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Dam Hazard Classification using ENTRO Guideline in the case of Fato Dam, Ethiopia

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The undersigned have examined the thesis entitled "Dam Hazard Classification using ENTRO Guideline in the case of Fato Dam" presented by Firaol Chalchisa Feyisa, a candidate for the degree of Master of Science, and hereby certify that it is worthy of acceptance.

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CERTIFICATION

I, the undersigned, certify that I read and hear and recommend for acceptance by Addis Ababa Institute of Technology a thesis entitled "Dam Hazard Classification using ENTRO Guideline in the case of Fato Dam" in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (Major Hydraulic Engineering).

Dr. Netsanet Nigatu (Supervisor)

DECLARATION

I, **Firaol Chalchisa Feyisa**, declare that this thesis is my own original work and that it has not been presented and will not be presented to any other University for a similar or any other degree award.

Signature

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DEDICATION

Dedicated to my grandmother **Berhane Desta Dori**!
I wish I could cherish this moment with you, **Akkoo**!

ABSTRACT

The phenomenon of spontaneous breaches of dams and consequent flooding that have occurred throughout history have made it necessary to design dam safety plans and hazard management measures. In this case, the preliminary job was analyzing the dam breach before the event and ranking these dams based on risk. The pre-event analysis of a dam breach scenario for the Fato dam was the focus of this thesis. A deterministic approach is applied to model the dam breach. The extent of the resulting flood inundation is mapped and overtopping, and piping failure scenarios are evaluated. The upstream boundary condition comprises a 2.3*PMF inflow hydrograph and a base inflow hydrograph, and the downstream boundary condition is made up of a riverbed slope assuming the bed slope and energy grade line slope are equal to HAEC. The main reason why the PMF flood to overtop to dam needed is large is that a large freeboard is provided for the Fato dam about 3 meters here between the spillway crest and the dam crest. Seven 2D simulations were run, five of which are for various overtopping failures and two of which are for piping failure modes. HEC-RAS Version 6.3.1 hydraulic modeling software is used. Furthermore, seven deterministic non-physical empirical are assessed and compared. The five deterministic non-physical empirical methods have resulted in peak flow values between 32,940.26m3/s and 22,404.28m3/s for overtopping and two of them were 20,224.88m3/s and 11,799.72m3/s for piping modes of failure, respectively. Of the failures, overtopping failure generates the biggest peak discharge, which is 32,940.26 m3/sec using Von Thun & Gillete Equation. Not only the peak discharge results from this equation but also the flood reach to Guder Town is short for it, which is 1.35Hrs. Guder town is the focus of the study since most of the population at risk was situated in this town. Based on this flood inundation map the possible people at risk (PAR) were found to be 9,425 people. And based on the DSO-99-06 manual the Loss of life is found to be 4,713. The study found the dam breach and its corresponding flooding to be catastrophic in Loss of Life therefore its categorized as a very high-hazard dam based on ENTRO guidelines. The author also checks for the level of reduction if EAP is introduced, and the Loss of life is found to be only 10 people. Therefore, the study highlighted the provision of EAP to this dam will reduce the possible loss of life in case the dam fails. Taking this dam as a case study recommended the need for a national-level regulatory framework for different levels of dam life (Design, Construction, Filling, Operation, and Decommissioning).

Keywords: HEC-RAS; HAEC; dam breach; Loss of Life; People at Risk; Guder Town; EAP, DSO, Overtopping, Piping

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LISTS OF ACRONYMS

ANCOLD	: Australian National Committee on Large Dams
ASDSO	: Association of State Dam Safety Officials
CSA	: Central Statistical Agency
DBD	: Design Bid Build
DEM	: Digital Elevation Model
DSO	: Dam Safety Office
DSS-WISE	: Decision Support System for Water Infrastructural Security
DWE	: Diffusion Wave Equation
EAP	: Emergency Action Plan
EBCS	: Ethiopian Building Code Standard
ECDSCo	: Ethiopian Construction Design and Supervision Works Corporation
ENTRO	: Eastern Nile Technical Regional Office
LULC	: Land Use and Land Cover
FEMA	: Federal Emergency Management Agency
FERC	: Federal Energy Regulatory Commission
FRL	: Full Reservoir Level
GIS	: Geographic Information System
HAEC	: Homicho Ammunition Engineering Complex
HEC-DSS	: Hydrologic Engineering Center Data Storage System
HEC-RAS	: Hydrologic Engineering Center River Analysis System
ICOLD	: International Commission on Large Dams
IDF	: Intensity-Duration-Frequency Curves
MCM	: Million Cubic Meters
MDE	: Maryland Department of The Environment
MWL	: Maximum Water Level
PAR	: Population At Risk
PCC	: Potential Consequences Classification
PMF	: Probable Maximum Flood
S. A	: Storage Area
SA/2D	: Storage Area 2d Connection
SCS	: Soil Conservation Service
SWE	: Shallow Water Equation
TOR	: Terms of Reference
UH	: Unit Hydrograph
USACE	: United States Army Corps of Engineers
USBR	: United States Bureau of Reclamation
WDE	: Washington State Department of Ecology

1. INTRODUCTION

1.1. Background

Dams are hydraulic structures that impound water in a reservoir behind their upstream side for a variety of purposes, including the production of hydropower, water supply, irrigation, navigation, and transportation. Despite the many benefits of dams, there is still a chance that they could fail. If dams are not built, managed, and maintained correctly, communities and properties downstream may be at risk.

A dam cannot completely remove the risk of flooding, even if it is intended to regulate flooding. Dam breaches are more likely when there are design defects, aging structures, insufficient maintenance, and a number of other problems. (FEMA, 2018).

Therefore, a dam breach study is typically carried out to establish the ultimate flow from a fictitious dam breach under such circumstances. The result is a breach hydrograph, which is routed across the river system to ascertain the flood arrival time, peak flow, and depth of flow at downstream points. The breach hydrograph is caused by a flood wave immediately downstream of the dam. The possible effects of a dam breach are estimated via mapping of inundation regions, which are places that are flooded by the flood wave. This mapping also helps to check the categorization and prepare for emergencies.

The "Danger Reach" refers to the region below a dam that would flood if the dam failed. As seen in Figure 1.1, the depth of flooding caused by a dam failure is typically significantly greater than the typical floodplain. An engineer creates maps of the risk reach, often known as "inundation maps," based on hydrologic and hydraulic calculations and the terrain of the impacted area. A danger reaches analysis will typically consider a variety of failure scenarios, including "Sunny Day" (dam fails on a sunny day, i.e., not during a storm), "Brim-Up" (reservoir fills to the very top of the dam), and during incidents like the 100-year storm or Probable Maximum Flood. (MDE, 2023)

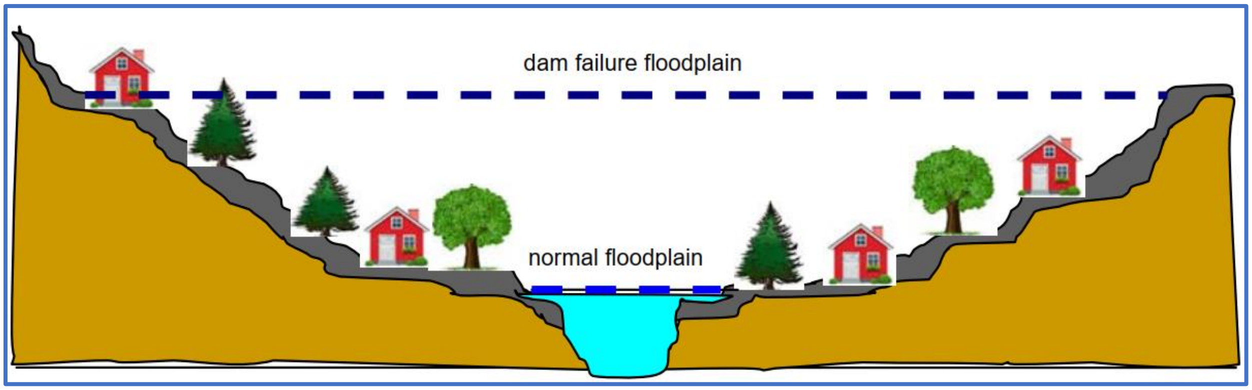


Figure 1. 1: Dam Breach Flood Plain (MDE, 2023)

Determining the outflow from the breach and routing the computed flood wave across the downstream flood plain are the two problems that need to be solved in order to model a prospective embankment-breaching failure. (Chinnarasri, Chaiyuth; J, S, 2004).

After the dam breach analysis has been done the hazard classification dam will be proceeded based on the fatalities in the flood inundation area, economic loss in the flood area, and environmental and historic place loss in the flood area.

This paper examines a rock-fill dam analysis incase Fato dam fails. As a case study, Fato Dam is located about 150km from Addis Ababa. It is accessible via Guder. The boundary mainly falls into two Woreda administrative boundaries namely Guder and Dere/Tikur Inchini.

Given the magnitude of many communities and the amount of the property found downstream of the dam, the Fato dam is anticipated to be categorized as a high-hazard structure. This classification necessitates modeling the dam breach and studying how the downstream floodplain is affected by the dam breach outflow hydrograph. For the assessment of the dam hazard class based on the ENTRO Guideline, factors such as population density, infrastructure, and historical sites within the flood plain have also been considered in addition to hydrologic inputs.

1.2. Statement of the Problem

Even if the dam is not explicitly constructed for flood mitigation, it can still lower the risk of flooding in communities downstream by temporarily ponding floods and reducing reported peak flood volumes in exposed low-lying areas. However, holding back water behind a dam also poses a risk to areas downstream since a dam failure might result in an uncontrolled release of the reservoir pool, which could produce a peak flow discharge that is significantly greater than any probable natural flood event. Dam failure may result from several factors, including those related to hydrology, hydraulics, geology, seismic activity, mechanical systems, and operations. (FEMA, July 2013)

The inability to pass flood flows safely is one of the most frequent reasons for dam failures. Hydrologic conditions can result in a variety of failures, including abrupt failure with whole breaching or collapse of the dam and gradual failure with increasing erosion and partial breaching. (FEMA, July 2013).

Dam failures may occur due to a variety of causes such as significant hydrologic and no hydrologic events. If a dam breach occurs, an uncontrolled release of water impounded behind the structure will cause flooding in the downstream area and affect the population and damage the economic resources. Nowadays, many embankment dams are existing, are under construction, are planned to be constructed in Ethiopia. Therefore, in addition to proper design and construction methods, dam failure and risk analysis must be undertaken for the sake of safety. Fato (Upper Guder) dam, a rock-fill dam consisting of an impervious clay core is now under construction. This dam is proposed to impound the water within the reservoir of capacity 57.71 -million-meter cube at its full pool level. The main purpose of this dam is to retain, control, and release this water for downstream huge irrigation fields without disturbing the downstream natural river flow system and mini hydropower as an expansion project. Unlike its benefits, its safety also should be considered for the consequence in case failure may exist during and after the construction time. Downstream of Fato Dam, there is a huge settlement of people 18km inside Guder town and around 25km large ammunition factory called Homecho Ammunition Engineering Complex (HAEC). The river over which the Fato dam is under construction passes through Guder town and HAEC which makes the town prone to flood danger from the uncontrolled release of water in case the dam fails.

Thus, for the Fato rock-fill dam, two dam breach scenarios are analyzed to the effect of the outflow breach hydrographs on the downstream floodplains and classify the hazard class based on the flood inundation area fatalities, economic loss, environmental and historical place damages. These scenarios are hydrologic (overtopping) and non-hydrologic (Sunny Day) or Piping failure.

The problem that needs to be solved in this research is the proper modeling of the Fato Dam breach and the effect of the breach outflow flood on the downstream flood plain and defining the hazard class of the dam so that emergency action be prepared for it. Therefore, for the proposed dam breach modeling, prediction of breach outflow hydrograph and computer simulation to evaluate the dam failure and its impact on the downstream area, which is the first phase of dam breach analysis, has been discussed for dam failure scenarios. Finally, classify the dam hazard class using ENTRO guidelines.

1.3. Objective

The main objective of the research is to model the Fato Rock-fill Dam breach and analyze the risk level (consequence) due to the dam breach/failure on the downstream floodplain and classify the hazard class of the dam based on the ENTRO guideline.

The specific objectives are:

- To identify potential mode of failure
- To do dam breach analysis
- To prepare floodplain inundation maps
- To compute the possible fatalities due to the dam breach and
- To define the hazard class of the dam

1.4. Significance of the study

In general, dam safety, hazard mitigation, consequence evaluation, and emergency management, including developing EAPs, can all be accomplished using dam breach inundation modeling and conduct mapping. Conducting pre-event analyses of flood maps and models for Fato dam breaches and its corresponding hazard class defining has implications for the following works:

- Choice of the magnitude of inflow design flood
- Requirement of dam instrumentation
- Deciding degree of inspection and maintenance

- Deciding degree of EAP necessity
- Frequency of dam safety risk assessment
- Define degree of Seismic Hazard Assessment

1.5. Research questions

- What are the main contributing factors for the Fato dam failure?
- How can the risk and Fato dam failure mitigated?
- What is the consequence of Fato dam failure with emergency action plan (EAP) and without any?
- What is the velocity of the released water from Fato dam Breach?
- What is the depth of flood from Fato dam breach?
- What is the future trend of dam safety in Ethiopian context?

1.6. Scope of the study

This study covers estimation of dam breach parameters and breach outflow hydrograph for 5 scenarios and then conduct inundation modelling and mapping. After mapping the inundation area with the worst situation of all the conditions then digitized the houses that fall within the inundation area. Based on these digitized houses computed the people at risk. Finally computed the possible loss of life due to Fato dam breach. Based on the computed loss of life then define the hazard class of the dam using ENTRO classification.

1.7. Limitations of the study

In this study while computing the people at risk it need door to door or updated population census data but the data from 2007 is outdated and from the in-person observation of the Guder town the population is far greater than the 2007 one. Therefore, to capture exact PAR door to door socioeconomic data need to be collected with spatial location to extract the people fall within the flood inundation area. But due to financial demand to do so the researcher goes with the estimation of the PAR simply following the guideline multipliers for the house's occupants within the inundation area.

1.8. Organization of the Research

This research is organized into 6 chapters. Chapter One deals with the general introduction (i.e., background, statement of the problem, significance of the study, and objectives of the study. Chapter two discusses the literature review on embankment dam breach modeling methods, breach characteristics, parameter estimation, and hydraulic modeling for the analysis of downstream routing the outflow hydrograph from dam breach, different researchers' literature on dam breach modeling, and outflow hydrograph routing, mapping flood inundation area, hazard classification of dams, etc. Chapter three discusses the methodology and steps to be taken in the research to process the study well. In this section, the study area, data collection, hydraulic model development, etc. has been discussed. In Chapter Four, dam breach parameter estimation model setups and People at Risk (PAR) have been discussed. In this part, the dam breach parameters, breach development time, and peak breach outflow discharges have been estimated, and the river (Fato) has been represented in the hydraulic model (HEC-RAS). Chapter five covers the results of the modeling and the discussions on the output results like flood inundation map both for the velocity and depth distribution and the corresponding PAR and Loss of Life. Chapter Six covers the conclusion from the results and makes recommendations.

2. LITERATURE REVIEW

2.1. Introduction

This chapter reviews a variety of literature pieces, including recent scientific journals, guidelines, and books, in relation to embankment dam breach modeling, breach parameter estimation methods, and dam hazard classification methods. It also considers personal studies conducted by other researchers in dam hazard classification field of study.

It is consensual that the higher the dam the more destructive is the generated flood wave associated to its breaching, as more potential energy is involved. Also, the larger the reservoir capacity, the more destructive is the generated wave, as the hydrograph will involve a much larger volume and therefore, the duration of the flood will be longer for larger reservoirs. (Melo, 2015)

Several authors have analyzed historical data of breached dams and attempted to setup simple equations to assess the peak discharge produced at the dam section in a breach event based on either dam height, reservoir volume, or a combination of both. As many real dam breach data were used, the resulting formulae encompass indirectly the time and mode of breach development that occurred in each accident. These expressions have, inevitably, a considerable margin of incertitude and are influenced by the specificities of the universe of dams considered, such as type and failure mode involved, so one shall adopt conservative criteria regarding the assumptions necessary to apply these equations. (Melo, 2015)

2.2. History of Dam Breach

A dam failure is a catastrophic type of failure that is characterized by the sudden, rapid, and uncontrolled discharge of impounded water coupled with held silt and debris that erode and pick up more material as they go. (Souza, Sanjay Pandit, & Prakash Chanekar, 2019)

Normally, the risk posed to lives and property downstream of dams is taken into consideration when discussing dam failure. This is typically true for big dams built immediately above significant population centers. These have the power to result in catastrophic losses. Failure of a dam can result in social effects, as well as loss of life, property damage, cultural and historic losses, and environmental losses. (Nyoni K., 2013).

The history of water defense and water retention structures coexists with the history of their failures. Across the course of several centuries, countless dams have been built across the world. However, due to excessive river flows, storm surges at sea, etc., hundreds of dams have fallen, and every year, numerous dikes fail. often leading to catastrophic consequences. The Banqiao Dam and the Shimantan Dam catastrophically failed in August 1975 in Henan Province, China, as a result of the overtopping brought on by excessive rainfall, making it by far the greatest dam disaster in history. Flooding claimed the lives of almost 85,000 people, and many more perished from ensuing illnesses and famine. Millions of people also lost their houses. (Qing, Dai, 1998).

The Teton Dam project of the Bureau of Reclamation in Idaho experienced a dam breach on June 5, 1976, which was one of the most widely reported dam failures in recent memory. Three miles northeast of Newdale, Idaho, on the Teton River, the Teton Dam failed during the initial filling of the reservoir. About 130 feet below the crest, a significant leak in the dam's right abutment caused the embankment to wash away and the dam to burst. 11 fatalities were reported, and $400 million worth of property was damaged. (Barnes, 1992)

Figure 2. 1: Teton Dam Breach Fully Developed

On September 12, 2019, Platte Dam was overtopped and burst as a result of a severe precipitation event on saturated ground that took place from September 11 to September 12. Due to the tremendous flash floods caused by this downpour, some areas had to be evacuated, and numerous water rescues had to be performed. Numerous sites experienced significant, record-breaking flooding as the water entered the river system. Water flooded a lot of buildings. Additionally, there was extensive and widespread road damage in the area. Notably, floods caused Interstate 90 between Sioux Falls and Mitchell, SD, to be blocked for a lengthy period of time. (ASDSO, 2020)

Figure 2. 2: Platte Dam Breached Drone Footage **(ASDSO, 2020)**

The Spencer Dam on the Niobrara River in northern Nebraska, USA, suddenly collapsed in the early hours of March 14, 2019, during a significant flood and ice rush on the river. The dam's and its spillways' capacity were severely surpassed by the surge of water and ice. The dam failure flood wave completely demolished the buildings right downstream of the dam. It is also obvious, though, given the position, that if the dam hadn't been there, the ice run would have swept them away. The loss of life and property that occurred directly below the dam caused the effects of the dam's failure to be felt most keenly in the area immediately downstream from the structure. (ASDSO, 2020).

Figure 2. 3: Spencer Dam Breached Photo **(ASDSO, 2020)**

Tailings dam B-I at Vale S.A.'s Córrego do Feijo Iron Ore Mine ("Dam I"), located 9 kilometers (km) northeast of Brumadinho, in the state of Minas Gerais, Brazil, suddenly failed at around 12:28 p.m. local time on January 25, 2019, causing a catastrophic mudflow that sped downriver. A crucial confluence of ongoing internal strains caused by creep and a strength reduction due to loss of suction in the unsaturated zone brought on by the cumulative rainfall since the end of tailings deposition, including the intense rainfall toward the end of 2018, led to a sudden strength loss and the subsequent failure of the marginally stable dam. This came after several years of rising rainfall after the cessation of tailings deposition in July 2016. On January 25, 2019, the observed failure occurred as a result of severe internal strains and strength loss in the unsaturated zone. The tiny deformations on the dam that were seen in the year before the failure match the calculated pre-failure strains and deformations from internal creep. (ASDSO, 2020).

Figure 2. 4: Feijão Iron Ore Mine (“Dam I”) Breaching Photo **(ASDSO, 2020)**

The 1920s project's design and construction, as well as the duration of the project's existence up until its failure in May 2020, all played a part in the interactions that led to the collapse of Edenville Dam. Inaccurate generalizations were made regarding the integrity of the Edenville Dam embankments and the capacity of the spillways during the project's nearly 100-year existence before it failed. Decisions and judgments were made that ultimately contributed to the failure or failed to avoid the failure because it was not realized that a non-extreme rainfall event could cause the lake to rise by several feet and rise to the embankment crest. (France, John W., 2022).

Figure 2. 5: Edenville Dam Developing to Full Breach Photo **(ASDSO, 2020)**

On December 14, 1963, around 11:15 a.m., the caretaker of the Baldwin Hills Dam and Reservoir heard running water in the spillway discharge pipe at the reservoir and then saw water flowing freely from the drains beneath the asphalt-paved reservoir bottom. The operating system engineer and the caretaker opened discharge lines to lower the reservoir level in 30 minutes. (1) would have required almost 24 hours to completely empty the reservoir. The Department of Water Resources requested that the police implement an evacuation plan because they anticipated a threat. Around 1,600 people left the area around 3:20 PM. (Barnes, 1992)

Muddy water was detected coming downstream from the east abutment of the dam not long after the reservoir started to empty. Ineffective attempts to control the outflow included cleaning the storm drain system's inlets of debris, inspecting the inspection chamber beneath the reservoir, and hanging by ropes from the dam's upstream face. At 3:38 p.m., a powerful surge of water swept mud and debris through the dam's lower face and spilled down the steep slope leading to a nearby residential street that was 900 feet distant. The reservoir ran dry in just over an hour. Only then could a breach in the asphaltic lining spanning the reservoir's whole bottom be detected. (Barnes, 1992)

In the subsequent flood, 41 homes were destroyed, five people killed, and almost 1,000 more were damaged. Had it not been for LADWP’S recognition of the danger, more lives would have been lost. (Barnes, 1992)

In the case of Africa, specifically East Africa, according to the Nile Basin Capacity Building Network for River Engineering (Kamal Eldin Bashar, 2005) team assessment on an inventory and performances of micro dams in Sudan, Uganda, and Ethiopia, several micro dams constructed in three countries were analyzed. As shown from their assessment results, most of the micro dams were constructed for water supply and small numbers of these micro dams were constructed to recharge groundwater. Among some of the causes of failure for these micro dams, most of them faced severe siltation problems and some failed due to spillway failure. Seepage of water beneath the dam or from the reservoir was also a problem.

Traditional small-scale irrigation systems have been used in Ethiopia for millennia, especially in the eastern, central, and northwest regions of the nation (Kamal Eldin Bashar, 2005) for water supply and irrigation purpose. The diversion constructions were made of wood, stones, grass, and earth, according to the Nile Basin Capacity Building Network for River Engineering study. They must be rebuilt every year since they have frequently been washed away during large river flows. Following the country's severe droughts in 1973–74 and 1984–88, a lot of focus was placed on modern small-scale irrigation using micro-dams.

According to a 2003 report from the Ministry of Water Resources technical team on site visits to micro and medium dams built in the Amhara and Tigray regions between the 1970s and 1990s, the most frequent reasons for dam failure were overtopping due to insufficient spillway capacity, flooding estimation issues, seepage through the foundation, abutments, and reservoir area, site selection issues, cracking or structural failure-geotechnical issues, sedimentation design issues, and a lack of watershed. (Kamal Eldin Bashar, 2005)

2.3. Dam Breach Mechanism and Breach Parameters Estimation

Estimating the dam breach parameters for dam breach modeling relating to the geometry and timing (e.g., width, depth, shape, and time of failure) of the breach development is a crucial step in generating a dam breach hydrograph for a specific dam. The storage in the impoundment at the time of the breach, reservoir inflow at the time of the breach, the size of the dam, and most crucially, the type of dam material and/or expected failure mode, all affect the shape of the peak breach outflow hydrograph.

For use in dam breach studies, there are several approaches for determining the breach parameters. Since each dam's breach parameters must be chosen individually, guidance is given explaining the techniques that experts in dam safety now use. Although there are many different reasons why embankment dams can breach, these failures are most frequently represented as overtopping or piping failures. (FEMA, July 2013)

2.3.1. Failure of Embankment Dams due to Overtopping

Typically, head cutting at the downstream toe of embankment dams (earthen/rock-fill) leads to upstream progress of the erosion until it reaches the dam crest and reservoir surface. Up until the breach reaches its final size, as shown in Figure 2.6, downcutting of the embankment and lateral erosion take place once the reservoir is connected to the breach as it progresses.

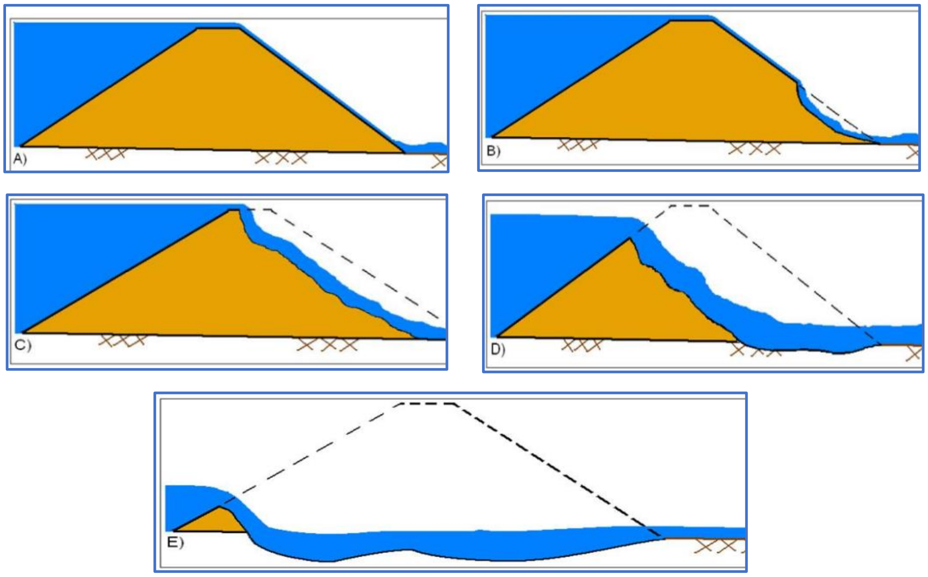


Figure 2. 6: Example Breach Process for Overtopping Failure **(USACE-HEC, 2014)**

When erosion spreads across the width of the dam crest, the breach is regarded as having started. The breach starts at the top of the dam crest and grows to its full size from there. The breach should be represented as beginning at the maximum section commonly found near the centerline of the downstream main channel if there is no practical reason to assume the embankment will fail there. Figure 2.7 below shows a generalized trapezoidal breach progression.

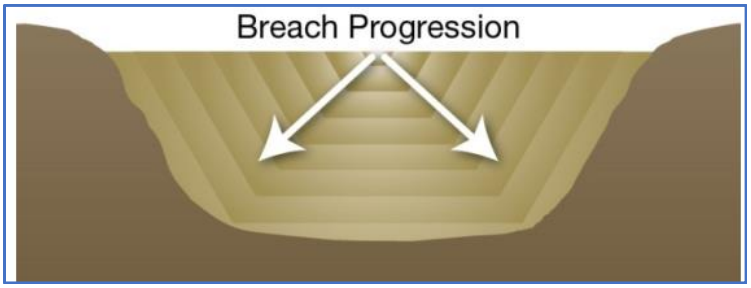


Figure 2. 7: Overtopping trapezoidal breach progression, source: **(Gee, 2010)**

The breach may stop growing when the reservoir has emptied and there is no more water to erode the dam, or the dam has completely eroded to the bottom of the reservoir or has reached bedrock (Gee, 2010). There are now three types of progression methods: linear, sine wave, and user curve.

- From the beginning to the end of the development time, the breach grows in equal increments of depth and width when using the **Linear method**.
- The **Sine Wave** method makes the breach grow more slowly as it approaches its maximum size and faster in the early stages of its development. In accordance with the sine wave's initial quarter cycle, the speed changes over the course of development.
- The **User Curve** method allows the modeler to specify a pattern for the breach growth by defining the relationship between the percent of the development time versus the percent of the maximum breach size.

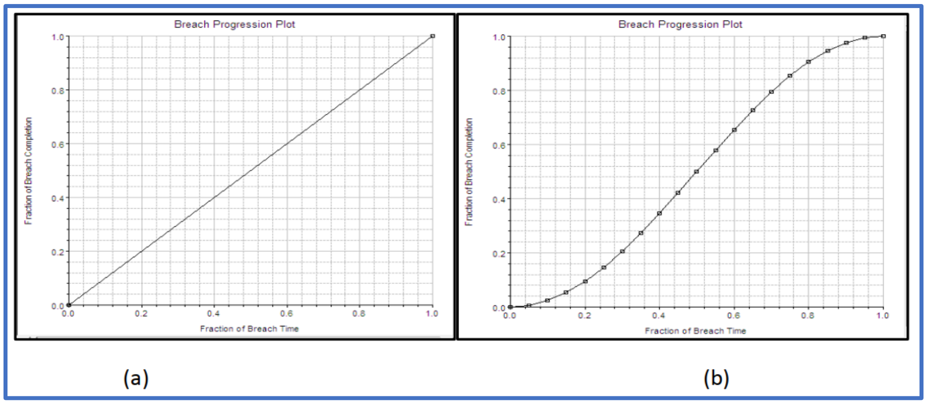


Figure 2. 8: Breach Progression, (a) linear, (b) sine wave, USACE HEC-RAS (6.3.1)

2.3.2. Piping / Internal Erosion Failures of Embankment Dams

Piping and internal erosion occur when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam, leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. Water flows through the embankment and will appear muddy as erosion increases. Once the erosion reaches the reservoir, the piping hole can enlarge and cause the dam to crest to collapse. Figure 2.9 below indicates the progressive steps the piping failure of the embankment dam passes through to reach full breach development.

