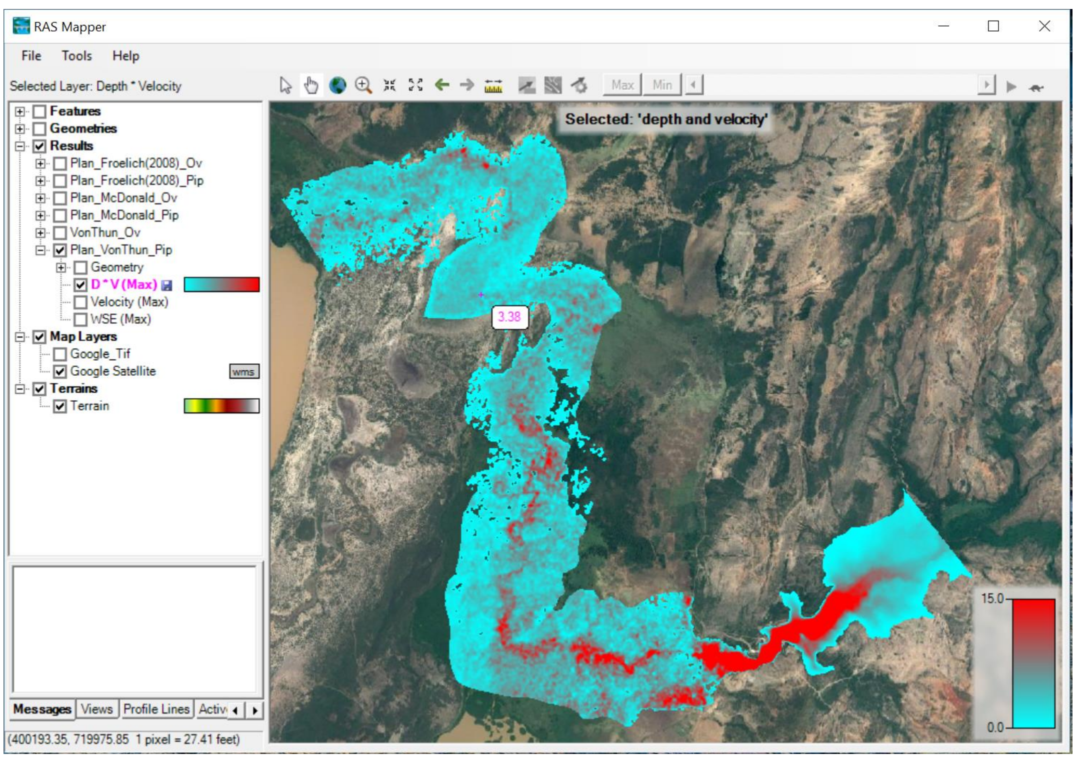
Flood Velocity map for piping failure using Von Thun and Gillete method



Water Surface Elevation map for piping failure using Von Thun and Gillete method



(Depth * Velocity) map for piping failure using Von Thun and Gillete method



Gidabo Dam site Elevation-Area-Volume Curve