ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY

School of Graduate Studies
School of Civil and Environmental Engineering

"Dam Breach Modelling and Flood mapping, a Case Study of Ribb Dam"

Thesis Submitted to the School of Graduate Studies of the Addis Ababa Institute of Technology in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering, Major Hydraulic Engineering

By: Fasika Worku

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Addis Ababa Institute of Technology

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Fasika Worku Tegbaru

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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 dissertation entitled "Dam Breach Analysis and Flood mapping (A Case Study of Ribb Dam)" in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (Major Hydraulic Engineering).

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Declaration

I, Fasika Worku Tegbaru, declare that this dissertation is my own original work and that it has not been presented and will not be presented to any other University for similar or any other degree award.

Signature

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DEDICATION

Dedicated to my late father WORKU TEGBARU!

I wish I could cherish this moment with you, Gashe!

ABSTRACT

The spontaneous dam breach phenomenon and the resultant flooding that happened in the world history leads to the requirement of establishing a dam safety plans and hazard management strategies. In this regard, the dam breach pre-event analysis will be the prerequisite work.

This thesis addressed a pre-event analysis of a dam breach scenario for Ribb dam located in Amhara regional state, Ethiopia. Deterministic and probabilistic approach for the modelling of the dam breach is used. Overtopping and piping failure modes are assessed and the resulting flood inundation is mapped. A 1.5 PMF inflow hydrograph and a base inflow hydrograph are used as upstream boundary condition, while Lake Tana makes the downstream boundary condition. Fourteen 2D simulations are carried out and of which ten are for different breach parameters, two are for uncertainty analysis on breach parameters, and two are sensitivity analysis on Manning's roughness coefficient 'n'.

HEC-RAS Ver 5.0.7 hydraulic model is employed, and McBreach is used for probabilistic dam breach modelling. In this study, five deterministic non-physical empirical methods and probabilistic breach modelling are assessed and compared.

The five deterministic non-physical empirical methods have resulted in peak flow values between 67,570m^3/s and 113,153m^3/s for overtopping and between 22,269m^3/s and 40,926m^3/s for piping modes of failure respectively. For both modes of failure, MacDonald and langridge-Monopolis and Frohelich (1995a) produced the lowest and highest peak discharge respectively. The 1% and 90% exceedance probability peak discharge for overtopping failure mode is 104,379m^3/s and 77,521m^3/s respectively. The Manning roughness coefficient 'n' sensitivity analysis showed a 0.11 to 39.9 percentage increase in flood depth and 2.20 to 20.67 percentage decrease in velocity for an increase of the Manning roughness coefficient by 30%. In addition, the Manning roughness coefficient 'n' sensitivity analysis showed a 0.00 to 15.32 percentage decrease in flood depth and 10.81 to 28.09 percentage increase in velocity for a decrease of the Manning roughness coefficient by 30%.

The study highlighted the dam breach and its corresponding flooding could be potentially catastrophic and high priority should be given to monitoring and surveillance of the dam.

Key words: HEC-RAS, RAS Mapper, DEM, McBreach, Ribb Dam, Dam Breach Analysis, Scenario, 2D.

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ACRONYMS

CSA	Central Statistical Agency
CL	Centerline
DEM	Digital Elevation Model
D/s	Downstream
EAP	Emergency Action Plan
EMP	Effective emergency actions plan
EPI	Exceedance probability inundation
FEMA	Federal emergency management agency
FERC	Federal Energy Regulatory Commission
Fig	Figure
ft.	Feet
GIS	Geographic Information system
ha	Hectare
HEC	Hydrologic Engineering Center
HEC-RAS	Hydrologic Engineering Center's for River Analysis System
hr	Hour
ICOLD	International committee on large dams
IDF	Inflow design flood
Km	Kilometer
Km2	Kilometer square
LiDAR	Light Detection and Ranging
m	Meter
m3	Cubic meters
m3/s	Cubic meter per second