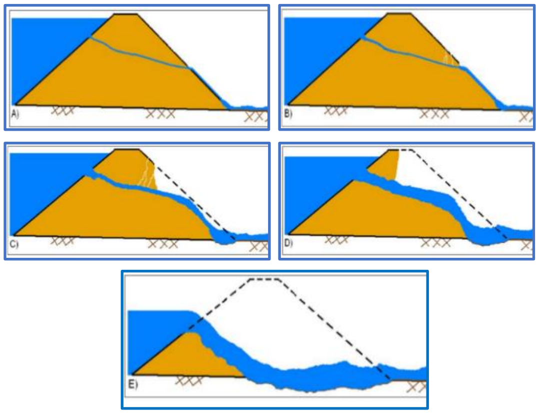


Figure 2. 9: Breach Process for Piping Mode of Failure **(USACE-HEC, 2014)**

Piping failures are typically modeled in two phases, before and after the dam crest collapses. Water flows through the piping hole and is modeled as orifice flow before the dam crest collapses and as weir flow after the dam crest collapses.

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

Formation Time: The period from the first breaching of the dam's upstream face (also known as the "breach initiation") until the breach reaches its full geometry is known as the "**breach formation time**".

Breach depth (also known as **breach height**): The breach depth is the vertical distance between the breach's starting point and the dam's invert.

Breach width: The breach width, which is commonly calculated at the vertical center of the breach, is the average of the final breach width.

Breach side slope factor – Measured as X horizontal to 1 vertical, the breach side slope represents the angle of the breach sides. (XH: 1V) (Wahl T. L., 1998)

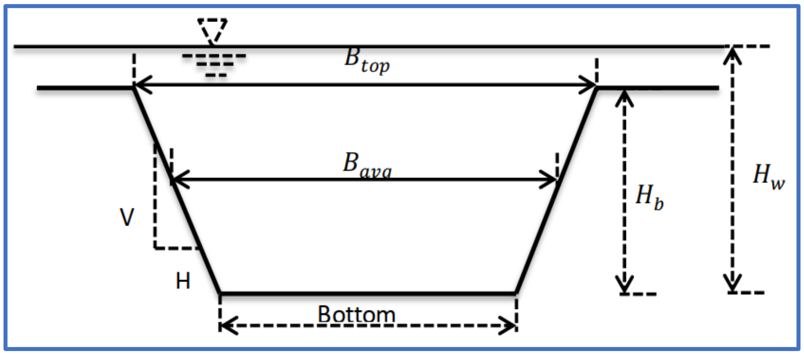


Figure 2. 10: Description of the dam breach parameters

2.4. Breach Parameter Estimation Methods

According to the dam breach modeling analysis methods overview made by Tony L. Wahl (Wahl, 2010), three principal strategies for dam-breach flood modeling have emerged since the 1970s. Since the 1970s, three main approaches to dam-breach flood modeling have been developed. The first tactic was to estimate the breach output hydrograph directly, and then to transport that flood downstream using one of the existing routing models (Wahl, 2010), so that flooding consequences could be determined. The second method involved parameterizing the breach in order to describe its development over time in relatively simple mathematical terms. This allowed for the determination of the breach outflow hydrograph by integrating the description of the development of the breach with a weir equation or other suitable model for simulating the hydraulic performance of the breach opening. Typical breach parameters determined were the maximum breach size, and rate of breach development (or total time needed for full breach development). In this second method, the breach outflow hydrograph was determined in the routing model, but the breach parameters may be established by a variety of methods external to the flood routing model. The maximum breach size and pace of breach development (or the total amount of time required for full breach development) frequently determine breach parameters. Early flood routing models that adopted this strategy operate independently from them, with the routing model taking the hydrograph produced by the breach as input. The integration of breach modeling and flood routing capabilities into a single model is currently being worked on.

2.4.1. Comparative Analysis

A given dam of interest is compared using this method to those in a database of thoroughly documented volumes and to a list of similarly sized dams that have failed. The dam under analysis is then directly affected by the dam breach parameters and peak flows reported from the failure case histories of dams with an equivalent configuration. The proper breach parameters or peak outflows may be determined by comparison if the dam under consideration is substantially comparable in size and structure to a dam that failed, and the failure is properly recorded.

2.4.2. Physically Based Erosion Models

Physically based models use generally recognized relationships based on physical principles to construct the framework of a model (also known as "process" or "causal" models). After that, the model tries to resolve those relationships given an input. This is a fairly straightforward idea, but when the input is changing over time, it can get very complicated. In the case of a dam breach study, as the dam erodes and the reservoir empties, both the input and physical constraints change over time.

These techniques make use of an erosion model based on hydraulic, sediment transport, and soil mechanics concepts to predict the development of an embankment breach and the breach outflows that will result.

Numerous physically based, numerical dam breach models have been developed since the 1960s. The first breach model was proposed by Cristofano in 1965 (Singh, 1965), which helped to pave the way for the creation of physically based models such as BREACH (NWS, 1988), BEED (1985), and the Dam Break Forecasting Model (DAMBRK) in 1977.

2.4.3. Predicting Breach Parameters from Case Study Data

The relations put forth by earlier researchers for predicting breach parameters (such as geometry and timing of formation) using case study data are summarized in Table 2.1. Johnson and Ille's (1976) classification of failure shapes for earth, gravity, and arch dams was one of the earlier contributions. As a breach progressed in an earth dam, the shape of the breach was said to change from triangular to trapezoidal. The bulk of earth dam breaches are classified in the literature as trapezoidal. Table 2.1 lists the most popular parametric regression equations that were created using data from case studies of previous dam disasters.

Table 2. 1: Parametric Regression Equations for Predicting Breach Parameters (Adapted from DSO- 98-004, USBR 1998) **(Wahl T. L., 1998)**

Reference	Number of Case Studies	Relations Proposed (S.I. units, meters, m^3/s, hours)
Johnson and Illes (1976)		$0.5hd \leq B \leq 3hd$ for earthfill dams
Singh and Snorrason (1982, 1984)	20	$2hd \leq B \leq 5hd$ $0.15\ m \leq dovtop \leq 0.61\ m$ $0.25\ hr. \leq tf \leq 1.0\ hr.$
MacDonald and Langridge-Monopolis (1984)	42	<u>Earthfill dams:</u> $Ver = 0.0261(Vout*hw)^{0.769}$ [best-fit] $tf = 0.0179(Ver)^{0.364}$ [upper envelope] <u>Non-earthfill dams:</u> $Ver = 0.00348(Vout*hw)^{0.852}$ [best fit]
FERC (1987)		B is normally 2-4 times hd B can range from 1-5 times hd Z = 0.25 to 1.0 [engineered, compacted dams] Z = 1 to 2 [non-engineered, slag or refuse dams] tf = 0.1-1 hours [engineered, compacted earth dam] tf = 0.1-0.5 hours [non-engineered, poorly compacted]
Froehlich (1987)	43	$B=0.47Ko\ (\boldsymbol{S}^*)^{\mathbf{0.25}}$ Ko = 1.4 overtopping; 1.0 otherwise $Z = 0.75\, \mathbf{h_w} * \ (\mathbf{h_w}^*)^{\mathbf{1.57}}\ (\mathbf{W}^*)^{\mathbf{0.73}}$ Kc = 0.6 with corewall; 1.0 without a corewall $tf^* = 79(S^*)^{0.47}$
Reclamation (1988)		$B = (3) * hw$ $tf = (0.011) * B$
Singh and Scarlatos (1988)	52	Breach geometry and time of failure tendencies Btop/Bbottom averages 1.29

Von Thun and Gillette (1990)	57	B, Z, tf guidance (see discussion)
Dewey and Gillette (1993)	57	Breach initiation model; B, Z, tf guidance
Froehlich (1995b)	63	B = 01803 Ko $\boldsymbol{V_w}^{\mathbf{0.32}}\boldsymbol{h_b}^{\mathbf{0.19}}$ t_f = 0.00254$\boldsymbol{V_w}^{\mathbf{0.53}}\boldsymbol{h_b}^{\mathbf{-0.19}}$ Ko= 1.4 for overtopping; 1.0 otherwise

These empirical regression equations were developed to predict the average breach width, breach depth, and time-of-failure or formation time. In the form of feasible ranges of values for breach width, side slopes, and development time, multiple federal agencies have released guidelines. These guidelines should be used as a minimum and maximum bound for estimating breach parameters.

Table 2. 2: Ranges of Possible Values for Breach Characteristics of Different Agencies **(USACE-HEC, 2014)**

Dam Type	Average Breach Width (B_w)	Horizontal Component of Breach Side Slope (H) (H: V)	Failure Time, t_f (hours)	Agency
Earthen/ Rockfill	(0.5 to 3.0) x HD (1.0 to 5.0) x HD (2.0 to 5.0) x HD (0.5 to 5.0) x HD*	0 to 1.0 0 to 1.0 0 to 1.0 (slightly larger) 0 to 1.0	0.5 to 4.0 0.1 to 1.0 0.1 to 1.0 0.1 to 4.0*	USACE 1980 FERC NWS USACE 2007
Concrete Gravity	Multiple Monoliths Usually 0.5 L Usually 0.5 L Multiple Monoliths	Vertical Vertical Vertical Vertical	0.1 to 0.5 0.1 to 0.3 0.1 to 0.2 0.1 to 0.5	USACE 1980 FERC NWS USACE 2007
Concrete Arc	Entire Dam Entire Dam (0.8 x L) to L (0.8 x L) to L	Valley wall slope 0 to valley walls 0 to valley walls 0 to valley walls		USACE 1980 FERC NWS USACE 2007
Slag/Refuse	(0.8 x L) to L (0.8 x L) to L	1.0 to 2.0	0.1 to 0.3 <0.1	FERC NWS
*Note: Dams that have very large volumes of water, and have long dam crest lengths, will continue to erode for long durations (i.e., as long as a significant amount of water is flowing through the breach) and may therefore have longer breach widths and times than what is shown m Table 2.1. HD= height of the dam; L = length of the dam crest; FERC - Federal Energy Regulatory Commission; NWS - National Weather Service				

Dams that have a very large volume of water and have long dam crest length will continue to erode for long durations (i.e., as long as a significant amount of water is flowing through the breach) and may therefore have longer breach width and times than what is shown in the table 2.2. According to the State of Colorado, Guidelines for Dam Breach Analysis, The MacDonald & Langridge-

Monopolis (1984) utilizes 42 data sets (predominantly earth-fill dams, earth dams with clay core, and rock-fill dams) to develop a relationship between the volume of water coming out of the dam and the height of water above the dam breach invert.

The ranges are:

- ✓ Height of the dams:4.27- 92.96 meters (with 76 %<30 meters, and 57 % <15 meters)
- ✓ Breach outflow volume:0.0037-660.0 m3*10^6 (with 79%<25.0m3*10^6, and 69 % <15.m3*10^6) (USACE-HEC, 2014)

Froehlich (1987), MacDonald and Langridge-Monopolis (1984), and other researchers' data were used by Von Thun and Gillette (1990) and Dewey and Gillette (1993) to create guidelines for calculating breach side slopes, breach breadth at mid-height, and time to failure. In contrast to dams with cohesive shells or very wide cohesive cores, where slopes of 1:2 or 1:3 (H: V) may be more acceptable, they suggested that breach side slopes be considered to be 1:1.

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

with hw being the depth of water at the dam at the time of failure, and C_b a function of reservoir storage as follows:

Table 2. 3: Values of coefficient **(Von and Gillette, 1990)**

Reservoir Size (cubic meters)	c_b (meters)	Reservoir Size (acre-feet)	c_b (feet)
$< 1.23*10^6$	6.1	< 1,000	20
$1.23*10^6$ — $6.17*10^6$	18.3	1,000-5.000	60
$6.17*10^6$- $1.23*10^7$	42.7	5.000 - 10.000	140
$> 1.23*10^7$	54.9	> 10,000	180

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

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

$$\mathrm{T_f} = 0.015\mathrm{h_w} \text{ (Easly erodable)}$$

Where: Tf – breach formation time

Hw – depth of water above the bottom of the breach

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

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

Today's practice in the dam breach modeling disciplines makes extensive use of the more modern Froehlich empirical technique (Froehlich, 2008) for dam breach analysis. It is dependent only on the volume of the reservoir, the height of the breach, and the assumed breach side slope. 74 earthen, zoned earthen, earthen with a core wall (i.e., clay), and rock-fill data sets were used in Froehlich's (2008) technique to create a set of equations for predicting average breach width, side slopes, and failure time. His data indicate:

- ✓ Height of the dams:3.05-92.96 meters (with 93 %<30 meters, and 81 % <15 meters)
- ✓ Breach outflow volume:0.0139-660.0 m3*10^6 (with 86%<25.0m3*10^6, and 82 % <15.m3*10^6) (USACE-HEC, 2014)

In 2011, three Egyptian researchers (Samir, 2011), modeled the prediction of breach formation through the Aswan Rock-fill High Dam. In their study, they assessed the breaching due to overtopping, numerically. A one-dimensional model (HR-BREACH) created by HR Wallingford was used for study. This model can simulate the overtopping or piping failure of homogeneous or composite embankment dams. It is based on the principles of hydraulics and sediment movement and considers the soil mechanics concept throughout the breaching process.

2.4.4. Predictor Regression Equations

Based on actual case study data, predictor regression equations are empirically created equations that are used to estimate peak discharge. In order to estimate an appropriate outflow hydrograph shape, these equations are applied.

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

Table 2. 4: Predictor Regression Equations for Prediction of Peak Breach Flow **(FEMA, July 2013)**

Reference	Case Studies	Relations Proposed	Notes
Babb and Mermel (1968)	>600 incidents		Many cases not well documented
Kirkpatrick (1977)	16 (plus 5 hypothetical failures)	$Q_p = f(h_w)$	
Soil Conservation Service (SCS) (1981)	13	$Qp = f(hw)$	
Hagen (1982)	6	$Qp = f(hw*S)$	
Reclamation (1982)	21	$Qp = f(hw)$	
Graham (1983)	6		Dams with large storage -to- height ratios

Singh and Snorrason (1982, 1984)	20 real failures and 8 simulated failures		Q_p relations based on simulations
Graham (undated)	19	$Q_p = f\,(h_w, S)$	
MacDonald and Langridge- Monopolis (1984)	42	$V_{er} = f\,(V_{out}{}^* h_w)$ $t_f = f(V_{er})$ $Q_p = f(V_{out}{}^* h_w)$	
Costa (1985)	31Constructed dams	$Q_p = f(hd)$ $Q_p = f(S)$ $Q_p = f(hd^* S)$	
Evans (1986)		$Q_p = f(V_w)$	
FERC (1987)		Guidance for B, Z, t_f	
Reclamation (1988)		B, t_f guidance	
Singh and Scarlatos (1988)	52	Guidance for B, Z, t_f	
Von Thun and Gillette (1990)	57	Z guidance $B = f\,(h_w, S)$ $t_f = f\,(h_w$, erosion resistance)	
Froehlich (1995b)	63	B, Z, t_f relations	
Froehlich (1995a)	22	$Q_p = f\,(V_w, h_w)$	
Froehlich (2008)	74	$Q_p = 3.1 B_{avg} H^{1.5} \left(\frac{\Upsilon}{\Upsilon + T_f \sqrt{H_w}}\right)^3$	

Simulating embankment dam breaches and the ensuing floods is essential for identifying and minimizing the risks associated with future dam failures. Accurate estimation of flooding levels and the time of flood wave arrival at a specific area are necessary for the establishment of hazard classification. Details of the breaching procedure have little impact on the outcome if population centers are far downstream of a dam; instead, routing factors such as travel time and attenuation take center stage. The proximity of population centers to dams, however, makes it increasingly important to the analysis to accurately forecast the breach parameters (such as breach breadth, depth, and rate of development). Increased conservatism with related higher costs may be necessary if breach parameters cannot be foreseen with a fair degree of certainty. This proposal wants to examine existing empirical procedures and a numerical model used to predict breach parameters, reviews new technologies relevant to dam breaches, and outline a program for the development of an improved numerical model for the simulation of embankment dam breach events.

2.5. Modeling Tools for Dam Break

Predicting the dam breach hydrograph and routing that hydrograph downstream are both necessary steps in performing a dam breach model. (FEMA, July 2013). Dam breach modeling can be done using a variety of modeling tools, from straightforward techniques to sophisticated models. Many models can now connect with digital topography data to create automatic delineations of dam break inundation zones thanks to developments in GIS-based modeling.

The tools for dam breach modeling can provide the dam breach peak discharge and/or hydrograph only, or they can produce a breach hydrograph and route the downstream flood using a one- or two-dimensional hydraulic model.

2.5.1. Dam Breach Hydrograph and Peak Outflow Generation Tools

The calculation of dam breach parameters using regression equations and the creation of a model that replicates the actual breach on the ground are required for the process of acquiring outflow hydrograph using HEC-RAS software.

Making accurate estimates of the peak discharge, outflow hydrographs, and downstream inundation requires knowledge of the breach's location, type, dimension, and development time. Using historical data from the past, a number of researchers have created a set of regression equations to calculate breach parameters such breach width, breach development time, and side slope.

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

Table 2. 5: Most Widely Used Dam Breach Modelling Tools **(FEMA, July 2013)**

Method	Computation of Peak Breach Outflow	Computation of Ultimate Breach Parameters	Breach Hydrograph Generation	Downstream Routing Capability			
				Steady-State	Unsteady-State	1-D Flow	2-D Flow
BREACH HYDROGRAPH GENERATION ONLY							
Empirical Equations	✓	✓					
NWS BREACH	✓	✓	✓				
USACE HEC-1 and HEC-HMS	✓		✓	Without downstream hydrologic routing	✓		
BREACH HYDROGRAPH GENERATION AND DOWNSTREAM HYDRAULIC ROUTING							
One-Dimensional Models							
NRCS TR-66(1)	✓		✓	✓	✓	✓	
WinDAM		✓	✓				
NWS SMPDBK	✓			✓		✓	

NWS FLDWAV	✓		✓		✓	✓	
USACE HEC-1 and HEC-HMS	✓		✓	Downstream hydrologic routing (3)		✓	
USACE HEC- RAS	✓		✓	✓	✓	✓	
TWO-DIMENSIONAL MODELS							
DSS-WISE	✓		✓		✓		✓
FLO-2D©	✓	✓	✓		✓	✓	✓
MIKE© FLOOD	✓		✓		✓	✓	✓

(1) NRCS TR-66 may be used in conjunction with TR-60 to determine input breach parameters such as breach width and time to breach.

(2) NWS FLDWAV program is embedded with NWS BREACH to determine breach parameters internally.

(3) Routing of breach hydrography downstream using a hydrologic routing method.

To produce a dam breach outflow hydrograph and a downstream routing analysis for this research project, the United States Army of Corps of Engineers' Hydraulic Engineering Center River Analysis System (USACE, HEC-RAS 6.3.1) was used.

2.5.2. Dam Breach Flood Routing

The primary job in modeling dam breach and downstream risk analysis is routing the breach outflow hydrograph downstream to assess the probable consequences of dam failure. HEC-RAS can be used to route an inflow flood hydrograph through a reservoir either with two-dimensional unsteady flow routing; or two- dimensional unsteady flow routing (Diffusive Equation).

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

Instead of being modeled in 1D HEC-RAS by storage area and inline structure, reservoir and dam can also be modeled in 2D HEC-RAS by storage area + SA/2D connection + downstream area in 2D domain in Figure 2.11. The advantages of using 2D HEC-RAS instead of 1D to model a downstream area include avoiding time-consuming cut line placement for 1D river cross sections, the flexibility to model flow in 2D rather than 1D, and simpler SA/2D link configuration. Make no mistake, a modeler still needs to put in a lot of work to setup an accurate terrain model, appropriate break lines, 2D refinement regions, SA/2D connections like dams/bridges/culverts, Manning's n layer, mesh sizes, and boundary conditions in order to create a thorough and valid 2D HEC-RAS model,. (Krest, 2022)

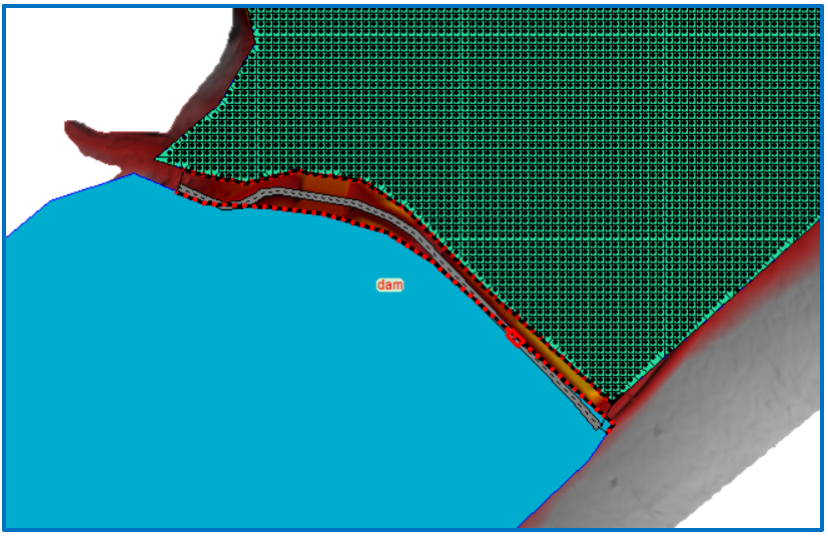


Figure 2. 11: SA/2D connection within HEC-RAS **(Krest, 2022)**

The storage area boundary representing a reservoir can be drawn by referring to ECDSCo. Design Report shapefile or google satellite images. The downstream side of a reservoir is usually the dam location while the upstream boundary of a reservoir is where the river flowing into it starts to become inundated. The maximum reservoir pool under PMF/PMP or IDF - Inflow Design Flood should be observed and the reservoir (storage area) boundary polygon should be drawn to at least cover this limit, as shown in Figure 2.12. This is a more "correct" method of determining the upstream boundary of a reservoir. (Krest, 2022)

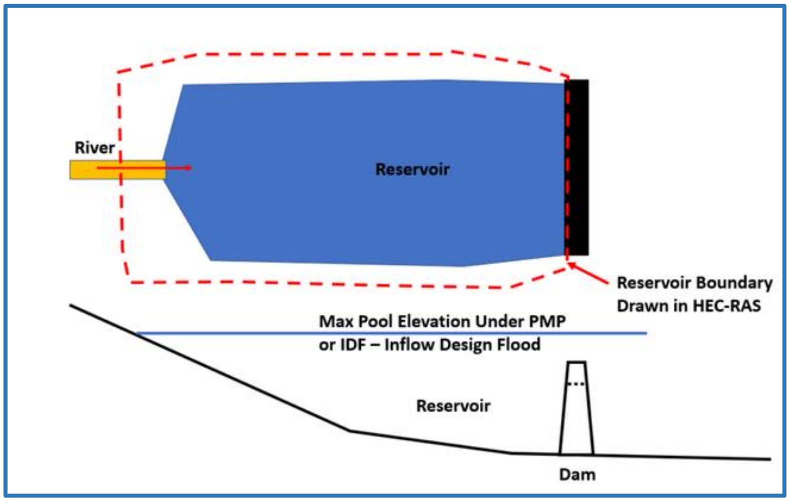


Figure 2. 12: Storage Area (Reservoir) and Elevation r/sh within HEC-RAS **(Krest, 2022)**

Using the data editor, when a reservoir (storage area) boundary has been defined, HEC-RAS can automatically pull an Elevation (m) - Volume (MCM) curve data set by clipping the surrounding topography. HEC-RAS actually uses this curve data set to do storage area routing calculation and it can be manually revised if deemed necessary. (Krest, 2022)

A 2D domain in HEC-RAS can be used to model the downstream portion of a dam. The toe of the dam embankment should serve as the upper boundary of the 2D domain. (Note: Placing the 2D domain upstream boundary at the top of the dam or on its slope may not be a good idea. 2D cells on steep surfaces with a gradient more than 10% do not like HEC-RAS. Make sure the cell size is large enough such that one cell covers the full slope from the top to the bottom of the dam if you must represent the dam slope in a 2D domain for some reason. (Krest, 2022)

2.5.2.1. Basics of 2D Flow Routing

Flow modeling will be utilized in inundation analysis to mimic the movement of a flood wave through a floodplain.

In the Unsteady Flow Computational Options dialog box, which is accessible through the Analysis ribbon menu, HEC-RAS offers three different ways to compute the flow field in a 2D mesh. As shown below, HEC-RAS has three sets of equations that can be used to solve for the flow over the computational mesh: (CivilGEO, 2023)

- **Diffusion Wave** – The default equation.
- **SWE-Eulerian-Lagrangean Method** – Solution to the Shallow Water equations originally.
- **SWE-Eulerian Method** – A new, more momentum-based solution to the Shallow Water equations.

2.5.2.2. 2D Diffusion Wave Computational Method

The computations can run more quickly and steadily thanks to the default solver, the 2D Diffusion Wave computational approach. This solver can accurately represent the majority of 2D modeling scenarios, such as excellent modeling, where inertial forces tend to dominate frictional and other forces. (CivilGEO, 2023)

The Diffusion Wave computational method can be used in the following situations: (CivilGEO, 2023)

- Flow is mainly driven by friction and gravity.
- Fluid acceleration is monotonic and smooth (i.e., no waves)
- Compute rough global estimates (i.e., good extents)
- Assess interior flooding (i.e., levee breach) •
- A quick calculation for the Full Momentum computational technique.

2.5.2.3. 2D Full Momentum Computational Method

The Saint Venant equations for shallow flow, also known as the 2D Full Momentum computational approach (Shallow Water Equations), can consider turbulence and Coriolis forces, making it suitable to a larger range of circumstances. But because it takes more computing resources to solve the 2D Saint Venant flow equations, it takes longer to run. Additionally, the numerical stability of the 2D Saint Venant flow equations might be affected by areas of the mesh where the water surface profile or flow direction is quickly changing. A finer mesh and correspondingly smaller time step must be employed to prevent an unstable model. (CivilGEO, 2023)

In a number of cases, the Shallow Water Equations should be employed for improved accuracy. Using the Diffusion Wave equations is a general strategy for creating the model. Once the model is operational, the user can create a different scenario and compare the outcomes using the shallow water equations. Results that differ significantly from each other indicate that the Shallow Water Equations solution is more accurate. (CivilGEO, 2023)

A more conservative version of the momentum equation serves as the foundation for the new SWE-Eulerian Method, an explicit solution scheme. Compared to the original SWE equation, it results in less numerical diffusion. The SWE-Eulerian Method is generally not required. when users are looking closely at tight contractions and expansions, piers/abutments, and changes in water surfaces and velocities at and near hydraulic structures. For the majority of issues needing the full momentum equation-based solution method, the original SWE-Eulerian-Lagrangean Method is more than sufficient. (CivilGEO, 2023)

In the following circumstances, it is recommended to apply the Full Momentum computational method: (CivilGEO, 2023)

- Dynamic flood waves, such as those caused by dam failure and abrupt rise and fall.
- High velocity changes and sudden expansion or contraction of the flow
- Exact flow solutions around hydraulic obstructions and structures (such as piers, abutments, and bridge openings)
- Detailed mixed flow regime, including critical flow and hydraulic leaps, etc.
- Wave propagation, or waves reflected off of surfaces like walls and objects.
- Tidal boundary conditions, or the propagation of upstream waves

2.5.2.4. 2D Flow Area External Boundary Conditions

By building polygonal areas that represent the regions to be modeled, 2D flow zones are constructed. Boundary condition polylines are constructed along the 2D flow area polygon mesh boundary to reflect various flow conditions or limitations that should be applied to the 2D flow area. Where flow enters or exits the 2D flow area, these boundary conditions serve as flux boundaries. In order to depict additional discharge that enters the 2D flow region, boundary conditions can also be defined inside the interior of the 2D flow area. (CivilGEO, 2023)

Examples of flux boundaries are: (CivilGEO, 2023)

- Inflow hydrograph
- Stage hydrograph (time series)
- Fixed water surface elevation
- Normal depth (given user-defined energy slope)
- Tidal (time-series)

2.5.2.5. HEC-RAS 2D Modeling Guidance and Assumptions

The following modeling guidelines and assumptions are put forward for the HEC-RAS 2D computational methodology: (CivilGEO, 2023)

- Vertical fluid motion is negligible.
- At the cell's center, velocity is vertically averaged (depth-averaged flow).
- At the cell center, the energy head will be computed.
- The roughness value at the cell face's center, was applied to the cell face.
- Even though each cell face can have a different value, Manning's roughness was thought to be constant across all cell faces.
- The 2D mesh must have at least one external boundary condition.
- The choice of time step should take wave speed and cell size into account.
- Use a SA/2D connection with gate openings and corresponding rating curve to simulate bridge pressure flow and road overflow.
- Use a SA/2D connection with culverts to simulate bridge pressure flow and road overflow.

2.5.2.6. Governing Equations

An overview of the discretization and solution algorithm of the DWE is described in section. The derivation begins with the discretization of continuity equation and momentum equations. The discrete form of the SWE is then obtained from the continuity and momentum equations. Finally, the solution algorithm is described. The continuity equation and the concept of conservation of momentum together make up the law that controls how water moves in a stream. These laws are mathematically described as continuity equations and momentum equations, two types of partial differential equations.

Discrete Boundary Conditions

Flow boundary conditions are also discretized: (USACE, 2023)

- Water Surface Elevation: The water surface elevation boundary condition is directly implemented as z_s^{n+1}=z_s^n.
- Normal Depth: The energy grade slope is specified and utilized to compute a flow at each computation face as Q=K$\sqrt{s_f}$. Boundary face flows are included in the internal cells as a source term on the right-hand-side of the system of equations.
- Flow: The flow boundary condition is similarly implemented as a condition on the water surface gradient using a finite volume approximation of Manning's equation. A rotation of the local coordinate system is necessary since, in general, the direction normal to the boundary does not coincide with the Cartesian directions.

Solution Algorithm

The complete solution algorithm is given here: (USACE, 2023)

1. The geometry, local orthogonality and sub-grid bathymetry data is obtained or pre-computed.
2. Solution starts with z0s as the provided initial condition at time-step n = 0.
3. Boundary conditions are provided for the next time step n+1.
4. Initial guess z_s^{n+1}=z_s^n.
5. Compute the θ-averaged water surface elevation $z_s^{n+\theta}$ and subgrid bathymetry quantities that depend on it (face area, horizontal surface area, hydraulic radius, Manning's n, etc.).
6. The coefficients $a_{i,j}$ are computed and the system of equations is assembled.
7. The system of equations is solved iteratively using the Newton-like algorithm with the given boundary conditions to obtain a candidate solution z_s^{n+1}.
8. If the residual (or alternatively, the correction) is larger than a given tolerance (and the maximum number of iterations has not been reached), return to step 5; otherwise continue with step 9.
9. The computed z_s^{n+1} is accepted and the velocities u_N^{n+1} can be calculated using the discrete version of the momentum equation.
10. Increment n. If there are more time steps go back to step 3, otherwise end.

The loop provided by steps 5 through 8 has the purpose of updating the coefficients $a_{i,j}$ so that the solution of the nonlinear system (rather than its linearization) is obtained at every time step. As expected, a fully nonlinear solution has very desirable properties such as wetting several cells or updating coefficients that are evaluated at time n+θ.

2.6. Inundation Map

The inundation map shows the geographic range of flooding that would result from the dam break flood, as is described in the Washington State Department of Ecology Dam Safety Guidelines. Additionally, it should indicate flooding for representative cross-sections of the channel and identify high-velocity flow zones.

When evaluating the downstream effects, normal pool and maximum pool storage elevation, two reservoir conditions, are typically evaluated. The inundation polygon, the most crucial dataset for inundation mapping, is provided by the inundation delineation. In a GIS, a polygon is a specific kind of layer. After a dam breach simulation is finished, the data can be stored in GIS to offer users a variety of potential advantages, including the possibility of increased accuracy and effectiveness. As a post-processing feature of HEC-RAS, numerous GIS applications, such as the USACE HEC-GeoRAS ArcMap extension and RAS Mapper, offer capabilities to automatically designate floodplains or inundations. (FEMA, July 2013).

2.7. Hazard Classification and Breach Failure Scenarios of Dams

The effect of the potential flood from a dam failure on downstream people, property, and the environment must be considered to arrive at a hazard classification after a dam break assessment has been completed to characterize the potential downstream flooding from a dam failure. (Klotz Mark, 2020)

The United States and State authorities employ a dam's hazard potential rating, together with its size (height and capacity) classification, to control dam design and dam breach modeling. According to FEMA recommendations, hazard potential classifications should be classified as low, considerable, or high depending on the likelihood of human life loss, financial loss, and environmental harm as a result of a fictitious dam failure. (FEMA, July 2013).

The dam safety industry utilizes a hazard potential classification to divide each dam into one of several categories due to the wide range of sizes of dams and the effects of dam failure. This enables dam safety initiatives to concentrate on the risky dams rather than the thousands or hundreds of tiny dams that provide little or no risk. (NPS, December 2013)

Regarding the examination of the safety threat associated with dams, various authorities have different classification methods for them. These classes can be used for licensing or design objectives, as well as for other purposes like budgeting and remedial action prioritization. Most of these systems employ criteria based on dam factors, including the height or size of reservoirs, as well as possible dangers that could result in fatalities and financial losses. In order to explain the idea guiding the dam safety measures currently in use around the globe, a few of these systems are provided here.

2.7.1. Classification of the International Congress on Large Dams (ICOLD)

The International Commission on Large Dams (ICOLD) considers any dam large dam if its height is 15 meters or more measured from the lowest point of the foundation to the top of the dam. Dams with a height of 10 m or more are considered as high dams if they satisfy one of the following criteria or a combination of more than one of them as set out in Table 2.6.

Table 2. 6: ICOLD large dams classification criteria for dams >10 m in height. **(ICOLD, 2011)**

Criteria	Requirement
Length of Crest	> 500m
Volume of Reservoir	> 1 million m3
Design Flood	> 2000 m3/sec
Geological Conditions	Difficult
Other Considerations	Usual Design

In considering the question of potential hazards posed by dams, ICOLD has modified these basic criteria to link the size of a dam to the hazards it could create and has adopted modified criteria based on the French Committee on Dams and Reservoirs' guidelines (ICOLD, 2011). These guidelines consider the two parameters (H), the maximum height of the dam in meters, and (V) which is the reservoir's volume in m3 in defining a numerical index H2√V to rate the Potential Hazard Classification (PHC). The relationship may be obtained by using semi-log paper; (V) on the X-axis and (H) on the Y-axis as shown in the chart given in Figure 2.13, this chart may be read in conjunction with Table 2.6 for obtaining the Potential Hazard Classification (PHC) for small dams not exceeding 25m in height and having reservoir volume not more than 100 million m3.

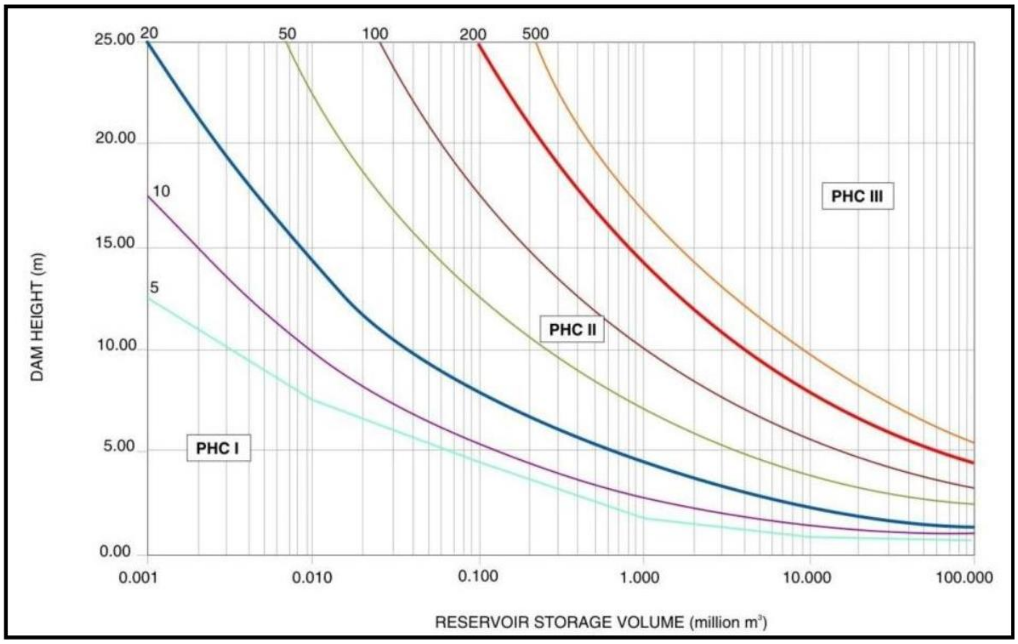


Figure 2. 13: Relationship H2√V for small dams. **(ICOLD, 2011)**

Table 2. 7: Potential Hazard Classifications (PHC). **(ICOLD, 2011)**

Component	**Potential Hazard Classification (PHC)**		
	Low – (I)	**Medium – (II)**	**High – (III)**
$\boldsymbol{H^2\sqrt{V}}$	$H^2\sqrt{V} < 20$	$20 < H^2\sqrt{V} < 200$	$H^2\sqrt{V} \geq 200$
Life Safety Risk (number of lives)	0	< 10	≥ 10
Economic Risk	Low	Moderate	High or Extreme
Environmental Risk	Low or Moderate	High	Extreme
Social Disruption	Low (Rural Area)	Regional	National

This guideline; however, modifies ICOLD's definition for small dams given in Table 2.7, and qualifies them to be high dams if they follow the criterion shown below: Dams with height 5< H < 15 m and V>3hm3.

Accordingly, the differentiation of small dams from large dams follows the boundaries in Figure 2.14. This classification is related directly to hazards posed by these dams and therefore, it is a better classification system than the original ICOLD classification.

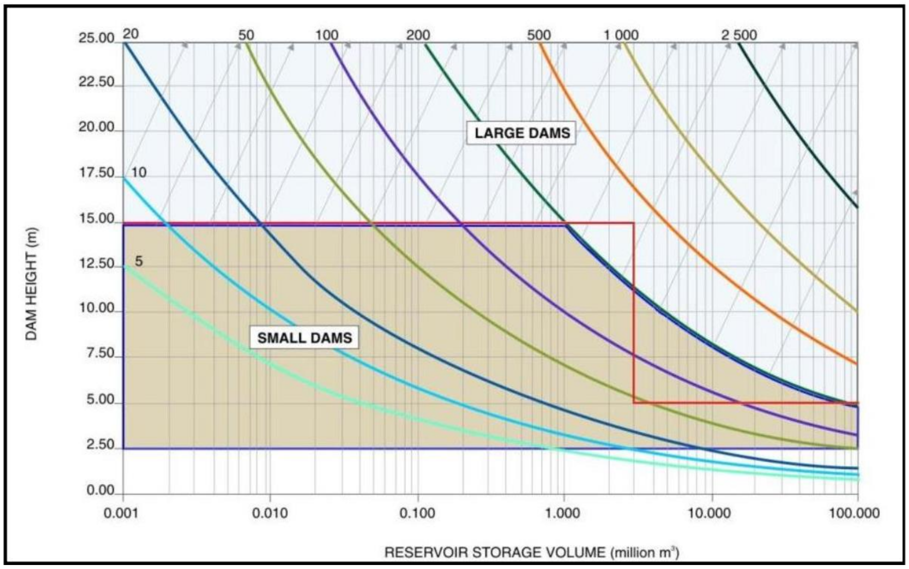


Figure 2. 14: Classification of small and large dams. **(ICOLD, 2011)**

2.7.2. Classification of the ENTRO Guideline

The experts at ENTRO conducted a comparative review of the dam safety recommendations from Malaysia, Canada, Uganda, and ICOLD in 2013 and suggested that the Eastern Nile Countries adopt the most suited and appropriate recommendation. These four recommendations were chosen due of their thoroughness, applicability to EN countries, and availability of the recommendations. (ENTRO, December, 2014)

The conclusions of the Comparative Analysis were:

- The objective of developing a dam safety guideline is to ensure safety of dams during their whole life cycle - from the conceptual phase, through design, construction, and operational stage to decommissioning.
- The dam safety guideline needs to consider existing policies, regulations, standards, directives, capability of professionals, technological advancement, institutional strength, and trans-boundary nature of projects, etc. In line with this, it was attempted to review the contents and adaptability of the four-dam safety guidelines; and
- Each of these guidelines has its own merits and limitations. However, considering its comprehensiveness; descriptive procedures; freely availability of supplementary reference material (ICOLD bulletins); applicability both for small and large dams; the ICOLD dam safety guideline is more suitable and appropriate for adapting to EN countries.

This categorization system, which is shown in Table 2.8, should be applied to the level of care required of dam owners and Approved Dam Engineers as well as employed when choosing design criteria. The dam's Potential Consequences Classification (PCC) should be the highest ranking determined from the numerous risk categories displayed.

Table 2. 8: Potential Consequences Classification for Eastern Nile Dams **(ENTRO, December, 2014)**

Dam Class	Loss of life	Infrastructure, Economic and Social Factors	Environmental & Cultural Factors
VERY HIGH **Level 4**	Large potential for multiple loss of life involving residents and working, traveling and/or recreating public. Development within the potential inundation area (the area that would be flooded if the dam fails), considering both national and international or trans-boundary areas, typically includes communities, extensive agricultural, commercial, and work areas, main highways, railways, ports, and locations of concentrated recreational activity. Estimated loss of life could exceed 1 000.	Very high economic losses affecting infrastructure, public and commercial facilities in and beyond the inundation area considering both national and international or trans-boundary areas. Typically includes destruction of or extensive damage to large residential areas, concentrated agricultural and/or commercial land uses, hydroelectric generation facilities, highways, railways, ports and shipping facilities, power lines, pipelines, water supply and other utilities. Estimated direct and indirect (interruption of service) costs could exceed $100 million.	Loss or significant deterioration of nationally or locally important fisheries habitat (including water quality), wildlife habitat, rare and/or endangered species, unique landscapes, or sites of cultural significance. Feasibility and/or practicality of restoration and/or compensation are low.
HIGH **Level 3**	Potential for multiple loss of life involving residents, and working, traveling, and/or recreating public. Development within inundation areas typically includes highways and railways, ports, agricultural, commercial, and work areas, locations of concentrated recreational activity and scattered residences. Estimated loss of life between 100 and 1 000.	Substantial economic losses affecting infrastructure, public, agricultural, and commercial facilities in and beyond inundation area. Typically includes destruction of or extensive damage to concentrated agricultural and/or commercial land uses, hydroelectric generation facilities, highways, railways, ports and shipping facilities, power lines, pipelines, water supply and other utilities. Scattered residences may be destroyed or severely damaged. Estimated direct and indirect (interruption of service) costs could exceed $1 million.	Loss or significant deterioration of nationally or locally important fisheries habitat (including water quality), wildlife habitat, rare and/or endangered species, unique landscapes, or sites of cultural significance. Feasibility and practicality of restoration and/or compensation is high.
MODERATE **Level 2**	Low potential for multiple loss of life. Inundation area is typically underdeveloped except for minor roads, temporarily inhabited or non-residential farms and rural activities. There must be a reliable element of natural warning if larger development exists. Estimated loss of life between 10 and 100.	Low economic losses to limited infrastructure, public and commercial activities. Estimated direct and indirect (interruption of service) costs could exceed $100,000.	Loss or significant deterioration of regionally important fisheries habitat (including water quality), wildlife habitat, rare and/or endangered species, unique landscapes, or sites of cultural significance. Likelihood of recovery or feasibility of restoration or and/or compensation is high.
LOW **Level 1**	Minimal potential for any loss of life. The inundation area is typically undeveloped. Estimated loss of life between 1 and 10.	Minimal economic losses typically limited to owners' property. Virtually no potential for future development of other land uses within the foreseeable future.	No significant loss or deterioration of fisheries habitat, wildlife habitat, rare and/or endangered species, unique landscapes, or sites of cultural significance.
REMOTE **Level 0**	No potential for any loss of life. The inundation area is typically undeveloped.	Minimal economic losses typically limited to owners' property. Virtually no potential for future development of other land uses within the foreseeable future.	No significant loss or deterioration of fisheries habitat, wildlife habitat, rare and/or endangered species, unique landscapes, or sites of cultural significance.

2.7.3. Classification of the Federal Emergency Management Agency (FEMA)

This organization was founded in the USA in April 1979 with the goal of safeguarding citizens from dangers related to nuclear power plants, the transportation of hazardous materials, safeguarding citizens from dangers related to natural disasters, and other dangers. Increasing public safety issues brought on by dams became one of its primary concerns,

The classification of dams adopted and used by FEMA follows two criteria:

- Loss of life
- Economic, Environmental and Lifeline losses

Three classification levels are used, which are: LOW, SIGNIFICANT, and HIGH, listed in order of increasing adverse incremental consequences. The classification levels build on each other, i.e., the higher-order classification levels add to the list of consequences for the lower classification levels, as noted in Table 2.9, and described below.

Table 2. 9: Classification of dams according to their hazard potential. **(FEMA, January 2004)**

Classification	Loss of Human Life	Economic Environmental, Lifeline losses
A – Low	Non-Expected	Low and generally limited to owner
B – Significant	Non-Expected	Yes
C – High	Probable, one or more expected	Yes

A. Low Hazard Potential

The designation of low hazard potential applies to dams whose failure or improper operation leads in minimal economic, environmental, and/or human life losses, but primarily solely personal property damages.

B. Significant Hazard Potential

The designation of "significant hazard potential" is given to dams whose failure or improper operation may not likely result in a loss of human life but may result in economic loss, environmental harm, disruption of life-supporting infrastructure, or other issues. Classified dams with a considerable risk of occurrence are frequently found in primarily rural or agricultural settings, but they can also be found in locations with significant population and infrastructure.

C. High Hazard Potential

Dams assigned to the high hazard potential classification are those where failure or misoperation will probably cause loss of human life in addition to economic, environmental, and lifeline losses.

This Hazard Potential Classification System acknowledges that any dam or water-retaining structure, no matter how tiny, that fails, or malfunctions could pose a risk to life and property downstream. No matter how unanticipated, if there is an uncontrolled release of stored water, there is always a chance that someone will be in the discharge's path. The loss of life criterion distinguishes the Significant and High Hazard Potential Classification levels, wherein a High-

Hazard Dam may likely result in the loss of human life while a Significant Hazard Potential Dam does not predict such a loss.

2.7.4. Other Classification Systems

Although several States in the United States have their own classification schemes for dams, the Federal Emergency Management Agency (FEMA) oversees all these dams. These systems are primarily designed to license the building of new dams, regulate the building process, and establish guidelines for the management and maintenance of such a dam. All aim to improve public safety and lessen the risks these dams offer to the general population (DEC, January 1989), (WDE, 1993).

Many European nations, including the UK, Australia, Canada, and Canada, have classifications with similar goals. A description of the rules adopted by the Australian National Committee on Large Dams (ANCOLD) for categorizing the potential dangers of dams is provided in Table 2.10. As a criterion for hazard classification, the term Population at Risk (PAR) is introduced in these guidelines. The term "population at risk" refers to all those living in the downstream flood plain who are potentially at risk from dam failure. (ANCOLD, 2003)

Table 2. 10: Dam Hazard Categories according to ANCOLD. **(ANCOLD, 2003)**

Population at Risk (PAR)	**Severity Damage and Loss**			
	Negligible	**Minor**	**Medium**	**Major**
0	Very Low	Very Low	Low	Significant
1-10	Low Notes 1 &4	Low Notes 4 &5	Low Notes 5	Low Notes 6
11-100	Note 1	Significant Notes 2 & 5	High C Notes 6	High B Notes 6
101-1000		Note 2	High A Notes 6	High A Notes 6
>1000			Note 3	Extreme Notes 6

Note 1: With a PAR of 5 or more people, it is unlikely that the severity of damage and loss will be: Negligible.

Note 2: "Minor" damage and loss would be unlikely when the PAR exceeds 10.

Note 3: "Medium" damage and loss would be unlikely when the PAR exceeds 1000.

Note 4: "Change to *significant* where there is the potential for one life being lost is recognized.

Note 5: Change to High where is the potential for one or more lives being lost.

Note 6: Refer to section 2.7 and 1.6 of ANCOLD Guidelines on the consequences of Dam Failure for an explanation of the range of High Hazard Categories.

In the previous table, the likelihood of occurrence of hazardous incidents causing medium or major damage and loss is classified into categories:

i. **High A**; Almost certain. The event is expected to occur in most circumstances,
ii. **High B**; Likely. The event will probably occur in most circumstances and,
iii. **High C**; Possible. The event could possibly occur at some time.

All of this suggests that public safety is scrutinized and evaluated with great attention in practically every country. According to their potential effects on the communities downriver, dams should be categorized, as was already stated. This classification must be adopted and included in the application; for new dams, this is accomplished through in-depth research and studies, secure designs, and appropriate construction methods that are appropriate for the class of the dam.

According to their operation and maintenance manuals, existing dams must be operated safely, and routine safety checks and maintenance must be performed. If necessary, new developments in these domains, such as new forms of monitoring systems, such as new instruments and/or remote sensing techniques, may also be implemented. All necessary corrective actions should be implemented right away if an abnormality is found in order to improve the dam's safety conditions.

In the past century, thousands of dams have been constructed, and some of them are already showing severe indications of age and posing a serious hazard to the public. Decommissioning them should be seriously considered if corrective actions appear to be impracticable or extremely expensive.

3. METHODOLOGY

3.1. Study Area and Data Collection

3.1.1. Location

Upper Guder Irrigation Project is in Oromia Regional State at about 150 km west of Addis Ababa. The dam constructed for this irrigation project is called Fato dam. The Fato dam site is located between 360252E and 979403N at the Right abutment and 360386E and 979644N at the left abutment. While the irrigation command area of the project lies on the right bank of Fato River, which later downstream of Guder Town become Guder River and is southern of the main Addis-Nekemte Road. The Flood Inundation area is located right below the Fato dam in the north direction later when it reaches at HAEC the river makes sharp left turn and flows through the desert to reach Nile River. This inundation area has an altitude range between 2428 to 1608m. The Fato River is the source of water for the proposed irrigation project.

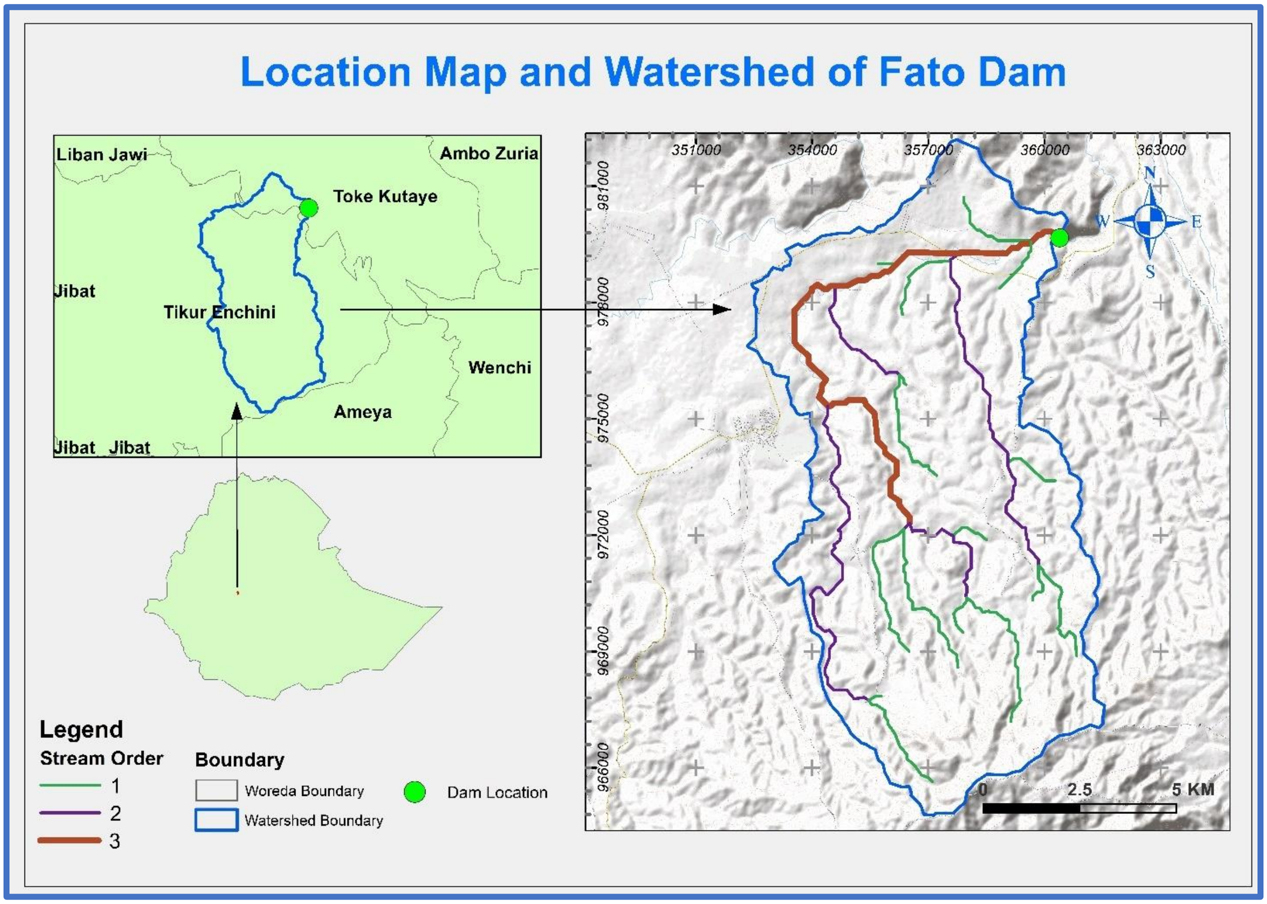


Figure 3. 1: Geographical Location of Fato Dam

3.1.2. Topography

The Fato headwater is at an elevation of about 3000m.a.s.l. The average slope of the Upper Guder watershed in general is estimated to be about 19% indicating the majority of the area falls under steep rolling, and moderately steep landforms.

Considering only the slope classes, more than 83% of the study area is susceptible to erosion by water coupled with improper land use practices and erodible nature of the soil in the Fato dam watershed**.** (ECDSCo., 2016)

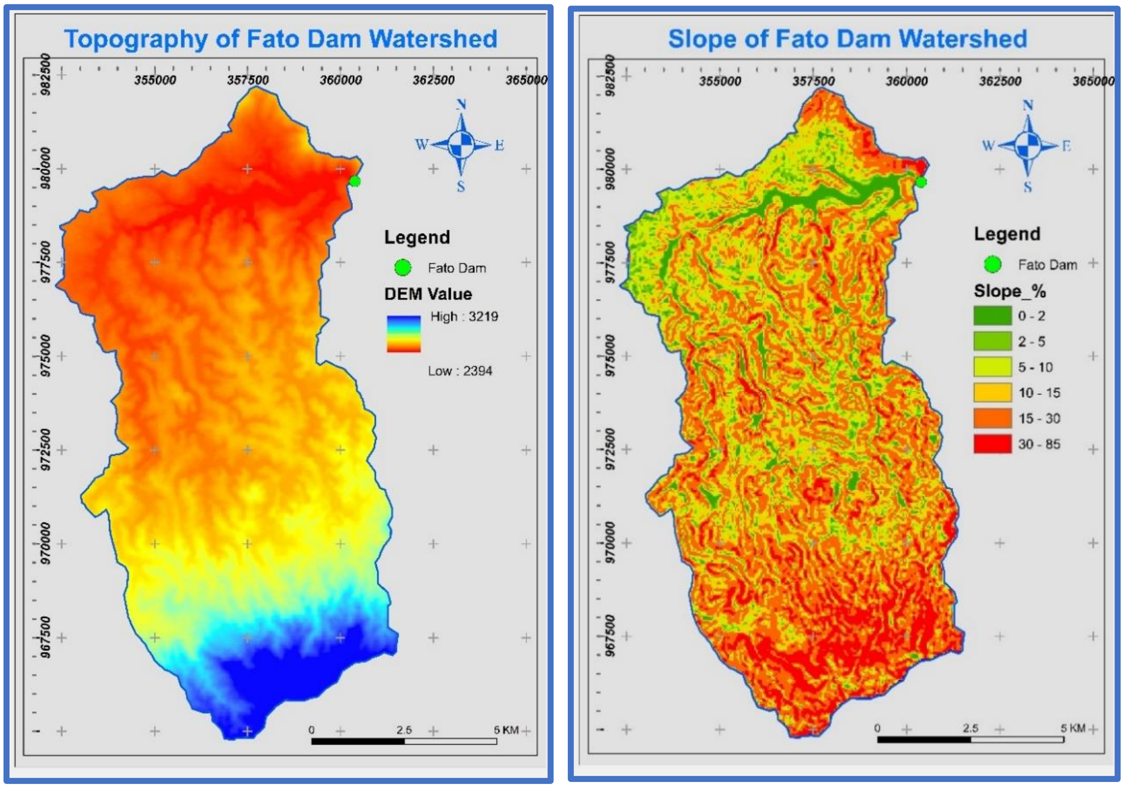


Figure 3. 2: Topography and Slope of Fato Dam Watershed

3.1.3. Hydrology

The Fato sub basin located within the Upper Guder River basin. The Fato watershed has Gravelius's index KG value of 1.4; hence the Fato watershed can be characterized by lower time of concentration, indicating the potential to generate lesser runoff at the outlet of the watershed system for a given amount of rainfall. (ECDSCo., 2016)

Ambo, Gedeo, Guder and Inchini rainfall gauging station are the key station for the project area while Inchin gauging station is much more representative of the reservoir area. The stations have a relatively long record covering 1954 to 2012 with missing values here and there. The mean monthly rainfall of the above stations has been used in the rainfall analysis for the catchment and command area after consistency and correlation is checked using the double mass curve technic. (ECDSCo., 2016)

Based on the UH developed by Snyder's method and effective rainfall excess increments, flood is convoluted. PMF peak of 353.49m3/s is found at peaks being at 17 hours accordingly. Peak floods of different return period are found as follows and are presented in Table 3.1. (ECDSCo., 2016)

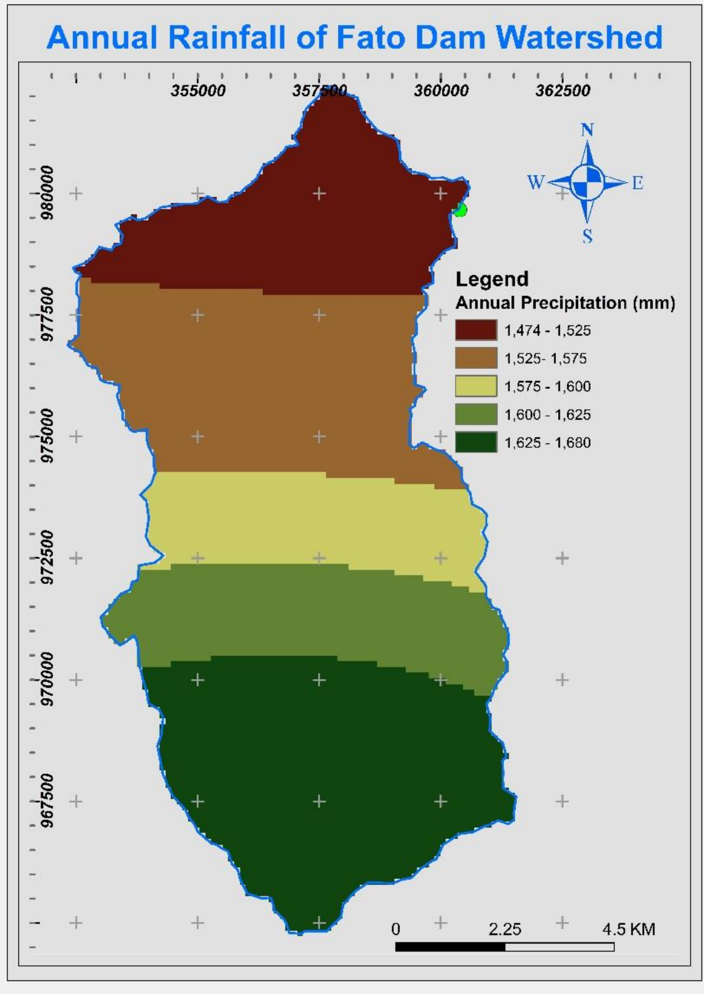


Figure 3. 3: Annual Rainfall Distribution of Fato Dam Watershed

Table 3. 1: Flood magnitude for different return period **(ECDSCo., 2016)**

Return Period	Flood Peak, m3/s @ Fato Dam site
25	**13.85**
50	**22.10**
10,000	**126.50**
1/2PMF	**176.75**
PMF	**353.49**