m.a.s.l.	Meter above mean sea level
Max	Maximum
Min	Minimum
MMC	Million Metric Cubes
MWL	Maximum Water Level
NWL	Normal Water Level
NPL	Normal Pool Level
NWS	National Weather Service
PDFs	Probability density function
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
t	time
SA	Storage Area
SDF	Spillway design flood
SMP	Surveillance and Monitoring Plan
USACE	US Army Corps of Engineers
UTM	Universal Transverse Mercator
WWDSE	Water Works Design and Supervision Enterprise
UNOCHA	United Nations Office for the Coordination of Humanitarian Affairs
USBR	United states of America Bureau of Reclamation
1D	One dimensional
2D	Two dimensional

1 INTRODUCTION

1.1 BACKGROUND

Dams are classified in hydraulic structures as it store, control and divert the water stored in a reservoir for variety uses. A guideline for Dam breach analysis (Colorado, 2010) defines a Dam as a man-made barrier, together with appurtenant structures, constructed mostly above the natural surface of the ground for the purpose of impounding water. Dams provide different benefits to communities in terms of irrigation, hydropower generation, navigation, flood control etc. However, it may cause a risk to downstream communities and properties without the proper design, operation and maintenance of the dam. In general terms, Dams are categorized based on the construction material as concrete dams or earthen dams and on storage volume and dam height as large and small dams.

Construction of dams has been a long-established practice with the oldest known dam, the Sadd el-Kafara near Cairo, Egypt, being built between 2950 and 2750 B.C. (Smith, 1971). According to Griffen (1974), most early dams were constructions built by residents with little or no expertise in engineering and nothing much was done in the safety of dams as the dam area was less highly populated.

Whereas in Ethiopia, according to Dr. Kamal Eldin Bashar, A hundreds of years ago, there were customary small irrigation systems, in particular for water irrigation and water supplies in the eastern, central, northern western parts of the nation. Structures for irrigation diversion were built of wood, stones and grass with earth, which were frequently swept away during high streams and rebuilt every year. According to Dr. Kamal Eldin Bashar, after major droughts hit the country in 1974/74 and 1984/85, modern small-scale irrigation with micro dams were given great weight.

Analysis of dam breaching and the resulting floods are essential to identifying and reducing potential for loss of life and damage in the downstream floodplain. The understanding of the term ‘failure’ varies from person to person. To a lay person, failure of a flood embankment will evoke images of large holes with water gushing through. To a flood defense manager, failure may simply mean failure to keep water from reaching the landward side and causing flood damage, rather than destruction of the embankment itself. Actually, ‘Failure’ is defined in the FLOOD site project Language of Risk Report T32-07-01 (Gouldby and Samuels, 2009) as “Inability to achieve a defined performance threshold (response given loading).Catastrophic failure describes the situation where the consequences are immediate and severe, whereas prognostic failure describes the situation where the consequences only grow to a significant level when additional loading has been applied and / or time has elapsed."

In the context of breach modelling, embankment failure is considered to be the situation where erosion or structural failure of the embankment allows the passage of flood water over or through the embankment in an increasingly uncontrolled manner.

The different stages of breach development are of greater or lesser interest to different end user groups, depending upon their role within the community. For example, an asset manager might be interested in the initial breaching process and indicators of this, so as to be able to avoid development of a catastrophic breach. An emergency planner would be interested in what might happen during a catastrophic event so as to be able to plan for all eventualities. A dam safety engineer would be interested in all stages of the process so as to be able to advise upon the safety of a structure during an incident and to advise on the likelihood and timing of any potential catastrophic failure. (Morris, 2009)

Thus, the scope of the study required to sufficiently define the negative impacts of dam breach will differ with many factors such as the extent of existing and potential downstream development, reservoir (depth and storage volume), type of dam, and purpose of the study. The modelling and flood mapping should advance until adequate information is produced to reach a comprehensive conclusion including the breach consequences. This shall also include sensitivity analyses to address the uncertainty in the breaching process.