3.1.4. Geology

The Project area is situated in the central Ethiopian plateau comprising tertiary volcanics, and quaternary superficial sediment deposits. (ECDSCo., 2016)

The volcanics of the project area comprise of tertiary basalt rocks and pyroclastic. The dam site and reservoir area are generally comprised of rock units from the basaltic lava flows overlain by thin to thick residual soil mass formed from prolonged time weathering of the parent basalt rock. (ECDSCo., 2016)

Three types of basalt rocks compose the dam site: vesicular basalt, amygdaloidal basalt, and aphanitic basalt from top to bottom sequence. And the soil mass generally is dominated with silt and clay with sand (silty clay, clayey silt, and sandy silt), and increased inclusion of angular basalt gravel grains near contact to the parent rock. The thickness of the residual soil varies 8 to 33 meters at dam site area. (ECDSCo., 2016)

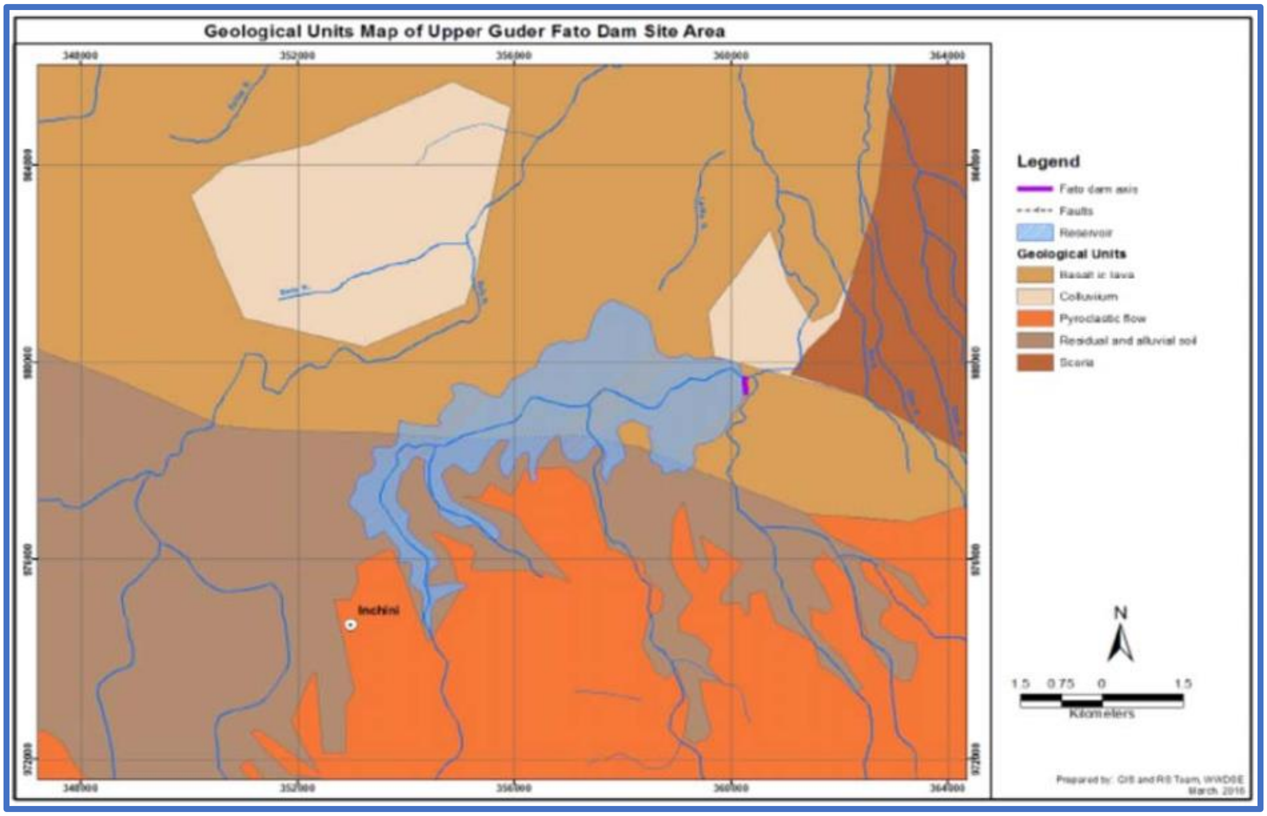


Figure 3. 4: Local Geological map of the Fato Dam Area **(ECDSCo., 2016)**

The geophysical survey, conducted at Fato dam site, has revealed the following:

- The middle and central portion of Fato dam site comprises of competent bed rock with little or no soil overburden at surface.
- The soil mass overlying the bed rock is significant on abutments with thickness estimated up to 30 meters.
- The investigation has interpreted the condition of the bed rock as weathered and fractured.
- A relatively weaker and deeper localized zone was intercepted at the left abutment of proposed dam site. Zone is interpreted as a geologic structure (possibly a fault NESW trending).

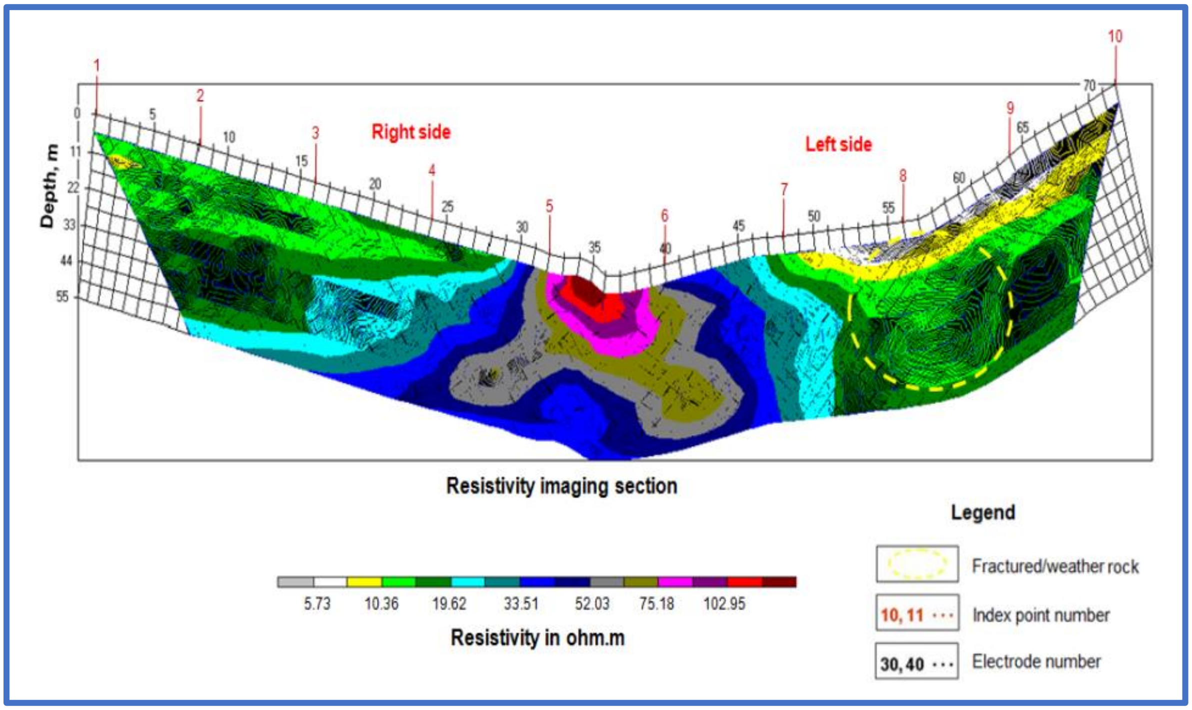


Figure 3. 5: Resistivity Imaging Section along Fato dam axis **(ECDSCo., 2016)**

3.1.5. Dam & Appurtenant Structures Type

A high rock-fill embankment dam on the Fato River, the Fato Multi-purpose Dam. It consists of a morning glory spillway at the center of a rockfill dam that is 40.1 meters high and 276.53 meters long (at its peak).

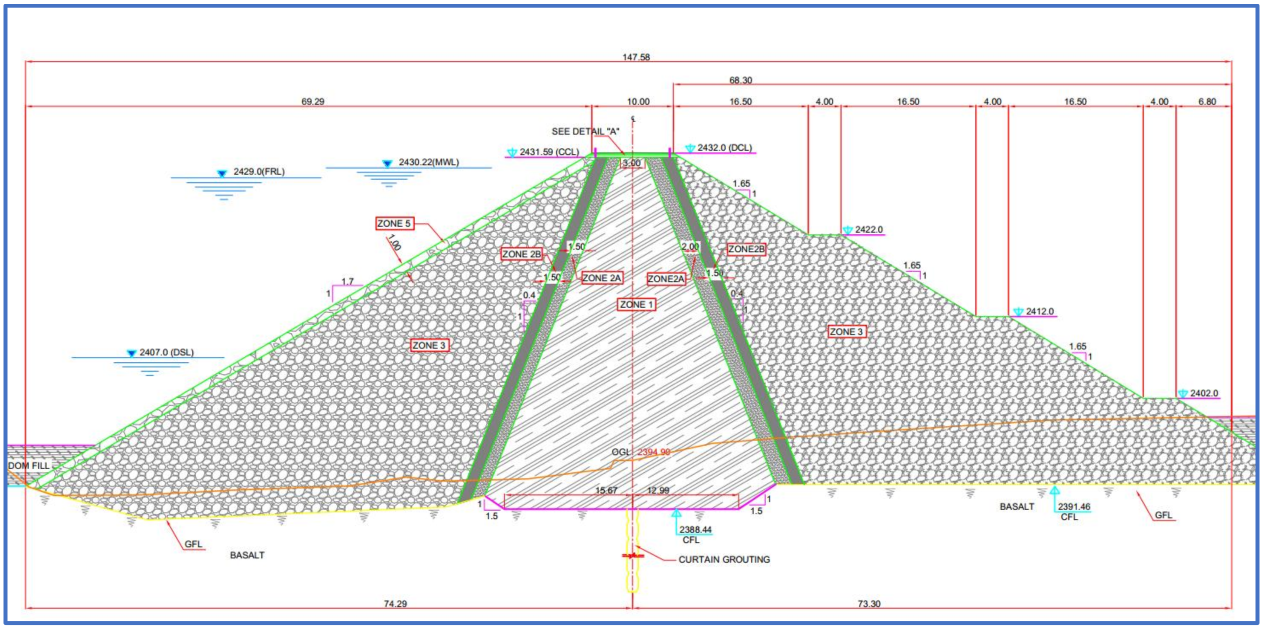


Figure 3. 6: Fato dam cross section

The irrigation main pipe designed to be 2 m diameter GRP pipe, the same size of concrete pipe is adopted for the irrigation conduit to get a uniform section and avoid expansion or contraction section and the head loss that may arise at the joints. The bed slope is made to be 1V in 200H. An invert level of 2407 m.a.s.l initially designed as the starting point for the intake gate pipe. (ECDSCo., 2016)

Based on the topographic, economic, and geologic considerations, morning shaft spillway considered as advantageously at dam sites in narrow canyons where abutments rise steeply or where a diversion conduit is available for use as the downstream leg. The near-maximum capacity of this type of spillway is obtained at relatively low heads, which makes it excellent for usage in situations where the maximum spillway outflow must be constrained. For this reason, they are most suited where temporary storage space in the reservoir is large enough to significantly attenuate the incoming flood. (ECDSCo., 2016)

The intake, spillway crest, transition from the crest to the shaft, bend, and conduit are the main components of a shaft spillway. The morning glory spillway is normally used in conjunction with a conduit spillway when the intake is a vertical shaft. Also, because of flow entry from the entire periphery, the crest capacity is relatively high.

Figure 3. 7: Cutoff trench excavation of Fato dam during construction

3.2. Data Collected

The accuracy and completeness of the data used in breach modeling is considered to foretell the failure impact analysis. The relevant data has been gathered in order to create a dam breach model and estimate how a collapse of the dam would affect the flood plain downstream. The results will be used to classify the risk associated with the Fato dam.

3.2.1. General Information

This section provides an overview of the research field as well as the information used for the hazard assessment of the Fato dam and hydraulic modeling of dam breach analysis. Natural floods are typically substantially smaller than floods caused by dam failure. The hydrologic data and reservoir capacity for the Fato Dam breach modeling and downstream effect analysis will be provided by the design report of ECDSCo. (ECDSCo., 2016).

Based on the Ethiopian construction design and supervision corporation report for the hydrology work the dam breach analysis was done. Since the main objective of this study is to define the hazard class of the dam the hydrological works like different years return periods flood hydrographs including the PMF was taken from their hydrologic report.

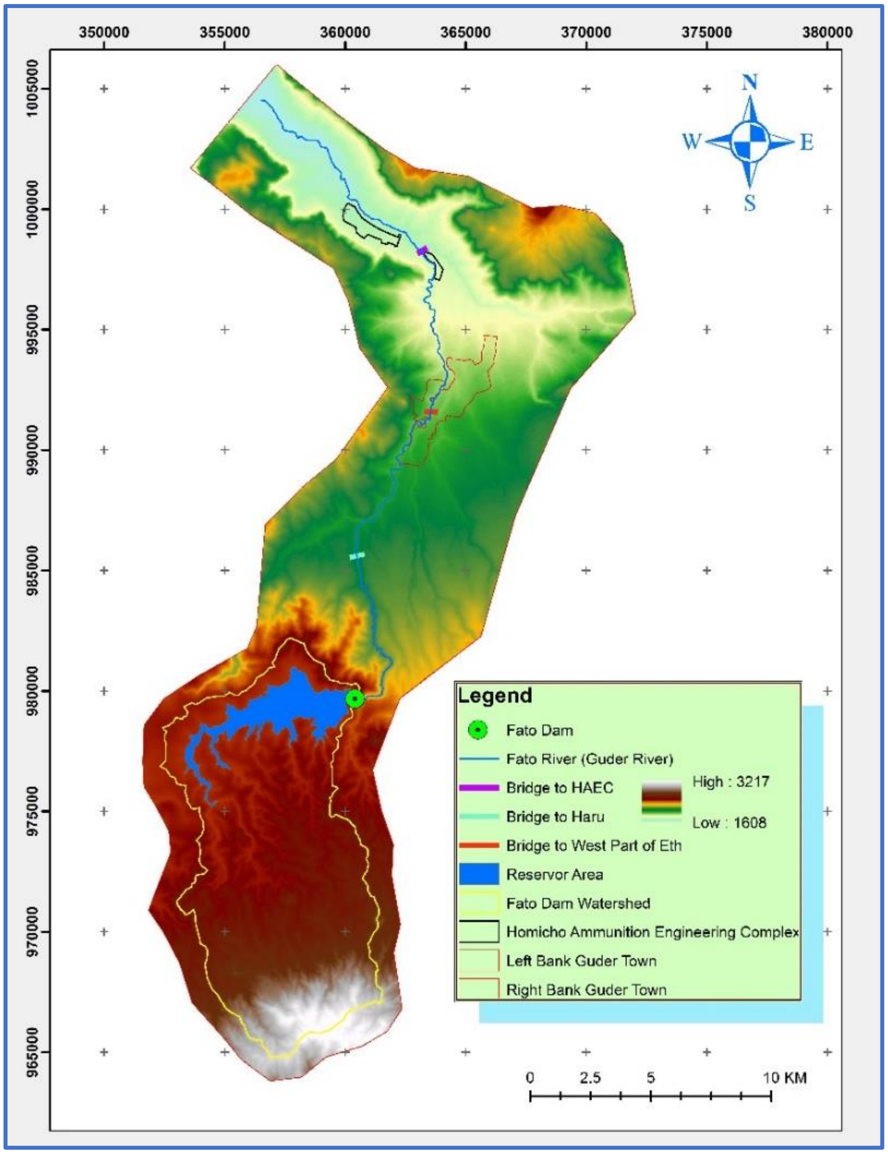


Figure 3. 8: Project Area and Flood Inundation Area Infrastructure and Settlements

3.2.2. Topographic Data

A study area description of the land surface was an important part of the process of the quality river hydraulics model. Elevation (topography) data is used to establish the area available to transport flow downstream. To map the flood inundation area due to the water surface profile outflow from the dam breach, topography data or elevation data must be known. The primary topographic (elevation) data for 2D flow area used for the analysis of the Fato Dam breach was extracted using a Digital Elevation Model (DEM) of resolution 12.5m by 12.5m from Alaska Satellite Facility. This was done by downloading it from their website. Global Mapper V23.0 and HEC-RAS V6.3.1 along the river up to 35 kilometers downstream of the dam DEM is downloaded.

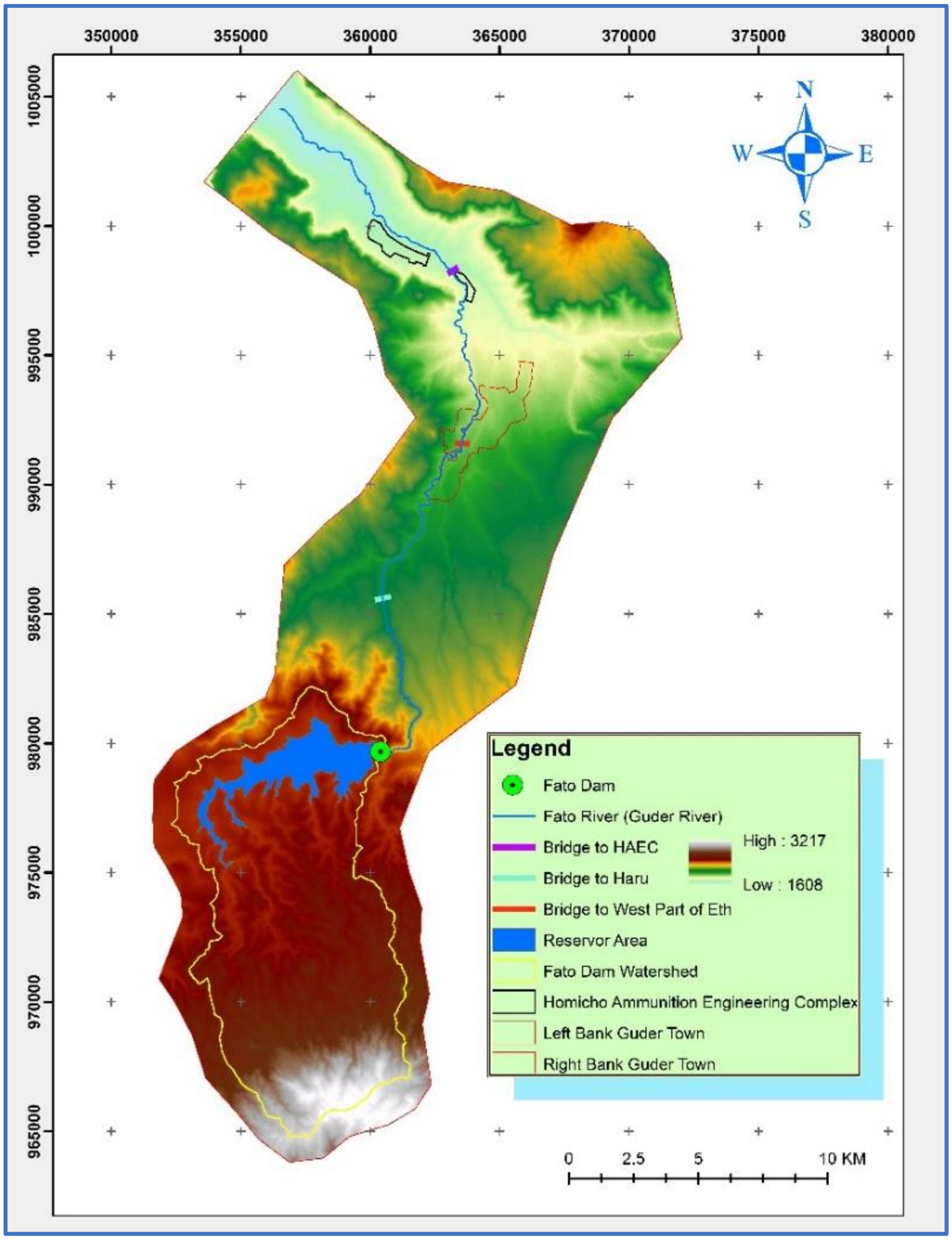


Figure 3. 9: Fato dam flood inundation area topography

3.2.3. Hydrologic Data

Ethiopian Construction Design and Supervision Corporation (ECDSCo.) supplied the hydrological information for the research, including the inflow hydrograph, maximum probable flood (PMF), and reservoir capacity. The Ethiopian Construction Design and Supervision Corporation provided the flood hydrograph with various return periods of rainfall in the catchment, (ECDSCo.).

Table 3. 2: Fato dam 25, 50, 10000 years Return period, PMF and 05PMF hydrograph. **(ECDSCo., 2016)**

Return period (years)	**25**	**50**	**10,000**	**PMF**	**0.5PMF**
Time (hr.)	Inflow (m^3/sec)	Inflow (m^3/sec)	Inflow (m^3/sec)	Inflow (m^3/sec)	Inflow (m^3/sec)
0	0.15	0.15	0.15	0.15	0.08
1	0.15	0.15	0.15	0.15	0.08
2	0.15	0.15	0.25	0.72	0.36
3	0.15	0.15	0.76	2.94	1.47
4	0.15	0.15	1.99	7.63	3.82
5	0.17	0.22	4.34	15.88	7.94
6	0.34	0.57	6.88	20.45	10.23
6	0.81	1.39	15.19	50.13	25.06
7	1.74	2.91	24.92	78.30	39.15
8	3.06	5.10	37.33	112.75	56.38
9	4.80	7.96	51.96	151.99	76.00
10	6.97	11.42	68.12	194.11	97.05
11	9.34	15.10	84.66	236.76	118.38
12	11.41	18.32	99.74	275.64	137.82
13	12.87	20.58	111.72	307.11	153.56
14	13.65	21.82	120.10	329.87	164.94
15	13.85	22.10	124.58	342.76	171.38
16	13.57	21.62	126.50	353.49	176.75
17	12.87	20.49	121.39	335.58	167.79
18	12.09	19.19	115.02	318.51	159.25
18	10.71	16.99	105.38	293.34	146.67
20	7.59	11.56	81.79	241.61	120.81
22	6.31	9.62	71.12	212.08	106.04
24	3.74	6.00	50.17	152.66	76.33
26	1.93	3.33	32.35	100.25	50.13
28	1.49	2.28	20.44	63.39	31.69
30	1.22	1.78	12.56	38.67	19.34

Maximum	13.85	22.10	126.50	353.49	176.75

Table 3. 3: Elevation-Area-Capacity Curve of Fato Dam Site **(ECDSCo., 2016)**

ELEVATION (masl)	AREA (km2)	VOLUME (MCM)	RPL (masl)	ELEVATION (masl)	AREA (km2)	VOLUME (MCM)	RPL (masl)
2394.82	0.003	0.003	2430	2437.82	10.69	181.783	2430
2395.82	0.028	0.032	2430	2438.82	11.065	192.849	2430
2396.82	0.082	0.114	2430	2439.82	11.459	204.308	2430
2397.82	0.144	0.258	2430	2440.82	11.884	216.192	2430
2398.82	0.224	0.481	2430	2441.82	12.313	228.505	2430
2399.82	0.344	0.825	2430	2442.82	12.728	241.233	2430
2400.82	0.484	1.309	2430	2443.82	13.178	254.411	2430
2401.82	0.622	1.931	2430	2444.82	13.633	268.044	2430
2402.82	0.786	2.717	2430	2445.82	14.084	282.128	2430
2403.82	0.938	3.655	2430	2446.82	14.577	296.706	2430
2404.82	1.101	4.756	2430	2447.82	15.05	311.756	2430
2405.82	1.276	6.032	2430	2448.82	15.386	327.142	2430
2406.82	1.452	7.484	2430				
2407.82	1.64	9.124	2430				
2408.82	1.821	10.945	2430				
2409.82	2.039	12.985	2430				
2410.82	2.25	15.235	2430				
2411.82	2.462	17.697	2430				
2412.82	2.678	20.374	2430				
2413.82	2.893	23.268	2430				
2414.82	3.105	26.372	2430				
2415.82	3.339	29.712	2430				
2416.82	3.604	33.316	2430				
2417.82	3.88	37.195	2430				
2418.82	4.142	41.337	2430				
2419.82	4.413	45.75	2430				
2420.82	4.701	50.451	2430				
2421.82	4.975	55.426	2430				
2422.82	5.274	60.7	2430				
2423.82	5.503	66.202	2430				
2424.82	5.83	72.033	2430				
2425.82	6.228	78.261	2430				
2426.82	6.589	84.85	2430				
2427.82	6.976	91.826	2430				
2428.82	7.315	99.141	2430				
2429.82	7.687	106.828	2430				
2430.82	8.066	114.894	2430				
2431.82	8.447	123.342	2430				
2432.82	8.832	132.174	2430				

2433.82	9.181	141.355	2430				
2434.82	9.525	150.88	2430				
2435.82	9.909	160.789	2430				
2436.82	10.304	171.094	2430				

3.3. Population and Infrastructure at Risk

The 2007 Population and Housing Census of Ethiopia: Statistical Report for Oromia Region; Part IV: Population Size of Kebeles, presents statistical information on the distribution of kebele populations by sex and by urban/rural areas, along with household and housing unit data. This report includes information on the population size, the number of households, and the number of housing units in the Guder Woreda kebeles' West Shewa zone. Guder town is located downstream of Fato dam at a distance of 15km. Around Guder town there are villages. Guder town, which is 12 km to the west of Ambo, is situated at 8°58′N 37°46′E, elevation 2101 meters above sea level, where the floods from the Fato Dam breach may affect in case of the failure.

Downstream of Fato dam, there are three bridges passing over the river. And next to Guder City there is an armory factory called Homacho Ammunition Engineering Complex. Therefore, these infrastructures and people living in these three critical places are prone to this breach flood.

Figure 3. 10: Bridge passing through the Guder town**.**

3.4. Dam Breach Modeling Process

The following steps were taken during the modeling of dam breaches and downstream risk analysis processes. First, three empirical equations have been used to estimate the breach parameters. Then, using HEC-RAS V6.3.1, downstream channel geometry (elevation data) was derived from the Digital Elevation Model (DEM). HEC-RAS has been used to model the dam break using the breach parameter and channel geometry. The downstream 2D mesh DEM is then used to channel the breach discharge in order to compute the hydraulic characteristics at key places. The flood inundation map in ArcMap may then be used to examine the impacted vital areas and calculate the number of fatalities, economic losses, and environmental damages within the inundation area. The flow chart figure 3.11 below outlines the general downstream analysis and dam breach modeling processes.

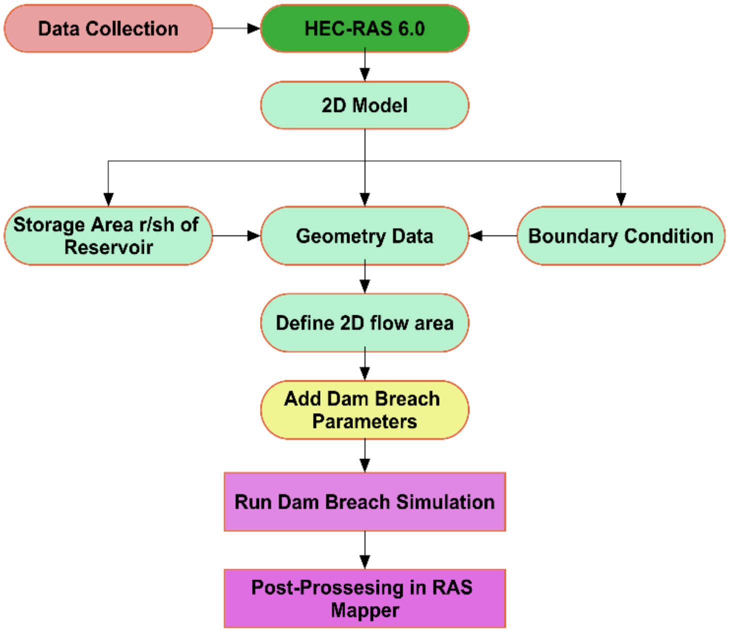


Figure 3. 11: Flow diagram of 2D Fato Dam Breach Modeling and Analysis Process

The reservoir capacity of the Fato Irrigation Project Dam is 114.89 Mm3 at Maximum Water Level (MWL) and 99.14 Mm3 at Full Reservoir Level (FRL). The dam measures 40.1 meters in height. A major village called Guder, and a recreation area called "Guder Fall" may be found near the downstream end of the dam. In addition to this, the downstream area has various buildings, such as bridges, and a population that may be impacted by flood-flow if the dam fails. Therefore, Fato Dam was categorized as a "High Hazard Dam" in this study in accordance with the ENTRO hazard potential classification standards and considering the downstream population size and property damage as a result of flood-flow caused by a dam failure.

For Fato Dam breach analysis, two-dimensional full dynamic flow routing is applied. This rock-fill embankment dam is designed for failure modes including overtopping breaches. While the dam has a low piping failure condition due to the dam spillway and intake passing through the foundation of the dam, the failure position for an overtopping failure mode is at centerline of the channel riverbed in the downstream side of the embankment dam.

The breach creation time and the maximum size of the breach opening (breach width) are the primary characteristics that must be calculated in this research. In order to anticipate the outflow hydrograph from the breach, these parameters are computed using empirical (regression) equations and input into HEC-RAS V6.3.1.

The regression equations applied for this purpose are Custom, Van Thun and Gilete -1990, Froehlich -2008, Froehlich -1995 and Xu and Zhang. The main reason why the author chooses these equations is that the breach section or geometry passes over the dam site topographic opening or canopy of the dam site. But the equations mentioned above fit to the canopy so choose them for the dam breach analysis.

3.5. Hydraulic Model Development

The 2D dam break analysis model is thoroughly covered in this section. The prediction of the dam breach hydrograph and the downstream routing of this hydrograph at crucial points are both part of the 2D dam breach analysis model. Hydraulic modeling (HEC-RAS V6.3.1) is being used for this analysis method. ArcMap V10.8.2 software is utilized for all GIS-related operations in addition to the hydraulic modeling, and RAS Mapper is used to assist with the visualization and mapping of the flood inundation mapping.

3.5.1. HEC-RAS Development

For a whole network of natural and artificial channels, overbank/floodplain areas, levee-protected areas, etc., HEC-RAS is made to do one-dimensional and two-dimensional hydraulic computations. The main HEC-RAS capabilities are described in the paragraphs that follow. HEC-RAS.V6.3.1 is used in this study in creating RAS Layers to extract information essential for hydraulic modeling. It is used to extract elevation data from DEMs over HEC-RAS V6.2 itself. In previous versions of HEC-RAS an Arc-GIS extension software called HEC-GeoRAS was used to create these layers, but in the recent version all the river channel features can be done over HEC-RAS V6.3.1.

The most recent HEC-RAS version (5.0 or later) now supports two-dimensional flow routing. The user can completely model the downstream area for a dam break study using 1D elements (Cross sections and storage areas); or they can combine 1D and 2D elements; as a combination of 1D and 2D elements (cross sections, storage areas, and 2D flow areas); or the entire downstream area can be modeled as a 2D flow area. (USACE, 2023)

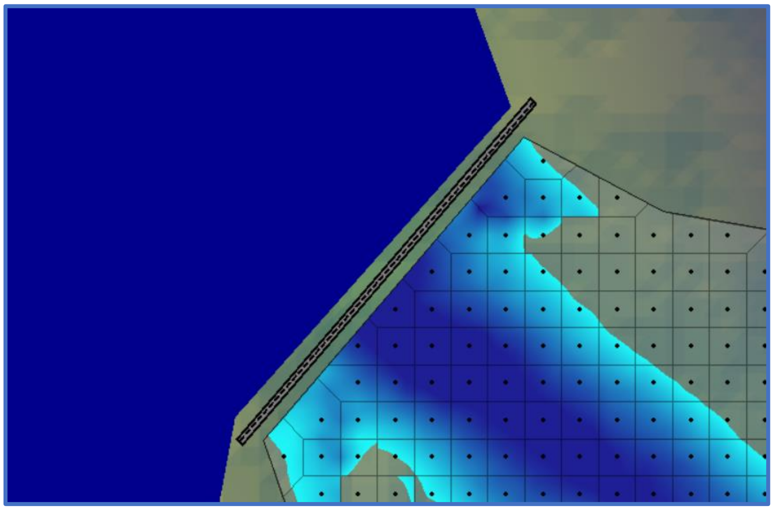
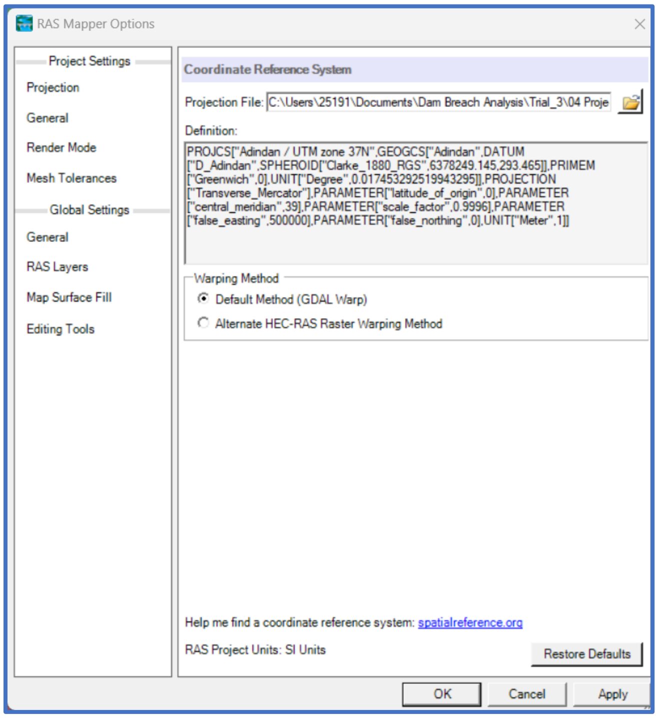


Figure 3. 12: Fato River “SA/2D Area Conn” by Ras Mapper HEC-RAS V6.3.1

3.5.1.1. Setting the Spatial Reference Projection

Before proceeding with setting the projection we need to download the specific site projection file which is Adindan zone 37N from the following address (https://www.spatialreference.org/ref/epsg/20137/). First Select the Project and then Set Projection menu item from the RAS Mapper menu bar to set the project's spatial reference system. When the Set Projection option is selected, the window shown below will appear (Figure 3.10).



Projection File: C:\Users\25191\Documents\Dam Breach Analysis\Trial_3\04 Proje

Definition:

```
PROJCS["Adindan / UTM zone 37N",GEOGCS["Adindan",DATUM
["D_Adindan",SPHEROID["Clarke_1880_RGS",6378249.145,293.465]],PRIMEM
["Greenwich",0],UNIT["Degree",0.0174532925199432955]],PROJECTION
["Transverse_Mercator"],PARAMETER["latitude_of_origin",0],PARAMETER
["central_meridian",39],PARAMETER["scale_factor",0.9996],PARAMETER
["false_easting",500000],PARAMETER["false_northing",0],UNIT["Meter",1]]
```

Warping Method

● Default Method (GDAL Warp)

○ Alternate HEC-RAS Raster Warping Method

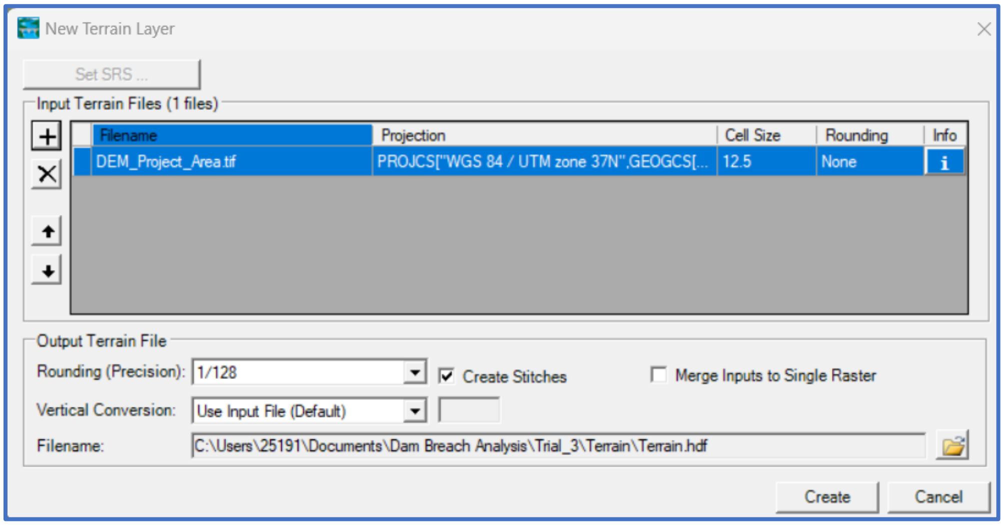
Help me find a coordinate reference system: spatialreference.org

RAS Project Units: SI Units

Figure 3. 13: Editor to set the RAS project's spatial reference system to Adindan.

3.5.1.2. Loading Terrain Data and Making the Terrain Model

The next stage is to load the terrain data that will be applied to build the 2D HEC-RAS model's terrain model. To develop a new terrain data set (terrain model), from the RAS Mapper main window (Figure 3.14), right click on the Terrains layer and from the shortcut menu select Create New RAS Terrain. The New Terrain Layer dialog (Figure 3.14) opens. This dialog allows the us to provide a name for the new Terrain Layer (Filename field, the default name is “Terrain”); selecting a directory for storing the terrain (Folder button); defined the elevation precision of the new terrain data layer (Rounding (Precision) field, 1/128 is the default for SI units); and select the files to be used in building the new terrain layer (Plus (+) button).



Input Terrain Files (1 files)

Filename	Projection	Cell Size	Rounding	Info
DEM_Project_Area.tif	PROJCS["WGS 84 / UTM zone 37N",GEOGCS[...	12.5	None	i

Output Terrain File

Rounding (Precision): 1/128 ☑ Create Stitches ☐ Merge Inputs to Single Raster

Vertical Conversion: Use Input File (Default)

Filename: C:\Users\25191\Documents\Dam Breach Analysis\Trial_3\Terrain\Terrain.hdf

Figure 3. 14: Fato dam breach model terrain Layer dialog.

3.5.1.3. 2D flow areas

Drawing a Polygon Boundary for the 2D Area

Making the 2D Area's Polygon Boundary With the help of the geometry editing capabilities in HEC-RAS Mapper, a 2D flow area polygon was first built to represent the 2D area's perimeter. Bringing in topography information and aerial photos into HEC-RAS Mapper first is the best approach to accomplish this in HEC-RAS. The terrain and background images have assisted in figuring out where to draw the 2D flow area boundaries.

To create the 2D flow area in HEC-RAS Mapper, do the following:

Follow these steps to build the 2D flow area in HEC-RAS Mapper:

1) To edit the geometry, perform a right click on the layer and choose Edit Geometry.
2) Select the Perimeters layer and expand the 2D Flow Areas layer.
3) Use the Add New Feature tool to draw the border.

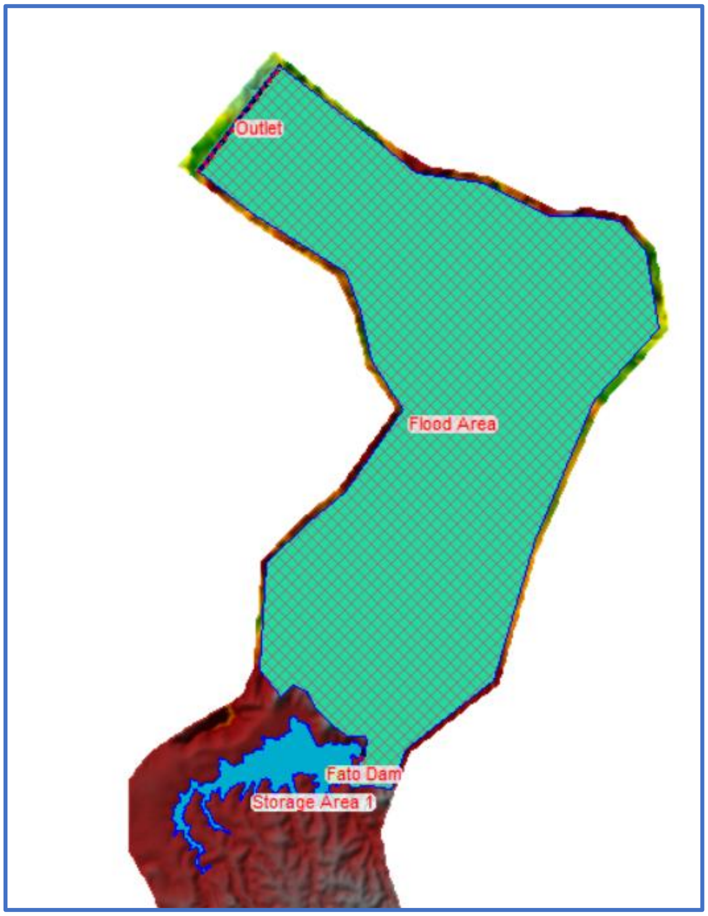


Figure 3. 15: 2D flow area polygon of Fato dam downstream.

Create the 2D Computational Mesh

The boundary for which 2D computations will take place is defined by the 2D flow area. Within the 2D flow area, a computational mesh (or computational grid) is produced. The following three characteristics apply to each cell in the computational mesh (Figure 3-16).

- **Cell Center:** The central part of the cell's computation. Here, the cell's water surface height is calculated. The centroid of the cell and the cell center are not always the same.
- **Cell Faces:** The cell boundary faces are as follows. Faces often consist of straight lines, but they can also have several points, as the 2D flow area's outside edge.
- **Cell Face Points:** The endpoints of the cell faces are known as the cell Face Points (FP). The 2D flow area is connected to 1D elements and boundary conditions using the Face Point (FP) numbers for the outer border.

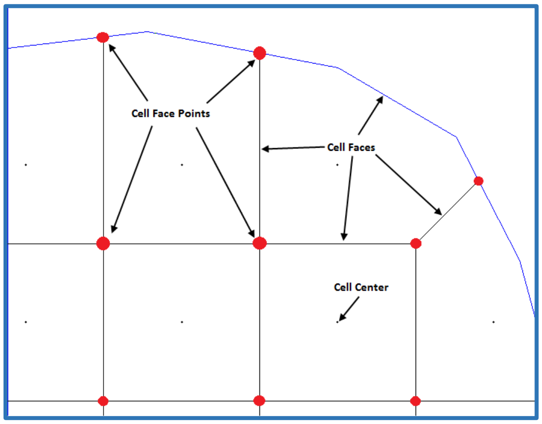
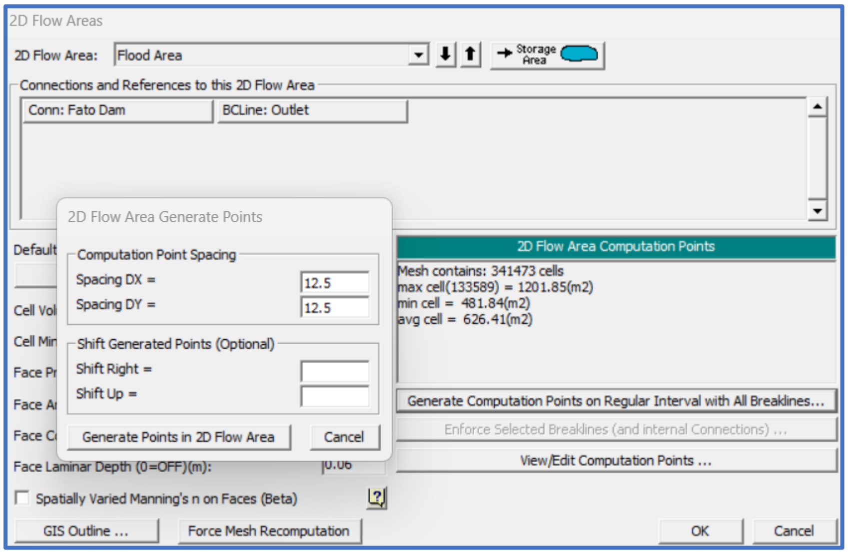


Figure 3. 16: HEC-RAS 2D modeling computational mesh terms.

The next stage is to start generating the computational mesh after the perimeter of the 2D Flow Area has been defined using polygons. The user should generally choose a base cell size for the 2D flow region. Keep in mind that you will be able to refine the mesh with breadlines and refinement regions where needed. To create the base mesh, do the following: Make the basis mesh by performing the following:

1) Decide which 2D Flow Area polygon you want to alter.
2) Select Edit 2D Area Properties by performing a right-click on the 2D Flow Area.
3) A window similar to the one in Figure 3.17 below will appear once that menu item has been chosen.



2D Flow Area: Flood Area

Spacing DX = 12.5
Spacing DY = 12.5
Shift Right =
Shift Up =

Face Laminar Depth (0=OFF)(m): 0.06

☐ Spatially Varied Manning's n on Faces (Beta)

```
Mesh contains: 341473 cells
max cell(133589) = 1201.85(m2)
min cell =  481.84(m2)
avg cell =  626.41(m2)
```

Figure 3. 17: 2D Flow Area Editor for Fato Dam.

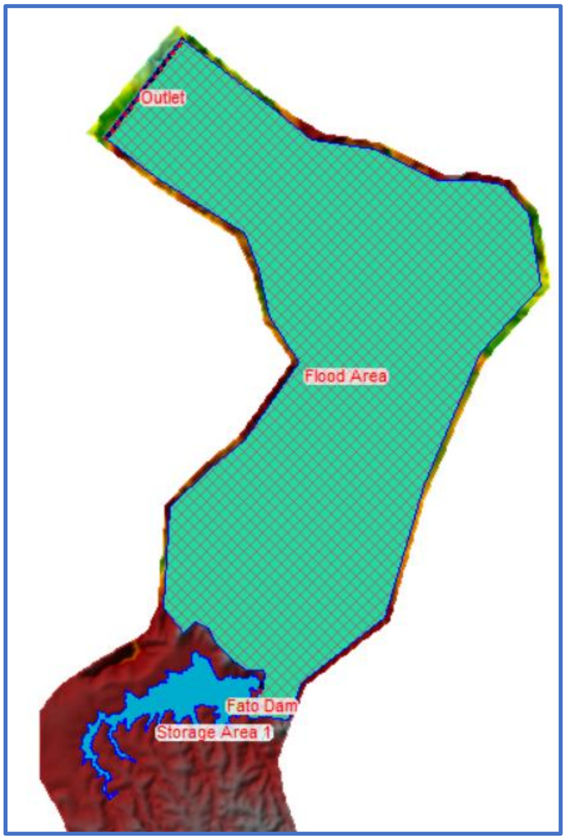


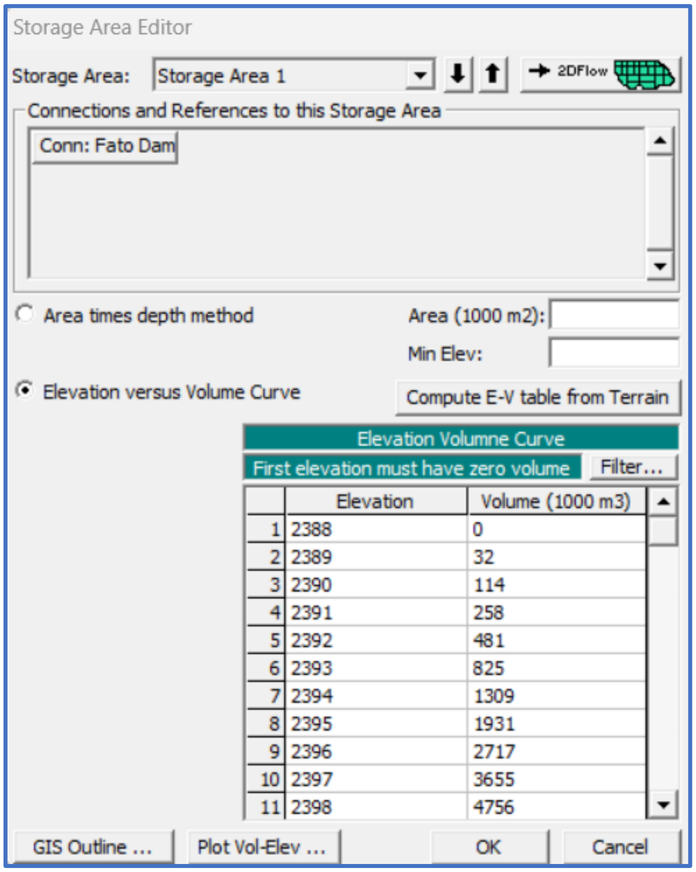
Figure 3. 18: 2D flow area of Fato Dam mesh (Using HEC-RAS V6.3.1)

3.5.1.4. Storage Area

Lake-like areas known as storage areas are places where water can be diverted to or from. Storage areas can be found lateral to a reach, at the start of a reach (as an upstream boundary to a reach), or at the conclusion of a reach (as a downstream boundary to a reach). A lateral structure connection can be used to link storage areas to a river reach. Using a storage area connection, one storage area can be joined to another storage area. Weir and gated spillways, weir and culverts, only a weir, or a linear routing option are all possible storage area connections. For Fato dam case storage area to 2d flow area connection was used.

To add storage area, use the geometric editor's top-most storage area drawing tool. After choosing the storage area drawing tool, drawing the storage area was started by clicking the left mouse button. Moving the mouse and single clicking gained extra points. a polygonal representation of the storage space. Double-clicking the left mouse button allowed you to complete designing the storage area. The storage space will then be filled up with a blue hue after the first and last points are connected. then prompted to give the storage area's name.

After the storage area was drawn and labeled, must enter data to describe the storage area. This is accomplished with the storage area editor, which is one of the buttons on the left side of the geometric editor. Pressed the storage area editor button and the following editor will appear:


Storage Area Editor

Storage Area: Storage Area 1

Connections and References to this Storage Area

Conn: Fato Dam

○ Area times depth method

Area (1000 m2):

Min Elev:

● Elevation versus Volume Curve

Elevation Volumne Curve

First elevation must have zero volume

	Elevation	Volume (1000 m3)
1	2388	0
2	2389	32
3	2390	114
4	2391	258
5	2392	481
6	2393	825
7	2394	1309
8	2395	1931
9	2396	2717
10	2397	3655
11	2398	4756

Figure 3. 19: Storage Area Editor with Fato dam data over HEC-RAS V6.3.1

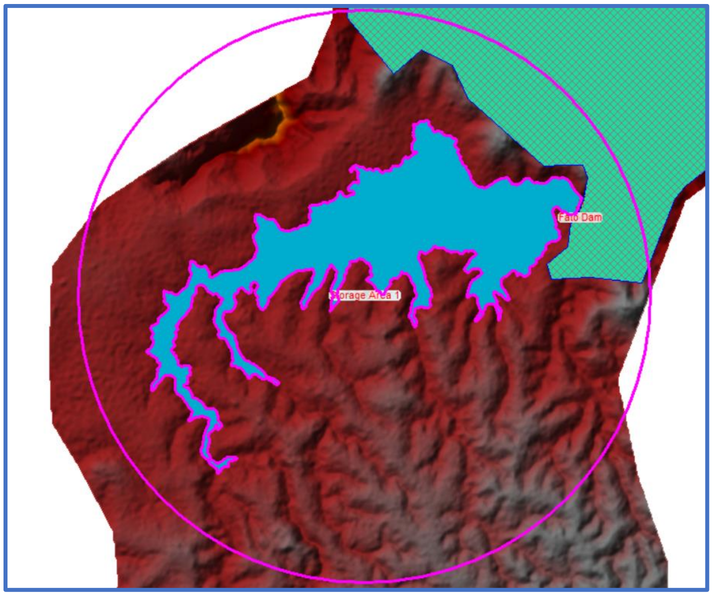
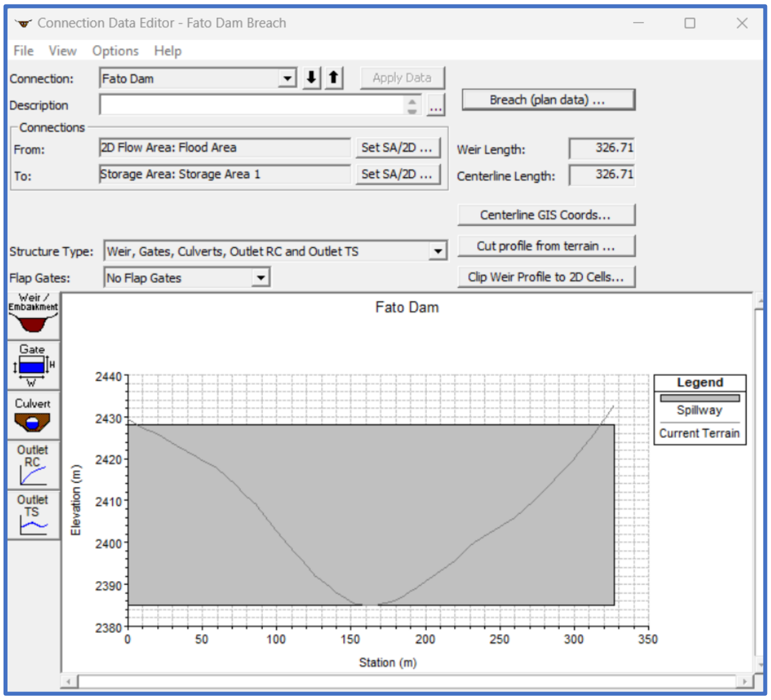


Figure 3. 20: Storage Area of Fato dam modeled over HEC-RAS V6.3.1

3.5.1.5. Storage Area 2 Dimensional SA/2D Area Conn

Two storage areas, two 2D Flow Areas, or a storage area to a 2D Flow Area can be connected by a hydraulic structure using a storage area to a 2D Flow Area connection (SA/2D Area Conn). To influence how flow moves from one series of cells to another series of cells, a hydraulic structure can be placed in the middle of a 2D Flow Area using the SA/2D Area Conn tool. There are three available Structure Types for the SA/2D Area Conn: Weir, Gates, Culverts, Outlet RC, and Outlet TS are the first option. The second is linear routing; the third is a bridge that is internal to a 2D Flow Area. To establish a hydraulic connection between two storage areas, 2D flow areas; or inside of a 2D Flow Area, press the "SA/2D Area Conn" button at the top of the geometric data window. Simply press the left mouse button once to begin creating the centerline of the hydraulic structure after activating the storage area connection drawing tool. Left-click repeatedly to digitize the hydraulic structure's centerline, then double-click to finish. Drawing this structure should be done while looking upstream, or in the positive flow direction, from left to right. If this structure is drawn between two storage areas, a storage area and a 2D flow area, or between two 2D flow areas, it needs to define the From and to locations within the SA/2D Area Connection editor. If the structure is drawn completely inside of a single 2D Flow Area, then the To and from connections are automatically set to the 2D area. Once a SA/2D Area Connection is drawn, must enter information describing the hydraulics of the connection. You can do this by clicking the SA/2D Area Conn editor button on the geometric data editor's left side. When this button is pressed, the following window appeared:


Connection Data Editor - Fato Dam Breach

Connection: Fato Dam

Description:

Connections

From: 2D Flow Area: Flood Area

To: Storage Area: Storage Area 1

Weir Length: 326.71

Centerline Length: 326.71

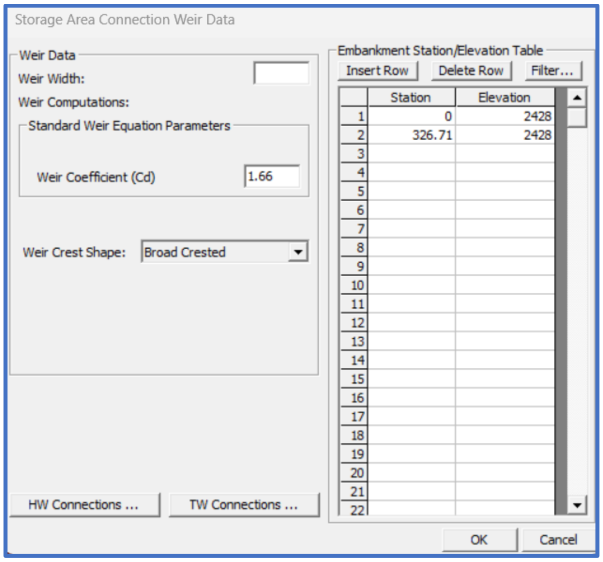
Structure Type: Weir, Gates, Culverts, Outlet RC and Outlet TS

Flap Gates: No Flap Gates

Figure 3. 21: SA/2D Area Conn editor of Fato dam modeled over HEC-RAS V6.3.1

As shown in the figure 3.22, hydraulic structure connecting a storage area to a 2D Flow Area. First entered a description for the SA/2D area connection. Next the From and To connections were set correctly. After that selected the Structure Type from the drop-down box. The window displayed below will open when the weir/embankment editor is chosen.

Entered data for the Embankment editor at a minimum. In this project data was entered for embankment. When the weir/embankment editor is selected, the window shown below will appear.



Weir Width:

Weir Coefficient (Cd) 1.66

Weir Crest Shape: Broad Crested

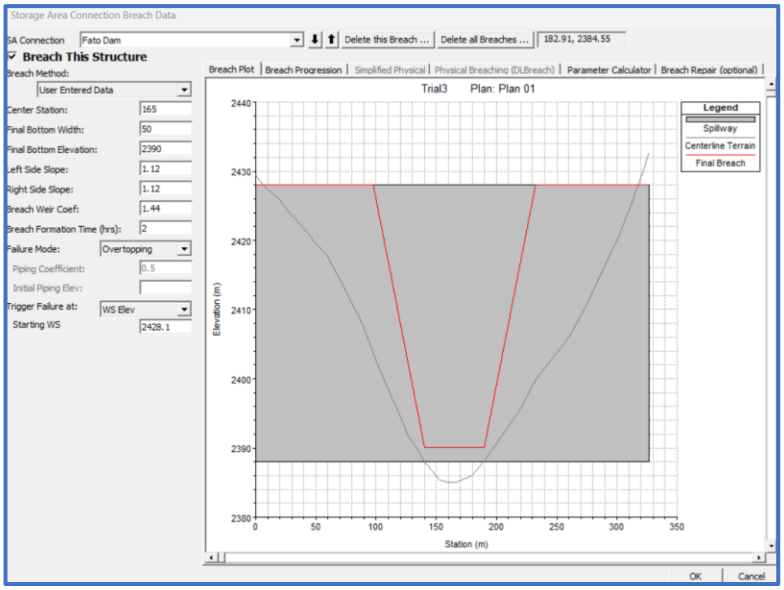
	Station	Elevation
1	0	2428
2	326.71	2428

Figure 3. 22: SA/2D Area Conn Embankment data of Fato dam modeled over HEC-RAS V6.3.1

To complete the data for the weir/embankment, entered a Weir Width (used only for drawing the schematic); a Weir Coefficient (used in the weir flow calculations); a Weir Crest Shape (used to define submergence requirements and aid in the computation of the weir coefficient); and the Station/Elevation Points that describe the top of the weir/embankment profile. The weir/embankment can have up to 500 points to describe the profile. The program used all of the information entered for calculating weir flow between the two storage areas. After all of the data is entered, simply pressed the OK button to have the data accepted by the program.

3.5.1.6. User Entered Breach Data

The User Entered Data method requires the user to enter all of the breach information (i.e., breach size, breach development time, breach progression. The Breach (Plan Data) button can be found on the Inline Structure editor first. Second, the user can choose Dam (Inline Weir) Breach from the Options menu in the Unsteady Flow Simulation Manager. The following window will show up when either choice is chosen (Figure 3.23).