A Dam Safety Surveillance and Monitoring Plan (SMP) should be prepared and effected for all dams with a safety risk and a Potential Consequences Classification, as per the Dam Safety Training Module (December, 2014). The potential benefits achieved from the analysis significantly used in the phases of dam life cycle such as in planning, study, design, construction, operation and maintenance or rehabilitation, in which thresholds of dam failure, probabilities of failure, and consequences of failure are all of prime importance. The preparing and implementation of such projects in Ethiopian dams will ensures the well-being of downstream communities and further growth of various facilities on the downstream areas that are critical to the country's economic development.

The study addressed the dam failure analysis of earth-rockfill dam. As a case study, Ribb Earth-rockfill Dam located in Southern Gonder, Ethiopia has been considered. Ribb Irrigation

Project dam is been completed on 6th of September, 2017. Because of large volume of people and assets present downstream of the dam, the dam is categorized as a high-hazard structure. Because of this classification, the dam is designed with PMF inflow hydrograph (WWDSE, 2010).

Breach parameters for the dam were estimated using several regression equations appropriate to the type and size of the involved dam and reservoir. The unsteady 2D HEC-RAS hydraulic model of Ribb dam is developed The dam break analysis of Ribb dam extends approximately 75.3 kilometer from the dam to downstream outlet (Tana Lake).Two modes of dam failure scenarios were evaluated for this dam; i.e. hydrologic (PMF inflows) as well as a "Sunny Day" (non-hydrologic) failure. A "Sunny Day" (non-hydrologic) failure assumes failure of the dam by means other than a storm induced flood event, such as a failure triggered by earthquake or internal dam erosion (piping). In addition, Through Monte Carlo simulation by McBreach, the hydraulic modeler produced exceedance probability peak discharge and their respective sampled breach parameters that can be used to produce probability inundation (EPI) map. The exceedance probability is conducted on 0.2%, 1%, 5%, 10%, 50%, 90%, 95%, and 99% probability.

In fact, major problems of flooding and waterlogging are frequent in the area and causing problem for the agriculture practice. Since, Ribb River typically exhibits lower river reach characteristics as it flows dissecting Ribb Irrigation and Drainage Scheme Command area. Meandering, minor bed shrinking, excessive sedimentation and sub division in to smaller channels and continuous reduction of bed slope and thus flow concentration. The reduction of the river slope result in a less and less in channel flow concentration and dominance of flood plain flow. Under pristine condition the local farmers harness the phenomenon by farming paddy rice mostly supplied by the flood plain water coming from the surrounding upland areas in Addis Zemen and Debretabor, highlands reaching 3000m above MSL peaks (BRL Ingenerie, 2016).

1.2 BREACH MODELING

All dams have higher stresses in extreme events which raise the potential threat of failure, irrespective of design and construction (Ahmad Asnaashari, 2014). Thus, the ultimate flow from the expected dam breach is generally determined through a dam breaching study. The result, dam failure breach outflow hydrograph, is routed through the entire river system in order to calculate the arrival timing, depth and flow of the flood. Flooding areas are mapped for emergency action plans, emergency response, hazard mitigation planning and dam breach consequence assessment.

To simulate the Ribb dam-breach flood, a two-dimensional HEC-RAS 5.0.7 model is used and determines the breach outflow and water-surface profiles in downstream. HEC-RAS 5.0.7 presents the possibility of representing and exchange of geo referenced cross-section and alignment graphics. For using the 2D modelling capability, it is absolutely essential to have this terrain model from the LiDAR Data and a gridded terrain model is developed and imported for further processing by RAS MAPPER. The resulting terrain layer is a basis for developing 2D mesh cells terrain property. Moreover, Mc-Breach software is used to produces exceedance probability peak discharge and their respective sampled breach parameters that can be used to produce probability inundation (EPI) map.

1.3 STATEMNET OF THE PROBLEM

Historically, The Ribb dam downstream region is particularly vulnerable to frequent floods that caused a significant loss of life and economic damages. According to the UNOCHA report in 2006, the Fogera-Dera flooding affected 43,127 and displaced 8728 people. Even last year, the overflows of Ribb River on 29th of July, 2020 submerged the main road from Wereta to Gonder and from Debre Tabor to Gonder in Fogera woreda and more than 1,000 people were affected in Shina kebele of Lebo kemikem and Fogera Woredas (UNOCHA, 2020). In fact, all these damages happen due to heavy seasonal rainfall and it will be worse if the assumed breach happens. The dam breach will cause more devastating effects on lives, livelihoods, and property including the planned Infrastructure due to the huge volume of water to be released.

However, like many other dams in Ethiopia, a dam break analysis for Ribb dam is not studied yet, even though it poses a high hazard potential due to flat terrain and higher population density per area along the water way in the downstream region. In addition, a breach phenomenon is mostly unexpected and abrupt in which a plan to evacuate the downstream residents after failure by itself may not be effective in saving lives and avoiding damage to property. Therefore, a dam breach study is required in the dam project cycle and downstream development plan to assess the full scale of the consequence and to prepare potential management plans. In fact, the breach analysis should be done for any dam in every certain period of time based on the downstream development and also to be input to the establishment of effective emergency action plans, design of early warning systems, and characterization of threats to lives and property. Hence, the project aims to close a research gap by producing flood inundation maps that reliably estimate dam breach flood depths and arrival times at key locations including the extent and severity of the flood.

1.4 OBJECTIVES OF THE STUDY

1.4.1 GENERAL OBJECTIVE

The general objective of the study is to model the Ribb dam breach phenomenon and analyze the potential flood hazards in the downstream low lying flat areas.

1.4.2 SPECIFIC OBJECTIVE

The specific objectives are:

- To check if the PMF inflow hydrograph could be evacuated safely
- To assess and estimate the dam breach parameters in a pre-defined breach development.
- To run the breach model in different scenarios.
- To generate the breach outflow hydrographs for each scenarios.
- To perform the two-dimensional unsteady flood routing.
- To produce the various types of maps such as inundation extent and severity, velocity and depth with time.
- To produce maximum velocity and depth profile line at selected key location or route.
- To produce an exceedance probability inundation mapping

1.5 SIGNIFICANCE OF THE STUDY

In general, dam breach inundation modelling and conduct mapping can be used for multiple purpose including dam safety, hazard mitigation, consequence evaluation and emergency management including developing EAPs. In specific, conducting pre event analysis of Ribb dam breach modelling and flood mapping have great significances on the listed below:

- Evaluating and establishing the hazard potential classification of the dam
- Estimating the potential for loss of life.
- Producing breach inundation zone mapping for flood warning and evacuation planning.
- Producing breach inundation zone mapping for risk communication so that the public is aware of the risks of living downstream of dams.

- Developing breach exceedance inundation zone mapping for downstream infrastructure development.
- Planning protection dikes to be provided at both downstream banks with their top level considerably higher than the propagating flood level, especially on the stretches along residence areas.
- A reference for further detailed researches on the study area.

1.6 SCOPE OF THE STUDY

This study shall cover estimation of dam breach parameters and breach outflow hydrograph for different scenarios and then conduct inundation modelling and mapping. The exceedance probability breach modelling and inundation mapping also will be conducted for overtopping and piping modes of failure. Eventually possible scenarios will be reviewed, conclusions will be drawn, and subsequent recommendations will be forwarded.