Storage Area Connection Breach Data

SA Connection: Fato Dam

☑ Breach This Structure

Breach Method: User Entered Data

Center Station: 165

Final Bottom Width: 50

Final Bottom Elevation: 2390

Left Side Slope: 1.12

Right Side Slope: 1.12

Breach Weir Coef: 1.44

Breach Formation Time (hrs): 2

Failure Mode: Overtopping

Piping Coefficient: 0.5

Initial Piping Elev:

Trigger Failure at: WS Elev

Starting WS: 2428.1

Figure 3. 23: User entered breach parameters of Fato Dam

3.5.1.7. Roughness Values

To accurately represent open channel flows, Manning's roughness coefficient, often known as Manning's n, must be estimated. The components of surface friction resistance, form resistance, wave resistance, and resistance resulting from flow unsteadiness are all included in the roughness coefficient as an empirical measure. (Y. WANG, January 2005). In investigating natural river flows, particularly unstable channel network flows, direct estimation of the roughness coefficient is practically impossible. The roughness coefficient (n) in natural channels is difficult to determine in field. Various factors affecting the values of roughness coefficients were presented by (Chow, 1959). Thus, the friction slope should be viewed as a crucial parameter whose value must be carefully chosen.

The Fato dam project area contributes the highest proportion of agricultural production and still there is a high potential for rain fed and irrigated agriculture. The vegetation cover is also better compared to other high land parts of the country. It can be observed that most of the farmers have experience of retaining scattered trees on crop land in addition to patches of forest, wood, and shrub vegetation. (ECDSCo., 2016)

However, at present most of the area are under threat of soil erosion that has constrained the agricultural potential of the area coupled with other limiting factors. It is evident that sheet and rills, eroded soil, rock outcrops, and gully erosion on steep cultivated land, open shrub lands and grazing lands and the like exist in the project area. (ECDSCo., 2016)

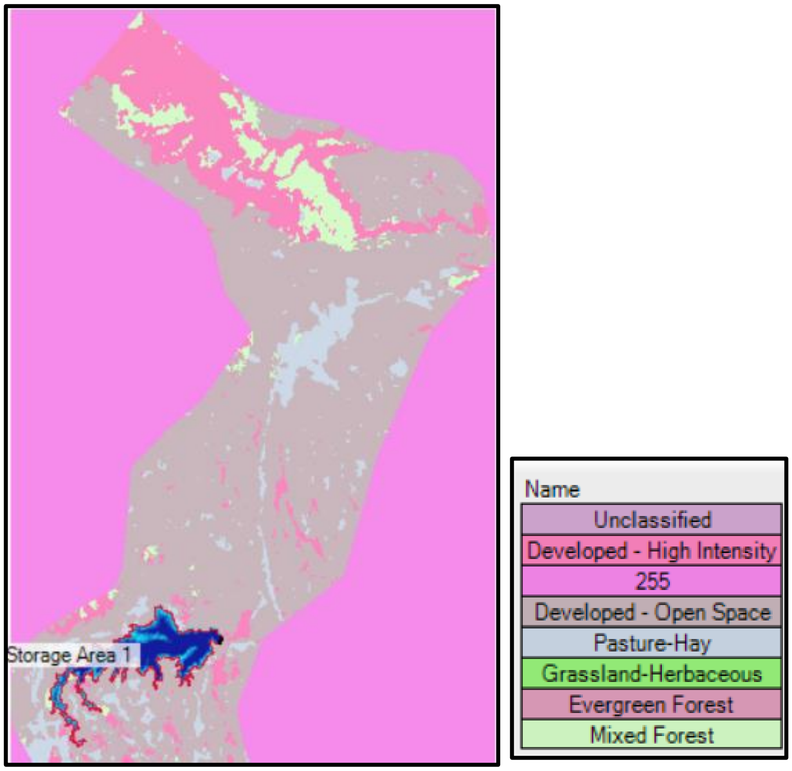
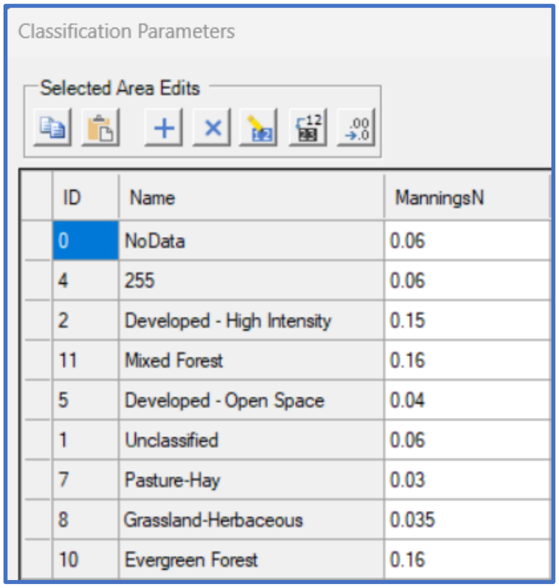


Figure 3. 24: The main LULC of Fato River downstream of the dam

The Manning's n values for the flood area downstream of the Fato dam range from 0.06 to 0.16 to reflect the dynamic and intense nature of a dam breach flood wave as well as diverse material kinds within the flood area. These values depend on the features of the flood plain. Manning's n-values were based on values for similar conditions that had been published. (Chow, 1959).



ID	Name	ManningsN
0	NoData	0.06
4	255	0.06
2	Developed - High Intensity	0.15
11	Mixed Forest	0.16
5	Developed - Open Space	0.04
1	Unclassified	0.06
7	Pasture-Hay	0.03
8	Grassland-Herbaceous	0.035
10	Evergreen Forest	0.16

Figure 3. 25: Assigned Manning's Value for the ESRI LULC 10m resolution.

3.5.1.8. Unsteady Flow Data

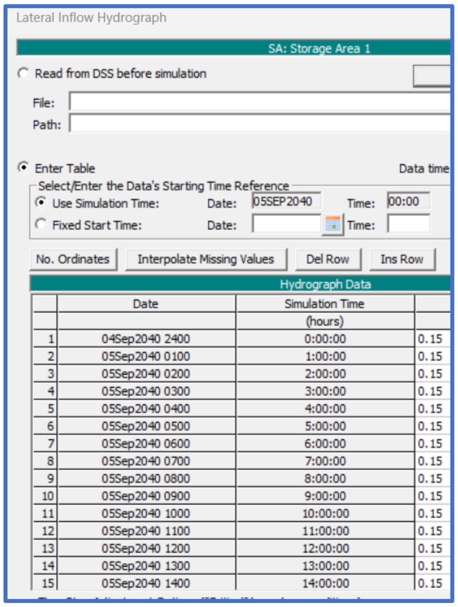
Unsteady Flow Data

The user must set the initial flow and storage area conditions at the start of the simulation, as well as boundary conditions at all of the system's external boundaries and any desired internal locations. The Unsteady Flow Data Editor’s Boundary Conditions tab is initially selected in order to enter boundary conditions. The table will automatically contain the locations of the system's rivers, reaches, and river stations. To insert boundary conditions, first choose a cell in the table for a specific place, then choose the desired boundary condition type for that site. Not all boundary condition types are applicable everywhere. When a user selects a specific location in the table, when a user selects a specific location in the table, the program will automatically grey out the boundary condition kinds that are not relevant. Additionally, users can enter locations for internal boundary conditions. Select the Add RS or Add Storage Area buttons to add an extra boundary condition location. Users can add more river station locations for boundary conditions by clicking the Add RS button. The user can add storage area locations for the insertion of boundary conditions by clicking the Add Storage Area button.

Boundary Conditions

A) Flow Hydrograph

The most frequent usage of a flow hydrograph is as an upstream boundary condition, while it can also be used as a downstream boundary condition. Pressing the flow hydrograph button will bring up the pane in Figure 3.26. The user has two options, as shown: they may either enter the hydrograph ordinates into a table or read the data from a HEC-DSS (HEC Data Storage System) file. The reservoir routing results for fixing the water level during overtopping was done using HEC-HMS the results were put under annex of this thesis work.



SA: Storage Area 1

○ Read from DSS before simulation

File:

Path:

● Enter Table

Select/Enter the Data's Starting Time Reference

● Use Simulation Time: Date: 05SEP2040 Time: 00:00

○ Fixed Start Time: Date: Time:

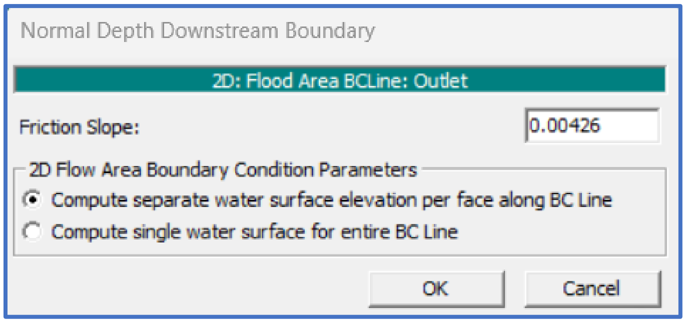
Hydrograph Data

	Date	Simulation Time (hours)	
1	04Sep2040 2400	0:00:00	0.15
2	05Sep2040 0100	1:00:00	0.15
3	05Sep2040 0200	2:00:00	0.15
4	05Sep2040 0300	3:00:00	0.15
5	05Sep2040 0400	4:00:00	0.15
6	05Sep2040 0500	5:00:00	0.15
7	05Sep2040 0600	6:00:00	0.15
8	05Sep2040 0700	7:00:00	0.15
9	05Sep2040 0800	8:00:00	0.15
10	05Sep2040 0900	9:00:00	0.15
11	05Sep2040 1000	10:00:00	0.15
12	05Sep2040 1100	11:00:00	0.15
13	05Sep2040 1200	12:00:00	0.15
14	05Sep2040 1300	13:00:00	0.15
15	05Sep2040 1400	14:00:00	0.15

Figure 3. 26: PMF hydrograph of the inflow boundary condition

B) Normal Depth

The Normal Depth option can only be used with the downstream boundary condition of an open-ended reach. A stage is calculated for each computed flow using Manning's equation. The user must specify a friction slope (slope of the energy grade line) for the reach near the boundary condition in order to apply this method. Although it can be difficult to predict in advance, the friction slope is typically well approximated by the slope of the water's surface. The normal bed slope in the vicinity of the boundary condition location is frequently used to determine the friction slope. As with the rating curve option, it is advisable to set this form of boundary condition far enough downstream so that any mistakes it makes won't have an impact on the results at the study reach.


Normal Depth Downstream Boundary

2D: Flood Area BCLine: Outlet

Friction Slope: 0.00426

2D Flow Area Boundary Condition Parameters

● Compute separate water surface elevation per face along BC Line

○ Compute single water surface for entire BC Line

Figure 3. 27: Average bed slope of exit Guder River channel boundary condition

3.5.1.9. Computational Time Step

The guideline for computational time step is to divide the time of hydrograph rise by 20, Therefore: Dam Breach Modelling and Flood Mapping, a Case Study of Fato Dam

- $\Delta t < \frac{T_r}{20}) = 60/20\ \Delta t < 5min$

Where, TR= Time of rise of hydrograph to be routed, Minimum 1hr is considered considering the breach development time.

Moreover, the stability and accuracy of the model will be accomplished by choosing a time step that fulfills the courant requirement.

The estimated courant number is:

- $C_r = \frac{V_w * \Delta t}{L} \leq 1.00$

Therefore, the Computational time steps will be:

- $\Delta t \leq (\frac{\Delta x}{V_w})$

Where, V_w= Flood wave speed, Taking the maximum velocity 15m/s

- Cr = Courant Number; 1.0 is a desirable number.

Δx = Distance between cross sections, average of 30m-by-30m mesh grid on a terrain layer, Take 30m, hence,

- $\Delta t = \frac{30}{15}\ \Delta t = 2$ Second

2 second is considered as computational time step for the simulation.

3.6. Hazard Classification Process

According to their possible incremental affects or consequences in the event of failure, all dams with a safety risk are categorized using the possible Consequences Classification (PCC). (ENTRO, December, 2014)

Table 3.4 provides a classification scheme that should be applied when choosing design criteria and the degree of care required of dam owners and Approved Dam Engineers. To identify dams where the risk is greater than others, estimates of probable additional consequences of dam failure are divided into categories. Both domestic and foreign, or trans-boundary, issues should be considered. The dam's Potential Consequences Classification (PCC) should be the highest ranking determined from the many risk categories displayed. (ENTRO, December, 2014)

Please refer table 2.8 for the ENTRO hazard classification.

The success of the Emergency Action Plan (EAP) should be taken into account while evaluating the possible loss of life. For instance, if a natural flood is being examined, the specific characteristics of the flood and evacuation scenarios should be taken into account to guarantee the right level of protection is supplied. A Population at Risk (PAR) assessment can be used as a starting point to cautiously estimate the possible loss of life, classify the dam, and establish the necessary safety standards and procedures. (ENTRO, December, 2014)

When classifying a dam, environmental, cultural, and third-party economic damages should be calculated individually and taken into account for both national and global or trans-boundary concerns. (ENTRO, December, 2014)

The most serious potential effects, such as losses to human life, infrastructure, the economy, society, or the environment and culture, should be used to decide the dam class. (ENTRO, December, 2014)

A broad classification for the dam system should be utilized for the purposes of general management oversight, as well as design, construction, inspection, maintenance, and surveillance programs, depending on the failure scenario that would have worse consequences: either a breakdown on a sunny day or a flood failure. (ENTRO, December, 2014)

For selecting relevant design criteria for brand-new projects, the PCC should be employed. When designing certain components for a site, the effects of component failure may be taken into account independently in order to avoid specific failure modes and their combinations. (ENTRO, December, 2014)

To make sure that the classification of the consequences has not changed due to changes in the population or industry or due to a better understanding of the social or environmental repercussions, all dams' classification should be reviewed during their required safety evaluations. (ENTRO, December, 2014)

3.6.1. Physical Factors Affecting the Consequences of Dam Failure

There are many factors which can affect the potential consequences of dam failure. These can include:

- The dam height (the higher the dam, the higher the potential energy of the water and the faster the water may escape).
- The volume stored behind the dam (the bigger the storage the bigger the damage potential); The nature of the stored materials (e.g., water versus mine tailings or toxic wastes)
- The shape and hydraulic characteristics of the downstream valley which affects the nature and extent of potential flooding.
- The downstream conditions, particularly habitation or public areas and the valley environment which would be exposed to the effects of dam failure; The effects to a community of depriving them of the stored water which may be critical for water supply.
- Other factors may affect the likelihood of a dam failure if they are not correctly dealt within the investigation, design, construction, or operational phases of the dam’s life. These may include:
    - Difficult or unusual foundation conditions.
    - Construction materials.
    - Proximity to active faults.
    - Catchment use (e.g., forestry operations with associated risk of debris);
    - Proximity to volcanic hazards; and
    - Landslides in the reservoir area.

3.6.2. Inundation Area

Utilizing the software that is currently accessible, the inundation area must be determined. The inundated area must stretch downstream for at least as far as it is expected the flood wave will go in a 12-hour period, or until it is projected that the flood wave height will have dropped to 600 mm above the river flow without a dam break. Care must be taken in routing flows through lakes to ensure that the effect of natural levees and the like is considered. (ENTRO, December, 2014)

The inundated area for dams under 10 meters high may be interpreted as the dam's crest level, which is lowering by 1 meter per kilometer as the wave travels downstream. Once the flood wave height is reduced to 600 mm above the river flow without a dam break, it will be considered to have traveled at least that far downstream in 6 hours. (ENTRO, December, 2014)

3.6.3. Population at Risk and Loss of Life

The Population at Risk is the estimate of the total number of people likely to be within the inundated area at any time. It is not an estimate of the likely number of casualties due to a dam failure as no reduction is made for those likely to escape or survive the flood. (ENTRO, December, 2014)

It is necessary to assess people are at risk from dam failure by:

- Counting the number or residences (including hotels, hostels, camps etc.) within the estimated inundated area and multiplying by the average occupancy rate determined for the area in question or by 3.5.
- Estimating the average number of people (in vehicles, walking, fishing etc.) on any bridge within the inundated area (including the dam where this functions as a bridge) during daylight hours and multiplying by 0.7.
- Estimating the number of people recreating, or employed, on the river within the inundated area during the busiest eight hours of a day and multiplying by 0.4.
- Obtaining the total number of students and staff at schools within the inundated area and multiplying by 0.3.
- Obtaining the total number of workers at industries within the area and multiplying by 0.4.
- Estimating the maximum number of patients, staff and visitors and hospitals, clinics and similar institutions and multiplying by 0.8.
- All other places where people could congregate (e.g., churches, temples, mosques, sporting grounds, community halls etc.) the average number of people should be calculated using a similar method which would produce the average over time of the number of people at risk.

By aggregating all these figures, the Population at Risk (PAR) is determined. The People at Risk (PAR) and the accessibility and efficacy of warning and evacuation systems should be taken into account when estimating the potential "Loss of Life" to be utilized in Table 3.4. (ENTRO, December, 2014)

According to estimates of potential loss of life using various tools, such as DSO-99-06, A Procedure for Estimating Loss of Life Caused by Dam Failure, USBR 1999, or the more recent Interim Guidelines for Estimating Life Loss for Dam Safety Risk Analysis, RCEM - Reclamation Consequence Estimating Methodology, USBR, February 2014, a more detailed classification may be appropriate in critical areas with a large permanent People at Risk (PAR). (ENTRO, December, 2014)

3.6.4. Procedure for Estimating Loss of Life

The procedure for estimating loss of life can be broken into various steps. Briefly, the steps are as follows:

Step 1: Determine Dam Failure Scenarios to Evaluate
A determination needs to be made regarding the failure modes to evaluate. For two scenarios, such as the failure of the dam during a significant flood that over-tops the dam and the failure of the dam with a full reservoir under typical weather circumstances, loss of life estimates may be required. (DSO, 1999)

Step 2: Determine Time Categories for Which Loss of Life Estimates Are Needed

Seasonality or day of the week considerations can affect how many people are at risk downstream from particular dams. For instance, campgrounds might not be used at all during the winter and be extremely busy during the summer, particularly on weekends. The variety of time periods (season, day of the week, etc.) assessed should show the shifting population at risk and the floodplain's fluctuating use. Each study should include a day category and a night category for each dam failure scenario considered because the time of day might affect both when a warning is issued and the number of individuals at risk. (DSO, 1999)

Step 3: Determine When Dam Failure Warnings Would be Initiated.

Identify the time at which warnings of dam failure would start. The most crucial step in predicting the number of deaths brought on by a dam failure is probably figuring out when warnings would start to be issued. Data from U.S. dam breaches that have occurred since 1960, as well as other occurrences like the Vajont Dam in Italy, the Malpasset Dam in France, and the Saint Francis Dam in California, were used to develop "Guidance for Estimating When Dam Failure Warnings Would be Initiated." These dam failure data were evaluated, and the results showed that the likelihood of timely warnings increased when the dam failed during the day, in the presence of a dam tender or others, and when the drainage area above the dam was big or the reservoir had room for flood storage. When a dam failed at night or without a dam tender or other nearby observers present, timely warnings of its impending failure were less likely to be given. When the reservoir was able to swiftly fill and cover the dam, such as when the drainage area was tiny or the reservoir had little to no space for flood storage, dam failure warnings were also less likely to occur. Despite the paucity of actual evidence, it seems that the breakdown of a concrete dam is less likely to be preceded by timely warning. Although warnings of impending dam failure are regularly issued for earthfill dams, this is not the case for concrete dam failure. (DSO, 1999)

When a dam breakdown warning might be sent depends on the availability of emergency action plans, upstream or damsite instrumentation, or the need for on-site monitoring during dangerous occurrences. These and other risk-reduction measures and programs should be taken into consideration when making assumptions about when a warning is issued. (DSO, 1999)

Step 4: Determine Area Flooded for Each Dam Failure Scenario

Determine the area flooded in each scenario where a dam fails. For each dam failure scenario, a map or some other description of the flooded area must be given in order to calculate the number of persons at risk. New dam-break studies may occasionally need to be created. To decrease study costs, existing maps should be utilized as much as possible. It will be necessary to determine whether the flooding from the various failure scenarios for which loss of life estimates are required is reflected in the currently published or draft inundation maps. For instance, a dam failure inundation map based on a failure caused by dam overtopping may not accurately depict the flooding caused by a piping failure with a much lower reservoir level. (DSO, 1999)

Uncertainty results from analyses that rely on dam failure inundation studies and maps are estimating: 1) The time for the breach to form, 2) The shape and extent of the breach, and 3)

Downstream hydraulic factors are all necessary for dam break modelling. Variations in estimates of these parameters can result in changes in flood width, flood depth and flood wave travel time. This can lead to uncertainty in the: 1) Population at risk, 2) Warning time and 3) Flood severity. (DSO, 1999)

Step 5: Estimate the Number of People at Risk for Each Failure Scenario and Time Category

The number of people who are at risk should be determined for each failure scenario and time group. The population at risk (PAR) is the total number of people who are present in the floodplain of a dam failure before any warnings are issued. As a general rule, "Take a picture and count the people." People who are at danger fluctuate throughout the day. (DSO, 1999)

Depending on the season, day of the week, and hour of the day that the failure happens, the PAR is likely to change. Use any and all sources, including census data, fieldwork, aerial photography, telephone interviews, topographic maps, and other information, to produce a reasonable estimate of floodplain usage. (DSO, 1999)

Step 6: Apply Empirically Based Equations or Method for Estimating the Number of Fatalities

Two distinct articles were created with methods for calculating the number of lives lost due to dam failure. "Assessing Threat to Life from Dam Failure" was published by Brown and Graham in 1988. The article "Predicting Loss of Life in Cases of Dam Failures and Flash Floods" by DeKay and McClelland was published in 1993." A summary of the procedures, and loss of life estimating equations presented by each pair of authors is presented below. (DSO, 1999)

The Brown and Graham procedure uses equations that were derived from the analysis of 24 dam failures and major flash floods. The concepts contained in the Brown and Graham paper were incorporated into Reclamation’s "Policy and Procedures for Dam Safety Modification Decision making" (1989) and equations from this document are presented below. (DSO, 1999)

The definition of the warning time employed in the equations is the amount of time that has passed between the start of a public evacuation notice and the arrival of dangerous floods reaching the population at risk. Therefore, the amount of time it takes for floodwater to reach the community or vulnerable population must be considered when calculating the warning time. (DSO, 1999)

When the warning period is under 15 minutes:

$$\text{Loss of Life} = 0.5*(\text{PAR})$$

When warning period falls between 15 and 90 minutes:

$$\text{Loss of Life} = \text{PAR}^{(0.6)}$$

When the warning period exceeds 90 minutes:

$$\text{Loss of Life} = .0002*(\text{PAR})$$

3.6.5. Infrastructure, Economic and Social Factors

The inundated region, the population at risk, the environmental repercussions, the impact on transportation and industry, and an estimation of the consequences for the affected communities are examined in order to identify the infrastructure, economic, and social elements. The ability of the communities to recover and to come up with temporary solutions will be taken into account in this study. Any health issues within this heading should be taken into account. (ENTRO, December, 2014)

The estimation of economic risk will consider:

- The value of the dam and associated assets and the cost of clean-up and replacement.
- The value of buildings, bridges, power lines, pipelines, communicating assets and the contents of buildings and the estimate the cost of clean-up and replacement.
- The cost of temporary accommodation required prior to replacement of the assets.
- The loss of production arising from the loss of assets.
- The loss of production arising from the loss of water for irrigation or water supply.
- This will consider any temporary works which could be constructed.
- The loss of production arising from the loss of power generated. This should not exceed the cost of alternative power being obtained.
- The economic impact on the disruption to communication and transport infrastructure.

The total economic consequences should estimate the total cost to the nation on the loss of the assets and the Economic Risk Classification.

3.6.6. Environmental and Cultural Factors

By looking at the area that has been flooded and taking into account the dam breach flow velocity that has been calculated using computer models or assessed using other methods, the environmental and cultural variables are identified. The flood wave will typically be anticipated to contain a significant amount of debris, which will lead to the total clearance of debris and the demolition of any building susceptible to an inundation of more than 600 mm. (ENTRO, December, 2014)

The sort of plant and environment that were lost, as well as comparable examples seen throughout the world, will determine how quickly the damage will recover. (ENTRO, December, 2014)

The importance of environmental losses should be evaluated in terms of how long it would take and whether or not it would be possible to restore the ecosystem. It would be unrealistic, if not impossible, to arrive at a single numerical estimate describing the scope of the devastation given the complexity of environmental and cultural loss. Because of these factors, a qualitative evaluation might be more appropriate. (ENTRO, December, 2014)

4. BREACH PARAMETER ESTIMATIONAND PEOPLE AT RISK (PAR) DETERMINATION

4.1. Introduction

A hydrologic event failure mode, or PMF, and a sunny day event failure mode, or non-hydrologic event, are the occurrences being assessed in this paper. The following list contains the data needed to estimate the Fato dam breach:

Table 4. 1: Different Fato Dam Reservoir Levels and Volume Capacity

Important Pool Levels	Elevation(m)	Volume (MCM)
Stream Bed Level	2394.82	0.003
Full Supply Level	2429.00	99.141
Top of Flood Control	2430.20	106.828
Top of Dam Crest Level	2432.00	123.342

The dam's crest measures 276.34 meters in length and 10 meters in width. The dam can be raised above the riverbed a maximum of 40.1m. The average embankment slopes upstream and downstream are 1.7H:1V and 1.65H:1V, respectively. The dam's embankment is made of a clay core with an impervious rock fill. Both 1000mm thick hand-placed riprap without filter layers and dumped rock riprap with filter are used to defend the upstream and downstream slope faces of the dam.

4.2. Estimating Dam Breach Parameters

For the research of the Fato Dam breach analysis, two types of failure were examined. Five regression (empirical) equations were used to estimate the parameters for the dam breach. Which are: The most widely used regression equations in the process of estimating dam breach parameters today are those from Macdonald and Langridge-Monopolis (1984), Froehlich (1995), Froehlich (2008), Von Thun and Gillette (1990), and Xu & Zhang.

4.2.1. Overtopping Mode of Failure

The main channel centerline (at a height of 2425.0 meters) is assumed to be the failure location for overtopping failure in this study. The following findings were obtained from the computation of breach parameters using the five techniques used in this investigation.

Using All the breach parameter estimation methods the bottom width of the breach section is wider than the dam opening at that level therefore considering the dam section be the breach area and the only difference will be the formation hour therefore of all the five empirical equations is Von Thun and Gillete is 0.99hrs. Therefore, for the overtopping case same breach width with this short formation hour is used since the shorter the formation hour results in the larger the flood wave and impact.

4.2.2. Piping Mode of Failure

The results for the breach bottom width and breach development time using Macdonald and Langridge Monopolis (1984) were 250m and 2.4hrs. These values, according to Von Thun and Gillette (1990), were 129m and 0.99hrs. 131m and 1.86 hours from Froehlich (2008). 156 minutes and 2.13 hours from Froehlich (1995) and 109 minutes and 4.01 hours from Xu & Zhang.

Table 4. 2: Summary of Breach Parameters Estimation

Methods	Bottom Breach Width (m)	Breach Side Slope (H:1V)	Breach Failure Time (Hrs.)	Peak Outflow (Q_{max}, m^3/sec)
Overtopping Case				
Custom_2023	30	3	1.5	30,128.28
Froelich (1995)	156	1.4	2.13	29,553.50
Froelich (2008)	131	1	1.86	27,293.92
Von Thun & Gillete	129	0.5	0.99	32,940.26
Xu & Zhang	109	1.12	4.01	22,404.28
Piping Case				
Froelich (2008)	88	0.7	1.59	20,224.88
Xu & Zhang	60	0.66	3.76	11,799.72

4.3. Representation (Setup) of Fato/Guder River and Dam in HEC-RAS Model

4.3.1. Fato/Guder River

The Homacho Ammunition Factory is located in the downstream portion of the Fato River Basin, which has a relatively steep drop of 810 meters over a distance of about 36 kilometers. Around Guder Town, the river flows through a number of deep and zigzagging valley portions interspersed with vast, flat reaches. At Homacho Ammunition Factory, the river abruptly changes flow direction. Due to these characteristics of the river, it is impossible to predict the flood depth and velocity using a 1D dam breach model, hence the researcher chose to use a 2D dam breach model instead. The distance between the cross sections and the simulation time step affect the stability of the HEC-RAS model. HEC-RAS permits values as low as 0.1 seconds. To ensure reliability of the calculations, the Fato dam model employs a two-second time step. Other, larger time steps were tested, but time steps greater than one minutes not found to result in stable calculations.

Since Fato dam breach model is 2D there is no need to define cross sections at different sections of the river reach but define the flood area mesh. But after computation of the 2D flow area flood defined the river so that be easy to extract the result from 2D depth and velocity results. While defining the river section through the river section followed the lowest level of the river somehow to capture the bed elevation of the river as shown in figure 4.1.

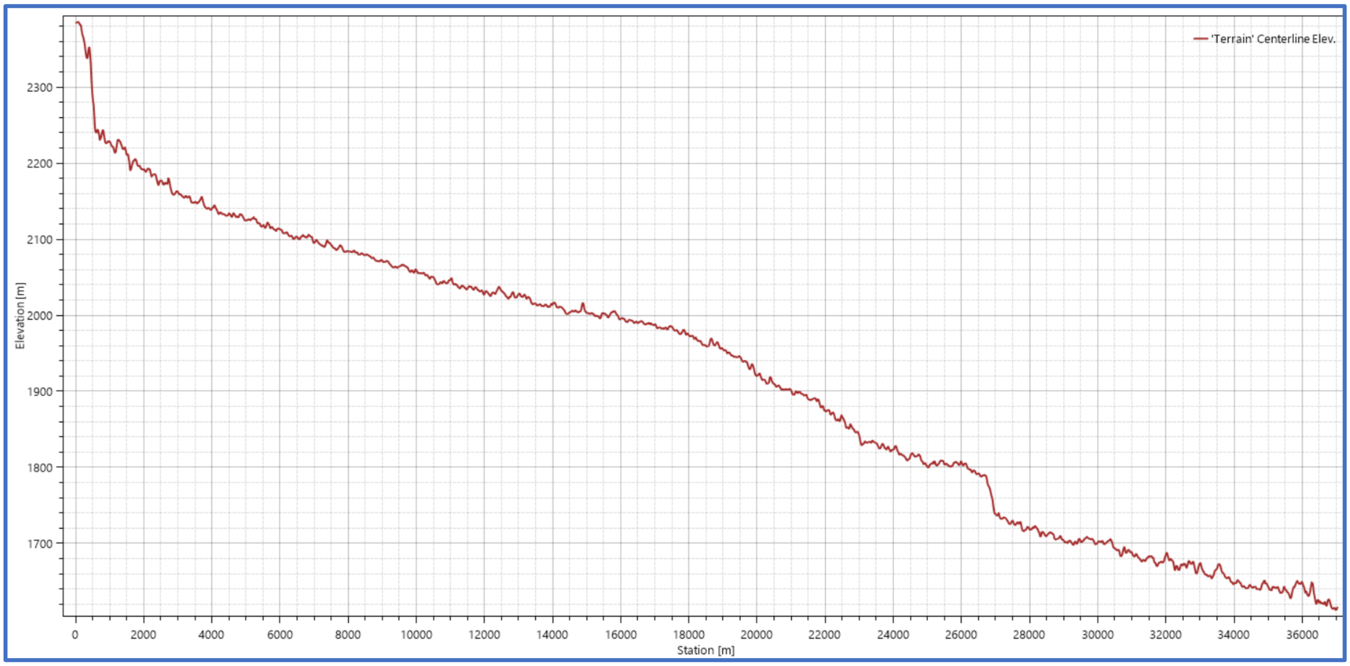


Figure 4. 1: Fato River topography from dam site to 36KM downstream

4.3.2. Fato Dam Breach Data

By adding crucial information and presumptions on the Fato dam, the reservoir, and the breach's most likely characteristics, as illustrated in Table 4.3, HEC-RAS 6.3.1 enables the modeling of the breach process. According to the Froehlich equations, the breach development time for the Fato embankment dam was calculated to be 0.99 hours for the overtopping mode of collapse. The breach is 30 meters wide at the bottom. The river was assumed to be moist at the start of the simulation with an initial flow of 1 m3/sec for the HEC-RAS model in order to have a stable model. Since Fato dam crest length is 276.34m with the two abutments side slope of 1H:3V (Steep abutments), the values from the three empirical equations at the lowest level of breach passes over the exiting ground width. Since both abutments are rocks then the researcher took the exiting width at that level as a bottom width and the two ridge slopes as a breach slope estimated the breach width. While doing so in all the 5-breach parameter estimation equation the bottom width is more than the existing and forced to take all the dam section, so the only difference is going to be the formation hour of all the 5 equations. For the same breach section, the shortest formation hour will have a great impact on the downstream in inundating large area. Therefore, for this study the formation of the dam breach is taken to be 1.5hours as custom type of breach parameter.

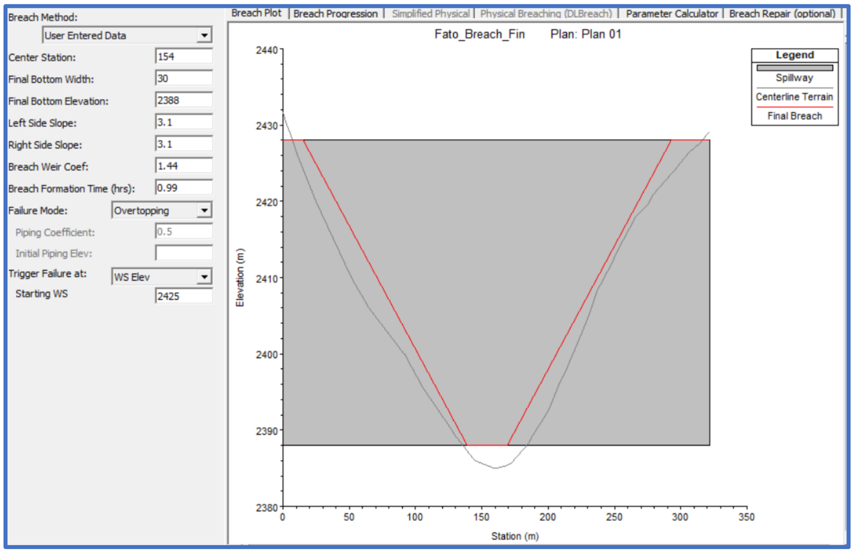
Table 4. 3: Fato Dam breach model data for overtopping mode of failure.

ITEM	VALUE
River station of Dam	26.91Km upstream of Guder Fall
Pilot Flow	0.15 m3/sec
Centre station	154m
Final Bottom Width	30m
Final Bottom Elevation	2388m
Left side slope	3
Right side slope	3
Full formation time	1.5hrs
Failure mode	Overtopping
Trigger Failure	2.3*PMF

4.4. HEC-RAS Unsteady Flow Analysis Parameters

An unsteady flow calculation is done in HEC-RAS 6.3.1 to mimic the process of a dam breach. The dam breach was started at the beginning of the simulation, which lasted for 24 hours. In the HEC-RAS model, the Fato Dam is treated as a SA/2D flow region. The plan Figure 4.2 and the information in Table 4.3 both reveal the dam's dimensions.

For the unsteady flow analysis, boundary condition and beginning condition data must be entered. The upstream river reach above the dam's boundary conditions were entered into the Fato River's inflow hydrograph. With a slope of 0.0426 m/m, the downstream boundary condition at the Homacho Ammunition Factory was a normal depth boundary condition. Additionally, the Fato River's initial flow in the basin at the beginning of the simulation period was set at 0.15 m3/sec.

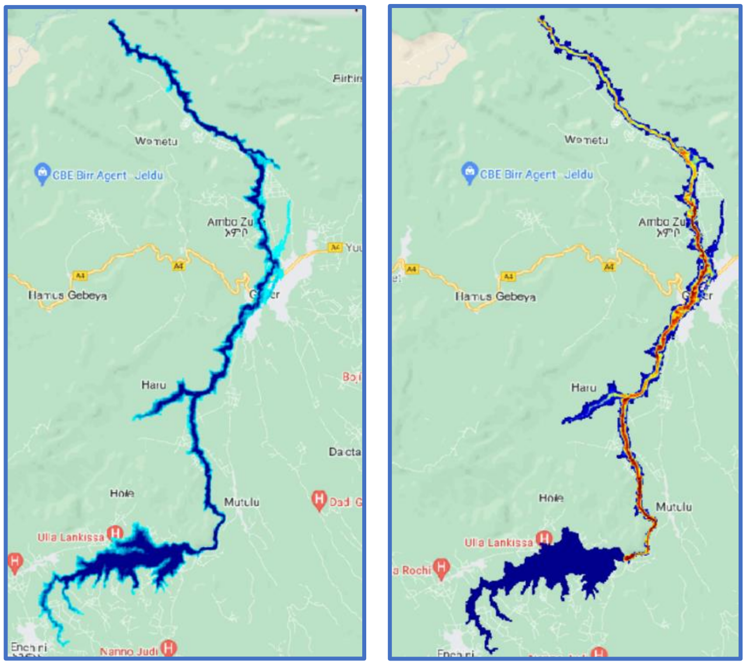


Breach Method: User Entered Data
Center Station: 154
Final Bottom Width: 30
Final Bottom Elevation: 2388
Left Side Slope: 3.1
Right Side Slope: 3.1
Breach Weir Coef: 1.44
Breach Formation Time (hrs): 0.99
Failure Mode: Overtopping
Piping Coefficient: 0.5
Initial Piping Elev:
Trigger Failure at: WS Elev
Starting WS 2425

Figure 4. 2: Dam Breach Plan of Fato Dam, on Fato River (HEC-RAS 6.3.1)

4.5. Flood Inundation Mapping Process

To build a flood inundation map from Fato Dam and downstream along Fato River using the Ras Mapper, the maximum water surface elevations from the PMF and fair weather (Maximum Pool) scenarios were integrated with topography data retrieved in ArcGIS. The distances between the PMF and the fair-weather breach were totaled. The flooding maps for the two dam break scenarios would then be mapped.



(a) (b)

Figure 4. 3: (a) Inundation Area flow depth; (b) Inundation area velocity distribution

4.6. Population at Risk Computation

Since we have already got the flood inundation area then the researcher exported the raster file for the maximum water depth result in HEC-RAS 6.3.1 during the simulation time and imported it to ArcGIS 10.8.2. Over ArcGIS 10.8.2 adding the base map of world map imagery as a background then digitized all the houses that fall within the inundation area and got the number of the houses in the inundation area. Two critical places are taken into consideration for the estimation of the population at risk; one is the Guder town while the other one is the Homacho Ammunition Engineering Complex (HAEC). The digitized houses within the inundation area are shown in the following figures.

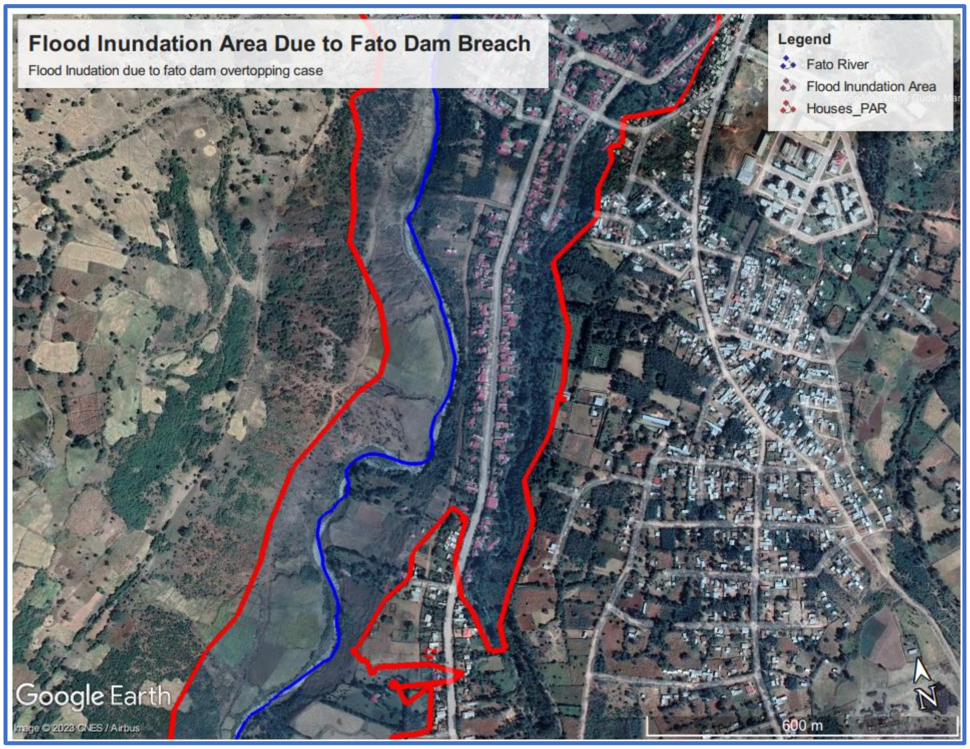


Figure 4. 4: Entry of Guder Town flood inundation area homes digitized.

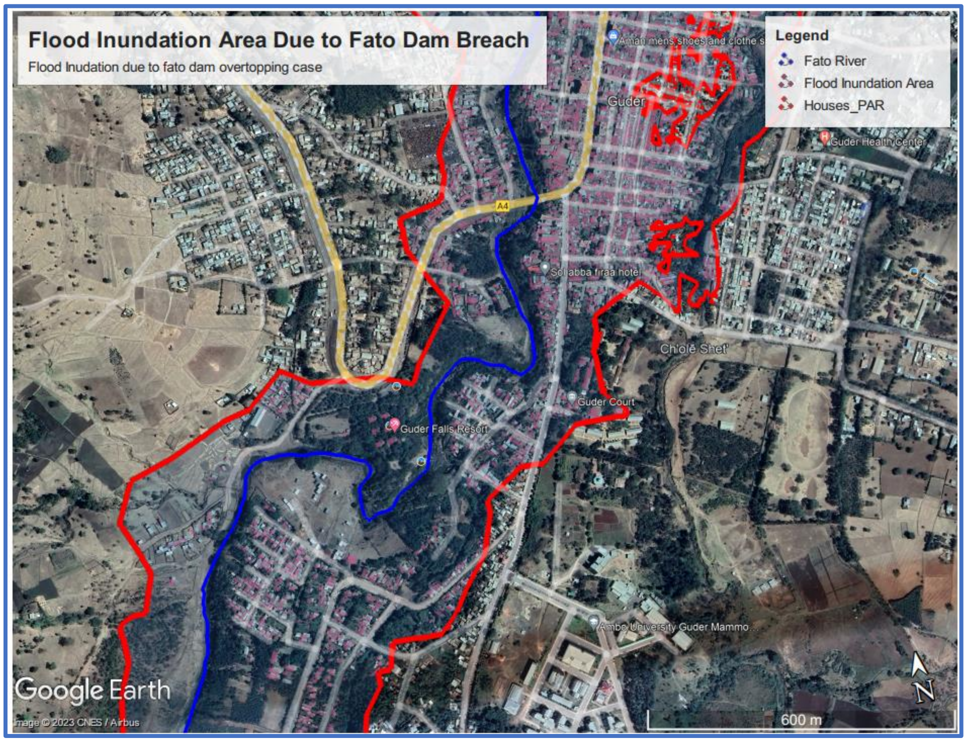


Figure 4. 5: Middle part of Guder Town flood inundation area digitized homes

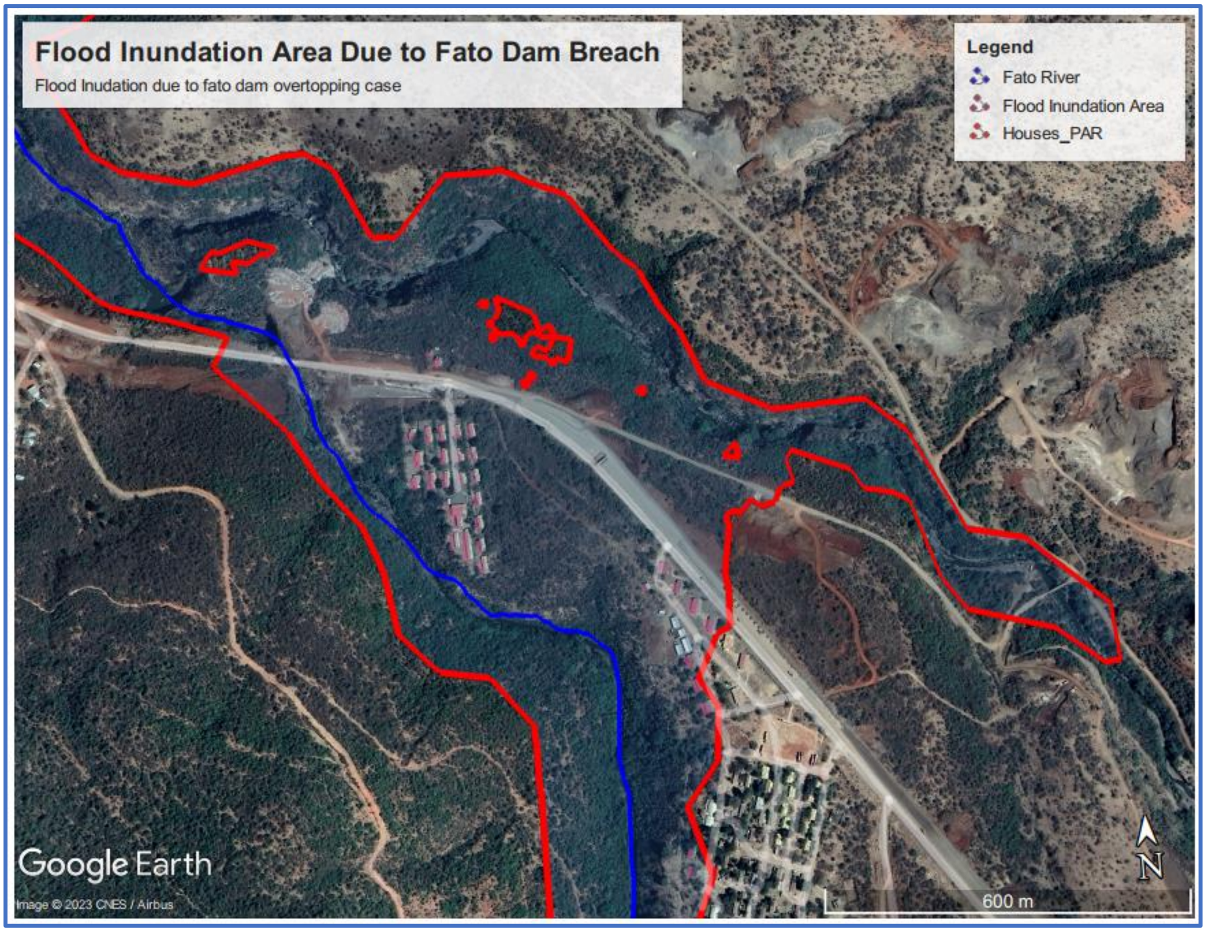


Figure 4. 6: HAEC entry flood inundation area digitized homes

After digitizing these homes within the inundation area then classified them based on their purpose into the following categories. From google map identified the hotels, churches, schools, and residential houses and justified these places being there in person except the Fato dam and first bridge close to the dam around 9KM due to security problem there. The occupant's number is identified with an interview with the school director of Guder high school and Mosque Sheik. The two hotels were estimated by the author being there. But here over google earth only these 2 hotels were shown but when the author visited the area there were a lot of hotels not defined over google earth, therefore the need for population and houses census project throughout our country need to be in near future.

- Hotels
- Churches
- Schools
- Residential Houses

Based on the inventory of the people in the inundation area the occupants within the flood area are summarized as per the following table 4.4.

Table 4. 4: PAR (Population at Risk) computation for all categories

Category	Name	Occupants	Factor	PAR
Hotels	Sol Abba Fira Hotel	25	1	25
	Macca and Tulema Hotel	16	1	16
Recreational Places	Guder Falls Resort	52	1	52
Industries	Guder Court	37	1	37
	Guder Bus Station	347	1	347
Churches	Guder Muslim Mosque	29	1	29
Schools	Guder Secondary School	3,723	0.3	1117
Residential Houses	Houses	2,229	3.5	7,802
Total PAR				9,425

5. RESULTS AND DISCUSSIONS

5.1. Dam Breach Parameters Result

The dam breach parameters were estimated using five most currently used regression equations, i.e., Macdonald and Langridge-Monopolis (1984), Froehlich (1995), Froehlich (2008), Von Thun and Gillette (1990) and Xu & Zhang. For the case of overtopping failure, the bottom breach width for five methods were 250m, 156m, 131m, 129m and 109m, respectively. Their breach development times were 2.4hrs, 2.13hrs, 1.86hrs, 0.99hrs and 4.01hrs. Similarly, the breach width and breach development times for the above five methods in the case of piping failure were: 168m, 97m, 88m, 117m and 85m and 2.02hrs, 1.79hrs, 1.59hrs, 0.48hrs and 1.98hrs, respectively.

5.2. Dam Breach Simulation Result

The breach outflow hydrographs for several failure modes (overtopping and piping) breach scenarios for fully clogged morning glory spillway conduit as well as for flood events 2.3*PMF have been calculated from the findings of the HEC-HMS model. The results of the five techniques were used to calculate the maximum breach discharges and time to peak flow of the two failure scenarios, namely overtopping and piping for the Fato dam.

Table 5. 1: Different inflow hydrograph and breach status of Fato Dam results from HEC-HMS

Multiplier factor of PMF	Maximum Water Level	Overtopping
PMF	2430.60	No
1.5PMF	2431.20	No
2PMF	2431.80	No
2.2PMF	2431.90	No
2.3PMF	2432.10	Yes
2.4PMF	2432.30	Yes
Fully Clogged spillway conduit	>2432	Yes

Since Fato dam crest elevation is found at 2432m the dam will be overtopped if 2.3PMF inflow flood get into the reservoir, which so unlikely to happen. Therefore, we can say that Fato is barely susceptible to overtopping failure except for informal settlement following the right abutment fault line of foundation. The hydraulic engineer provided large freeboard to the dam which makes it costly but on the safety side safe dam. Keeping in mind the PAR downstream of Fato dam the hydraulic engineers design assumption is appreciable.

5.2.1. Overtopping Mode of Failure Results

The maximum breach outflow and arrival time to peak breach outflow results obtained from HECRAS model simulation at dam for five methods in case of hydrologic (2.3*PMF) breach scenarios are given in table 5.2, 5.3, 5.4, 5.5,5.6 and 5.7.

Table 5. 2: PMF Event Breach Peak Outflows for Overtopping Mode of Failure

Methods	Peak Flow (m3/sec)	Peak Time	Time to peak
Custom (2023)	30,128.28	9/5/2040 1:29	1Hr and 29 Minutes
Froelich (1995)	29,553.50	9/5/2040 0:49	49 Minutes
Froelich (2008)	27,293.92	9/5/2040 0:59	59 Minutes
Von Thun & Gillete	32,940.26	9/5/2040 0:52	52 Minutes
Xu & Zhang	22,404.28	9/5/2040 1:27	1Hr and 27 Minutes

Table 5. 3: Maximum flow, time to peak, rate of flow and flood height for some Fato River stations below Fato Dam (Custom_2023)

Methods	Peak Flow m3/sec	Peak Time	Time to peak	Peak Velocity (m/sec)	Maximum Depth (m)
At Dam Site (0.1KM)	30,132.43	9/5/2040 1:29	1Hr and 29 Minutes	12.61	21.16
At Bridge (9KM)	29,301.91	9/5/2040 1:37	1Hr and 37 Minutes	13.96	19.22
Guder Town (15KM)	24,728.50	9/5/2040 1:56	1Hr and 56 Minutes	9.79	18.84
At Homacho (25KM)	24501.639	9/5/2040 2:11	2Hr and 11 Minutes	7.90	29.87*

*Depression Area

Fato Dam will overtop in the case of full 2.3*PMF by 0.1m by using the breach parameters obtained from the above five methods in the model. As per the reservoir routing for Fato dam using HEC-HMS for different flood magnitudes the reservoir will be overtopped if the inflow of the dam is 2.3*PMF. The dam is factiously breached on 05 September 2040 at 00:00 time. Deducting the peak times from the breach time one can get the time to peak in hours.

Table 5. 4: Maximum flow and time of peak flow for some Fato River stations below Fato Dam. (Froelich_1995)

Methods	Peak Flow m3/sec	Peak Time	Time to peak	Peak Velocity (m/sec)	Maximum Depth (m)
At Bridge (9KM)	29,510.23	9/5/2040 0:58	58 Minutes	14.00	19.36
Guder Town (15KM)	26,677.93	9/5/2040 1:28	1Hr and 28 Minutes	11.74	18.59
At Homacho (25KM)	26,543.60	9/5/2040 1:41	1Hr and 41 Minutes	8.61	27.01*

*Depression Area

Table 5. 5: Maximum flow, time to peak flow, and flood depth for some Fato River stations below Fato Dam (Froelich_2008)

Methods	Peak Flow m3/sec	Peak Time	Time to peak	Peak Velocity (m/sec)	Maximum Depth (m)
At Bridge (9KM)	27,253.74	9/5/2040 1:09	1Hr and 09 Minutes	13.35	18.18
Guder Town (15KM)	25,393.58	9/5/2040 1:34	1Hr and 34 Minutes	11.63	17.43
At Homacho (25KM)	25,282.66	9/5/2040 1:49	1Hr and 49 Minutes	8.7	29.70*

*Depression Area

Table 5. 6: Maximum flow, time to peak flow, and flood depth for some Fato River stations below Fato Dam (Von Thun & Gillete)

Methods	Peak Flow m3/sec	Peak Time	Time to peak	Peak Velocity (m/sec)	Maximum Depth (m)
At Bridge (9KM)	32,825.47	9/5/2040 1:00	1Hr and 00 Minutes	14.32	20.51
Guder Town (15KM)	28,843.66	9/5/2040 1:21	1Hr and 21 Minutes	12.05	17.84
At Homacho (25KM)	28,629.68	9/5/2040 1:33	1Hr and 33 Minutes	8.95	29.30*

*Depression Area

Table 5. 7: Maximum flow, time to peak flow, and flood depth for some Fato River stations below Fato Dam (Xu & Zhang)

Methods	Peak Flow m3/sec	Peak Time	Time to peak	Peak Velocity (m/sec)	Maximum Depth (m)
At Bridge (9KM)	22,088.40	9/5/2040 1:41	1Hr and 41 Minutes	12.66	17.26
Guder Town (15KM)	20,020.50	9/5/2040 2:13	2Hr and 13 Minutes	10.51	16.96
At Homacho (25KM)	19,907.01	9/5/2040 2:30	2Hr and 30 Minutes	7.72	26.43*

*Depression Area

5.2.2. Overtopping Breach Outflow Verification

As a test of reasonableness, the computed peak output from the model is compared to the regression equations and the envelope curves of historical failures. Based on data from 14 previously failed and documented dam breakdowns, the following curve was created. We can't declare with certainty if it fits or not based on the data from only 14 dams, but we can assess how logical the results of the Fato dam breach analysis are.

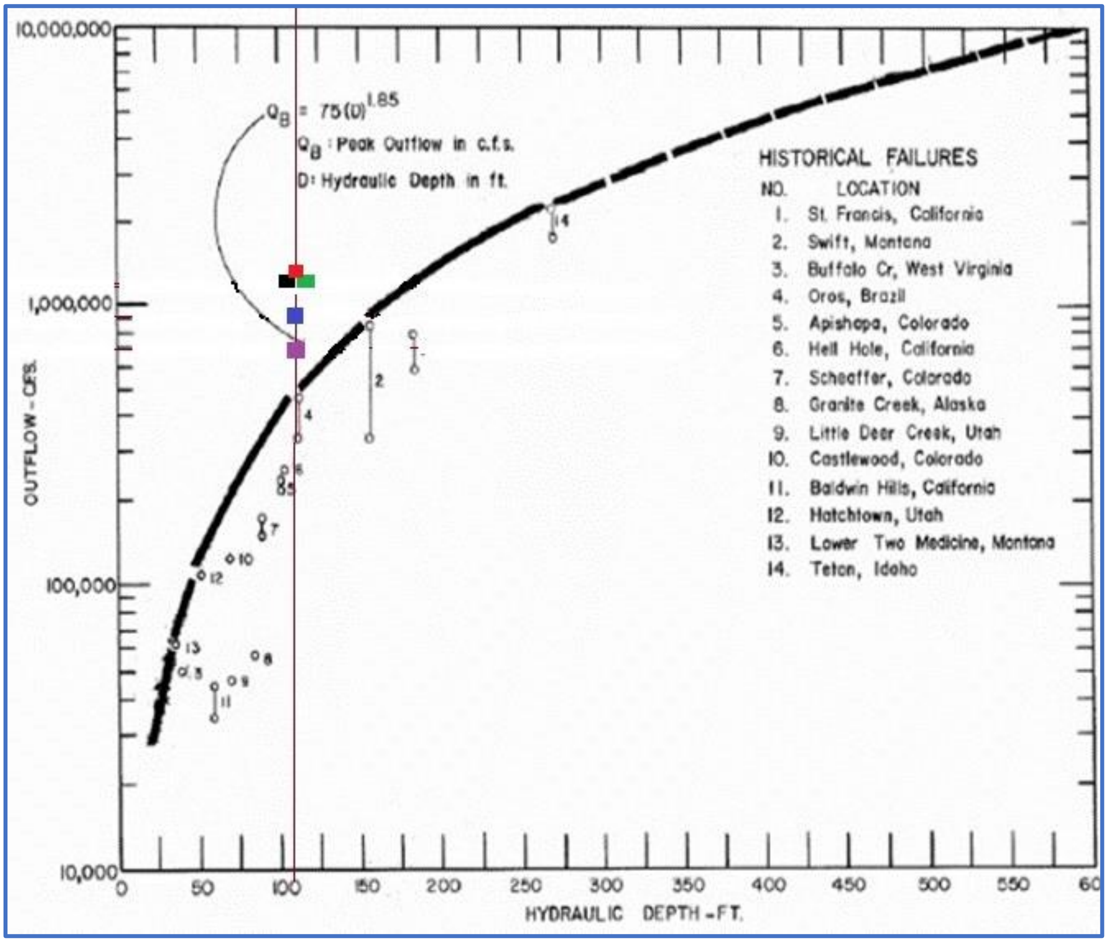


Figure 5. 1: Verification of Fato dam overtopping outflows using historic outflow rates envelope.

Table 5. 8: Different empirical equations hydraulic depth vs peak flow results in imperial units

Sign	Empirical Equation	Hydraulic Depth (feet)	Peak Flow (cfs)
🟥	Custom (2023)	109.19	1,063,970.15
⬛	Froelich (1995)	114.76	1,043,672.00
🟦	Froelich (2008)	110.82	963,875.69
🟩	Von Thun & Gillete	106.59	1,163,274.30
🟪	Xu & Zhang	118.34	791,199.68

Table 5. 9: HEC-RAS results and peak discharge equations summary of Fato dam

Methods	Peak flow from HEC RAS (m3/s)	Peak flow from the equation (m3/s)	Difference (m3/s)	Difference (%)
Custom (2023)	30,128.28	12,524.04	17,604.24	58.43
Froelich (1995)	29,553.50	13,732.36	15,821.14	53.53
Froelich (2008)	27,293.92	12,873.56	14,420.36	52.83

Von Thun & Gillete	32,940.26	11,978.86	20,961.40	63.63
Xu & Zhang	22,404.28	14,534.46	7,869.82	35.13

Since no technique is consistent according to the validation and reasonableness work, the author chose to take the worst-case situation into consideration, which is the Von Thun and Gillete method. Additionally, it is noted that all of the breach model results are significantly inflated when compared to the others. In reality, all modes of failure involving piping have plainly produced smaller magnitudes than overtopping.

5.2.3. Piping (Sunny Day) Mode of Failure Results

The maximum breach outflow and arrival time to peak breach outflow results obtained from HEC - RAS model simulation at dam for two methods in case the of non-hydrologic (piping) breach scenario are given in tables 5.10 and 5.11. Since the bottom width from the empirical equations generate wider length when related to canopy opening width. The only equation that fit the canopy width was Froehlich (2008) and Xu & Zhang. Since the geology of the both the left and right abutment is fractured rock assumed it can’t be easily eroded by the piping water through the dam when reaching dam abutments. Therefore, the piping failure mode is analyzed only for these 2 cases.

Table 5. 10: Maximum flow, time to peak flow, and flood depth for some Fato River stations below Fato Dam (Froelich (2008))

Methods	Peak Flow m3/sec	Peak Time	Time to peak	Peak Velocity (m/sec)	Maximum Depth (m)
At Bridge (9KM)	20,111.59	9/5/2040 1:42	1Hr and 45 Minutes	12.07	16.25
Guder Town (15KM)	18,678.74	9/5/2040 1:59	1Hr and 59 Minutes	8.94	15.77
At Homacho (25KM)	18,513.46	9/5/2040 2:15	2Hr and 15 Minutes	7.54	26.88*

*Depression Area

Table 5. 11: Maximum flow, time to peak flow, and flood depth for some Fato River stations below Fato Dam (Xu & Zhang)

Methods	Peak Flow m3/sec	Peak Time	Time to peak	Peak Velocity (m/sec)	Maximum Depth (m)
At Bridge (9KM)	11,797.39	9/5/2040 3:22	3Hr and 22 Minutes	10.27	12.33
Guder Town (15KM)	11,657.33	9/5/2040 3:43	3Hr and 43 Minutes	7.53	12.35
At Homacho (25KM)	11,638.824	9/5/2040 4:02	4Hr and 02 Minutes	3.42	23.72*

*Depression Area

5.2.4. Piping Breach Outflow Verification

As a test of reasonableness, the computed peak output from the model is compared to the regression equations and the envelope curves of historical failures. Based on data from 14 previously failed and documented dam breakdowns, the following curve was created. We can't declare with certainty if it fits or not based on the data from only 14 dams, but we can assess how logical the results of the Fato dam breach analysis are.

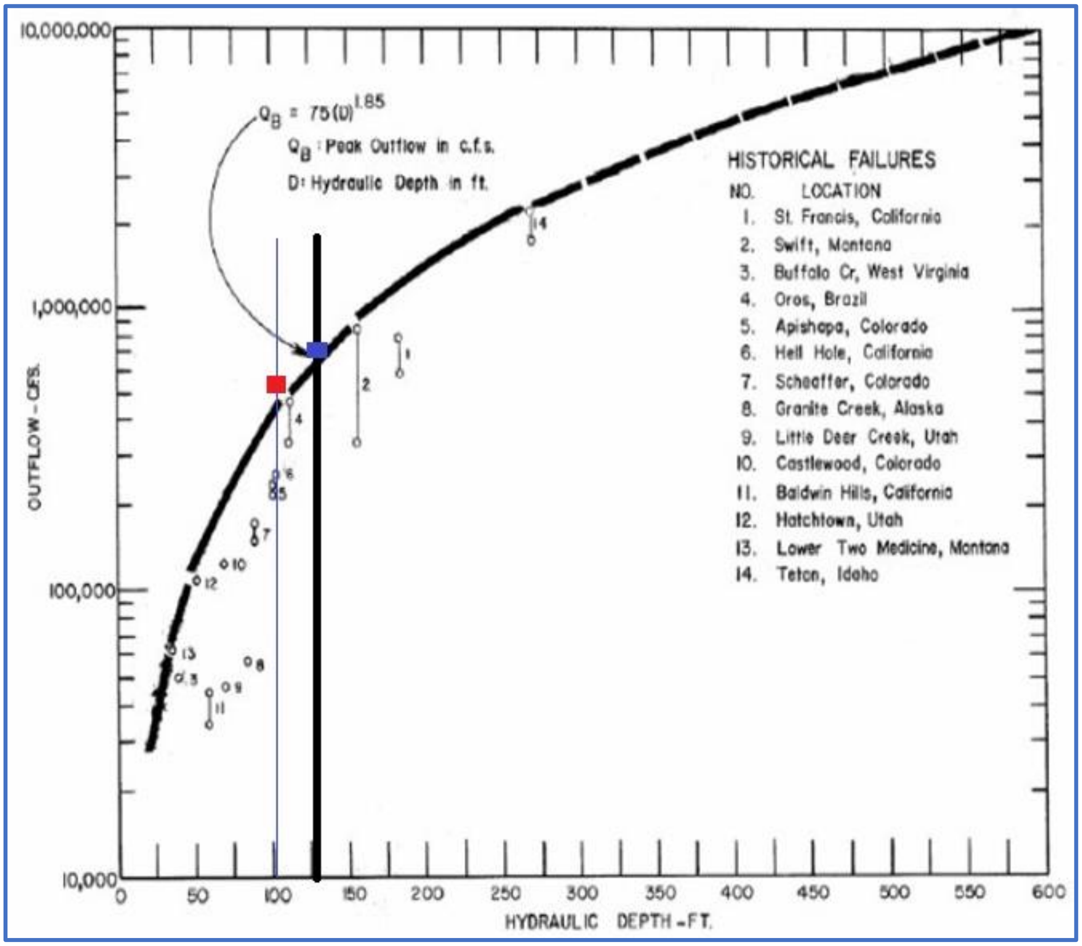


Figure 5. 2: Verification of Fato dam piping outflows using historic outflow rates envelope.

Sign	Empirical Equation	Hydraulic Depth (feet)	Peak Flow (cfs)
🟦	Froelich (2008)	129.16	714,234.89
🟥	Xu & Zhang	103.51	416,703.17

Table 5. 12: HEC-RAS results and peak discharge equations summary

Methods	Peak flow from HEC RAS (m3/s)	Peak flow from the equation (m3/s)	Difference (m3/s)	Difference (%)
Frohelich (2008)	20,224.88	17,088.07	3,136.81	15.51
Xu and Zhang	11,799.72	11,34 5.49	454.23	3.84

5.3. Flood Inundation Mapping

To define the flooded areas in depth and area, the downstream flood inundation extent and depth are demarcated and plotted using the breach outflow hydrograph produced by the HEC-RAS model (V 6.3.1).

Using the output of five approaches, water-surface profiles for the 2.3*PMF and sunny-day dam-breach scenarios were created. The next graphic illustrates the maximum flood-inundation extent and depth for each important location for the 2.3*PMF and sunny-day dam-breach scenarios.

The river has narrow gorge with depth therefore most of the flood wave passes through this section of the river but when it reaches around Guder town the flood wave will began to impact most of the Guder town and next to Guder town there is Homecho Ammunition Engineering Complex (HAEC) residential place will also be impacted by the breach flood wave.

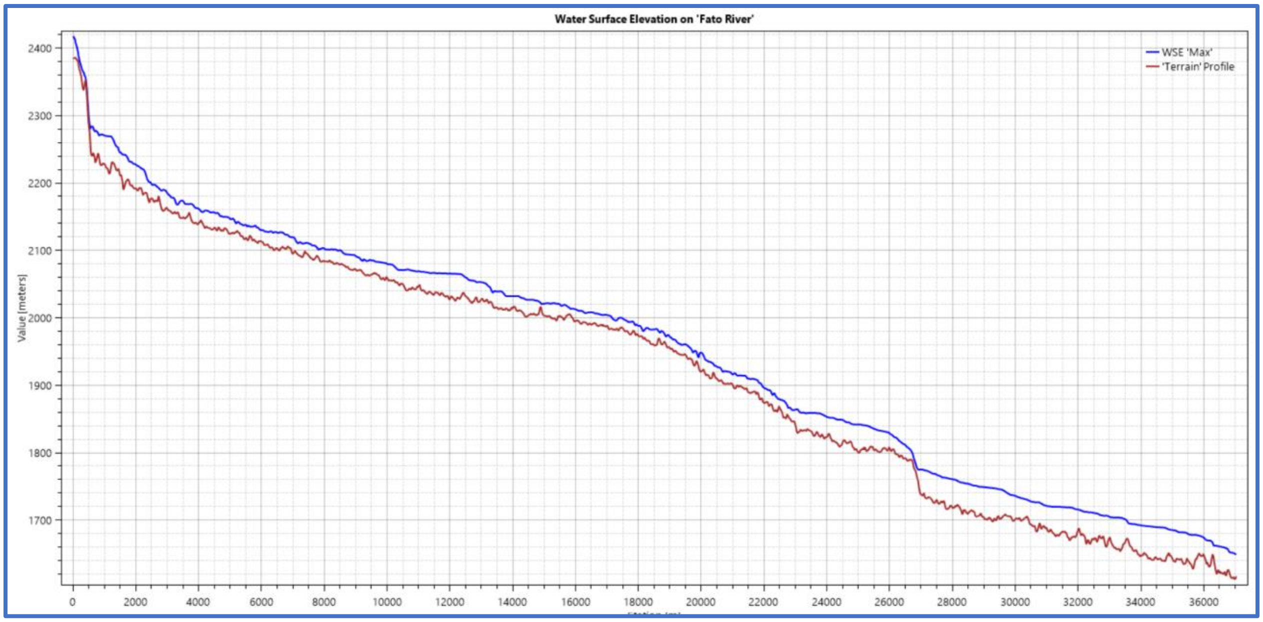


Figure 5. 3: Water Maximum Water Surface Longitudinal Profile (Von Thun & Gillete)

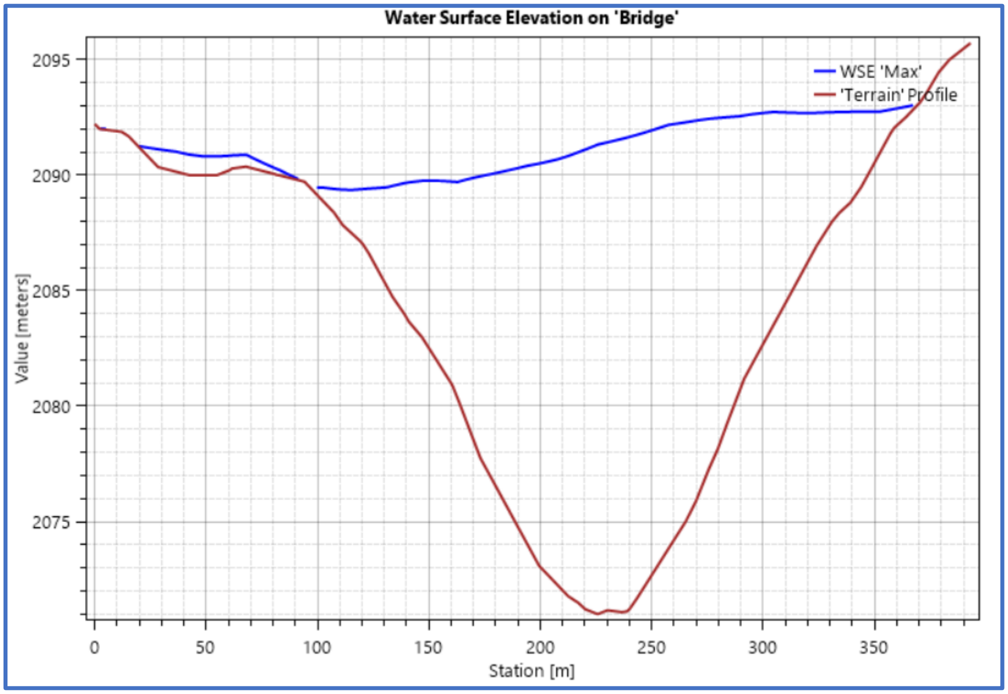


Figure 5. 4: Maximum Water Surface Extent Boundary at Chainage 9 + 00 (Bridge)

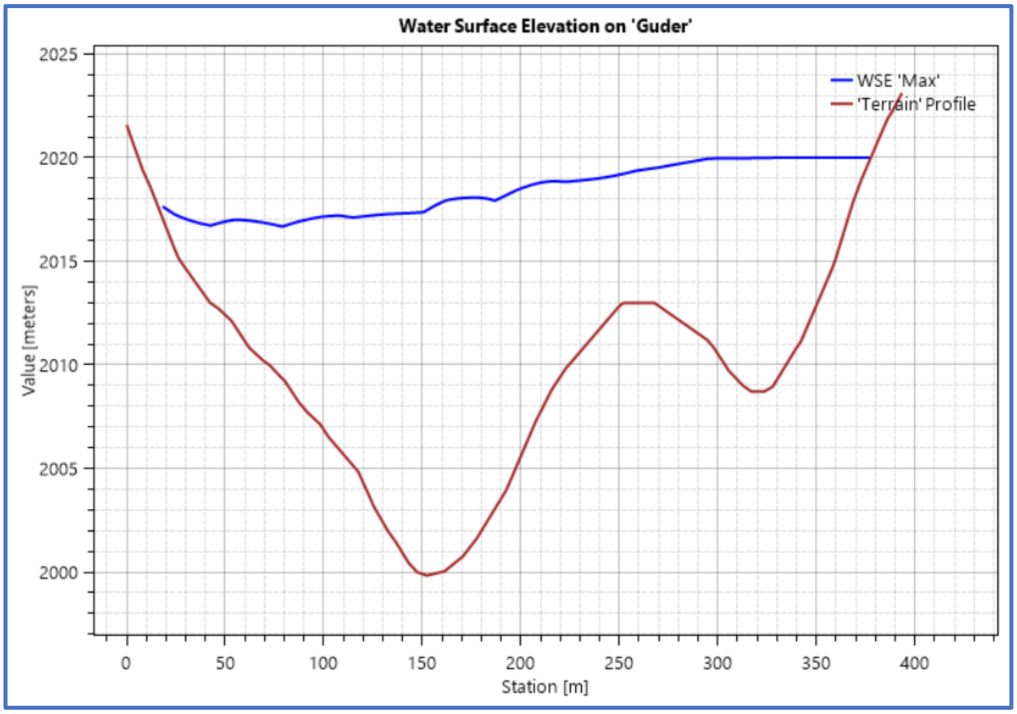


Figure 5. 5: Maximum Water Surface Extent Boundary at Chainage 15 + 00 (Guder Town Entry)

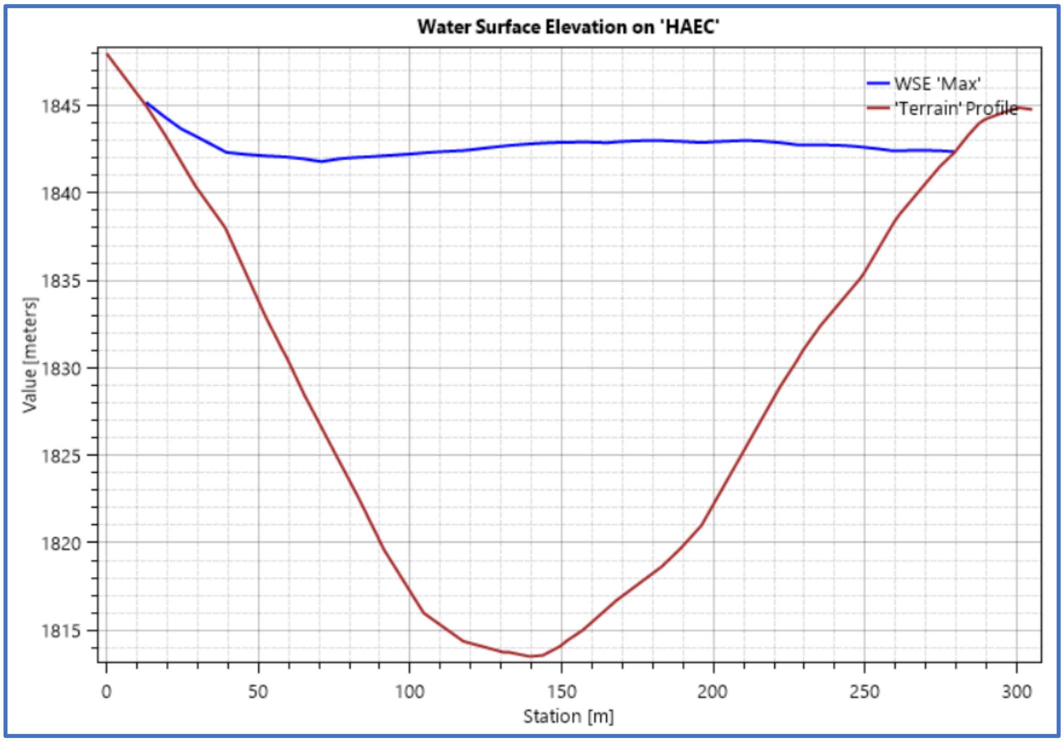


Figure 5. 6: Maximum Water Surface Extent Boundary at Chainage 25 + 00 (HAEC Entry)

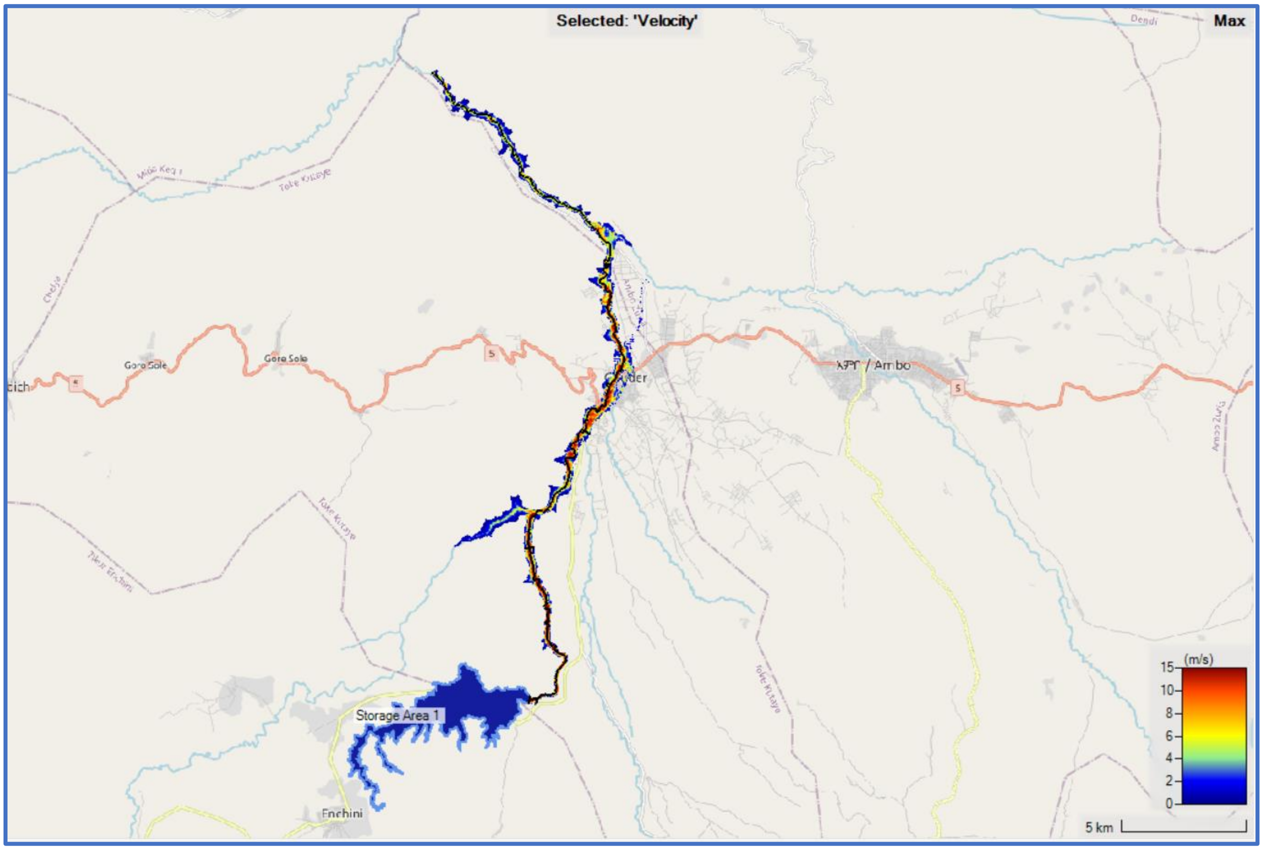


Figure 5. 7: Flood Inundation of Fato Dam Vicinity (Von Thun & Gillete)

5.4. Loss of Life

5.4.1. Without Emergency Action Plan EAP

Since the dam have no emergency action plan prepared for the emergency situations of failure, the researcher follows the less than 15minute warning time loss of life equation developed by Wayne J. Graham, P.E. The emergency action was not prepared for the dam since the project TOR have no emergency action plan preparation for the Fato dam design project. Therefore, the warning time will be less than 15 minutes.

Taking the warning time is less than 15 minutes: **(Brown and Graham)**

- Loss of Life = 0.5*(PAR)
- Loss of Life = 0.5*(9425)
- Loss of Life = 4,713 people

Taking the warning time 0 hrs. **(DeKay and McClelland)**

- $\text{Deaths} = \dfrac{\text{PAR}}{1+13.277\ (\text{PAR}^{0.44}) * e^{(2.982(\text{WT})-3.79)}}$
- $\text{Deaths} = \dfrac{9{,}425}{1+13.277\ ((9{,}425)^{0}) * e^{(2.982(0)-3.79)}}$

- Deaths = $\frac{9,425}{1.3}$
- Deaths = 7,250 people

5.4.2. With Emergency Action Plan (EAP) Prepared

Although the Fato had no EAP prepared for it, assuming EAP prepared for it the author computed the possible loss of life due to failure of this dam. DSO-99-06 manual was used for estimating the possible loss of life due to dam failure. Assumed 20% or more of flooded residences are either destroyed or heavily damaged because the water depth and velocity at Guder town is deep and fast therefore this flood will induce large damage to flood area. The warning time was considered to be 1.35Hrs from Von thun & Gillete results, because flood wave reaches Guder town early than all the rest.

When warning period falls between 15 and 90 minutes: **(Brown and Graham)**

Loss of Life = $\mathrm{PAR}^{(0.6)}$

Loss of Life = $9425^{(0.6)}$

Loss of Life = 243 people

When the warning period exceeds 90 minutes: **(Brown and Graham)**

Loss of Life = .0002*(PAR)

Loss of Life = .0002*(9425)

Loss of Life = 2 people

Taking the warning time 1.35hrs: **(DeKay and McClelland)**

$$\text{Deaths} = \frac{\mathrm{PAR}}{1+13.277\,(\mathrm{PAR}^{0.44})*e^{(2.982(\mathrm{WT})-3.79)}}$$

$$\text{Deaths} = \frac{9,425}{1+13.277\,((9,425)^{0.44})*e^{(2.982(1.35)-3.79)}}$$

$$\text{Deaths} = \frac{9,425}{943.14}$$

Deaths = 10

5.5. Discussion

The breach parameters estimated using five regression equations (Macdonald and Langridge-Monopolis (1984), Froehlich (1995), Froehlich (2008), Von Thun and Gillette (1990) and Xu & Zhang) are different for the two modes of failure discussed in this paper. The results from all the four methods except for MacDonald breach geometry fall within the dam body while it falls outside therefore custom breach geometry was assumed for this specific dam site and named Custom (2023) and compared with other results. Of all the conditions von thun creates large peak discharge, therefore this one is used for the PAR estimation. The people at risk were computed from flood inundation area resulting from Von Thun & Gillete breach parameter estimation. The possible loss of life was computed using two literatures equations one by Brown and Graham the other was DeKay and McClelland. The results for loss of value from these literatures indicates that provision of EAP with implementation can dramatically reduce the possible loss of life to occur from dam breach.

Based on the above-mentioned depth of flow, velocity distribution of Fato dam breach flood area large area of Guder town fall under the flood inundation area. This makes Guder town be the mostly affected town incase this dam fails. Next to Guder HAEC if the next infrastructure prone to danger. Based on this flood map then digitized the houses that fall within the inundation for people at risk (PAR) computation. The people at Risk were found to be 9426 Peoples. Then the loss of life using DSO-99-06 manual. And it is found to be around 4713 people are in loss of life from the failure of this dam using Brown and Graham equation without EAP. And it is found to be around 7,250 people are in loss of life from the failure of this dam using DeKay and McClelland equation without EAP. This makes dam class be very-high hazard dam based on ENTRO classification guideline. But assuming Emergency action plan be prepared the possible loss is 2 people using Brown and Graham equation and 2 people using DeKay and McClelland equation.

The equation used for the estimation of the loss of life is directly adopted from the Unites States of America manual DSO-99-06 as the ENTRO dam safety guideline recommends. But directly adopting the American guideline might not work for our country where the emergency response level of our people is far slow than the Americans response to emergency. This results in underestimating the possible loss of life incase dams fail in our region dams (ENTRO dams). Therefore, for the possible loss of life for our region dams (ENTRO dams) our region customized loss of life equation needs to be developed that takes into consideration the slow response to emergency.

The breach section used for the breach analysis were only the sections that fit to the topography of the Fato dam. This is due to the construction of Embankment dam in narrow canopy where concrete dam was supposed to be an option. This is due to loos foundation as shown in the dam longitudinal geophysics map with low resistivity. Therefore, going for physical or numerical method breach modeling Fato dam will be of a great indication of the outflow flood hydrography.

6. CONCLUSION AND RECOMMENDATION

6.1. Conclusion

The classification of a dam's risk potential reflects the consequences of its failure or breach based on the most likely worst-case scenario that is thought to be reasonable. A dam should therefore be designed so that it can withstand failure even in the worst scenarios that may reasonably be predicted to occur, given that it is estimated that a dam's failure would have substantial or catastrophic effects. This suggests adopting rigorous design standards as well as ensuring the highest standards of inspection and maintenance. Therefore, through the analysis of seven alternative scenarios, probable flow conditions, their potential consequences, and the hazards class in the case of a likely breach of the Fato dam in Ethiopia's Oromia Regional State were determined. The US Army Corps of Engineers' Hydrologic Engineering Center River Analysis System (HEC-RAS V6.3.1) software was used to conduct the analysis.

This study modeled Fato dam for 7 breach situations of these 5 overtopping and 2 piping mode of failures. The results from these models indicate that the breach flood wave arrives the Guder town nearly within 1.35hrs and 1.5hrs to reach Homecho Ammunition Engineering Complex (HAEC) with large flood depth and velocity ranging 12.05m/s(Guder) 8.95m/s(HAEC) and 17.84m(Guder), 29.30m(HAEC) respectively. The above-mentioned flood reach time and corresponding flood velocity is taken from Von Thun and Gillete equations.

Based on the above-mentioned depth of flow, velocity distribution of Fato dam breach flood area large area of Guder town fall under the flood inundation area. This makes Guder town be the mostly affected town incase this dam fails. Next to Guder HAEC if the next infrastructure prone to danger. Based on this flood map then digitized the houses that fall within the inundation for people at risk (PAR) computation. The people at Risk were found to be 9426 Peoples. Then proceeded with the computation of loss of life using DSO-99-06 manual. And it's found to be around 4713 people will lose life from the failure of this dam. This makes dam class be very high hazard dam based on ENTRO classification guideline.

If there were EAP preprepared for this dam before construction the loss of life will be dramatically reduced. Therefore, from this study the author concluded that if there were regulatory framework for any dam design although its fictious we might not lose this all people for this a good example is Baldwin Hills disaster people at risk was 16,500 with warning time of only 1.5Hrs the recorded loss of life was 5. The existence of EAP and its implementation can reduce the loss of life induced from dam breach significantly. When we come to Fato dam breach the flood wave reaches Guder town in 1.5hrs nearly same as Baldwin.

Finally, the author concluded that as long EAP is not prepared for Fato dam the PAR are in great danger in case the dam fails. The nonexistence of the EAP will cost our country a great loss in life, economically and environmentally incase the dam fails, but if EAP be prepared this loss will be reduced dramatically with small investment in dam safety plan for Fato Dam.

6.2. Recommendation

Fato dam is currently under construction. The construction started in 2020 G.C and now the construction process has stopped due to security problems. The good thing about this dam is that the status of the construction of the dam is only cutoff trench is excavated and stopped. Therefore, in the next steps of the construction process, specially while filling the dam, the constructor and the consultant supervising the dam need to be sharp in following the fill materials properties (shell, filter, and core). So that the dam be safe against piping failure.

In our country we do not have an official dam design manual and regulatory framework for following any dam designs and its construction permit process. The owners of the dam give the design work to the consultant if the contract is DBD and for the contractor if the contract is turnkey and there is no regulating institution to check these designs for safety and give construction permit. A good example for this is Ethiopian building code of standards (EBCS), any building design procedure passes through or following this standard and experts from government side in different levels review these designs and approve it for construction. Following the footstep of EBCS it’s must that we prepare dam design manual country level and then the corresponding regulatory framework for its implementation. Here finally the recent EBCS is adopted from European Standard, therefore we could also adopt USBR dam design manual or ICOLD manuals officially like EBCS do. That way we will have a figure of how many dams constructed in Ethiopia with their location and corresponding salient features. Based on these frameworks we can assess the hazard class of our dams and schedule on the maintenance of these dams based on their severity and risk they induce incase these dams fail.

For Fato dam the emergency action plan was not prepared since the EAP preparation was not stated in the TOR of the design work. Therefore, before filling the dam EAP of the Fato dam needs to be done so that life of people at risk within the inundation area be saved.

For the next researcher or EAP preparing expert over Fato dam breach modelling, I recommend modeling the dam using physical method or numerical method of breach section estimation since the empirical equations results go beyond the dam site canopy opening.

As a reservoir routing simulation result shown that the Fato dam will not be over topped with PMF inflow when the reservoir was full and only spillway was functioning as outlet of incoming flood from reservoir with 0.9m still freeboard to the dam crest to over topping it. Bearing this in mind since the dam foundation have fault there might be a significant settlement in this area. Therefore, during construction, it would be advisable if the consultant goes with inspection in case of foundation material change in the fault zone, so that there be no significant settlement which might makes the dam prone to overtopping.

Finally, even if this dam was designed to pass the inflow design flood safely, the dam will overtop in the case of erratic conditions like 2.3*PMF. Therefore, breach model also can be analyzed by other physically based methods to ensure the safety of the dam for the future.

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