2 LITERATURE REVIEW

2.1 BACKGROUND

Dams provide many beneficial uses and value for our society; however floods caused by dam breaches have been responsible for some of the world's worst tragedies in the previous two centuries. As per guidelines for dam breach analysis (Colorado, 2010), “breach” is defined as the opening formed in the dam body that cause the dam to fail and this phenomenon causes the concentrated water behind the dam to propagate towards downstream regions. In general, dam failure can result in death, property destruction, cultural and historic loss, environmental losses as well as social impacts (Graham, 1999).All these negative consequences led to an attention and importance as well as opportunity to achieve significant improvements in technology in analyzing dam breach process. The potential benefits achieved from the analysis significantly used in the phases of dam life cycle such as in planning, study, design, construction, operation and maintenance or rehabilitation, in which thresholds of dam failure, probabilities of failure, and consequences of failure are all of prime importance. In this regard, analyzing the failure of dam can be viewed as a two-step process. First the actual breach of the dam must be analyzed, and second the outflow from the breach must be routed through the downstream valley to determine the resulting flood at the population center or highly important property location. The breach analysis is also crucial to development of effective emergency actions plan (EMP), design of early warning system, and characterization of threats to lives and property for the planned or existing dams.

2.2 DAM BREACH HISTORY

A dam breach occurs when a dam fails partially or completely, resulting in an uncontrolled discharge of water (Fread, 1993). Dam failures have occurred in many countries with serious consequences of loss of life and property. Herewith listed below, some of the major dam failures in the world which resulted in high loss of lives. (Xu et al 2008), (Noorani A.G, April 1984,) and (David McCullough et al, 2019)

Table 2-1: List of major dam failures

List of major dam failures in the world	Year of failure	State, Country	Fatalities	Details

Banqiao and Shimantan Dams	1975	Zhumadian, China	171,000	Typhoon Nina poured extreme rainfall in China, exceeding the dam's planned design capacity. 11 million individuals were displaced from their homes. Between 1986 and 1993, the dam was reconstructed.
Machchu-2 Dam	1979	Morbi, India	5000	Flooding above the capacity of the spillway due to heavy rain. The old estimate was 1,800–25,000, but a 2011 book by Sandesara and Wooten update the data to 5,000–10,000.
South Fork Dam	1989	Pennsylvania, United states of America	2,208	Owners' inadequate care, which resulted in a crests height being lowered by a meter or more; followed by unusually heavy rainfall.

Dams with a size category of intermediate or large have caused the majority of dam failure casualties as per the United States dam failure data. In this regard, a dam size classification policy was developed by the U.S. Army Corps of Engineers (USACE) to provide a beneficial approach to explain the size of the failed dams and to size the classification established by either storage or height, whichever provides the larger size classification.

In Ethiopia, there were traditional small scale irrigation schemes practices in the earlier times, predominantly in the eastern, central, north western parts of the country (Dr. Kamal Eldin Bashar M. M., 2003).According to the Nile Basin Capacity Building Network for River Engineering study, the diversion structures were built of wood, stones, and grass and swept off during floods and rebuilt each year. However, a dam construction in Ethiopia started in the late thirties and the first modern dam is Aba Samuel Dam, constructed on the Akaki River, tributary of the Awash River and was commissioned in 1939(Dam safety Guidelines,2010).

Regarding dam breach, there is no event recorded in Ethiopia so far. Nevertheless, most dams' safety status are not well identified at present due to major problems to conduct dam safety assessment such as lack of available technical information (design and construction documents), Insufficient or no monitoring and surveillance of the dams and lack of capacity in interpretation of data recorded from monitoring instruments. However, some of the major dams are suffering easily visible piping and structural stability problems which may lead to breach and considerable loss. Kesem and Tendaho Dams are among some of the dams in which a 1m3/s flow along its left side abutment and a total leakage of nearly 0.5 m3/s respectively observed (Tendaho dam leakage and associated problems, 2011).Moreover, this safety problem will worsen also with ageing of dams in the future.

Furthermore, According to ICOLD 2012 (International committee on large dams), the several reasons contributing to earth dam failure can be categorized in to the following three classes;

Hydraulic failure (40%)

- Overtopping,
- Erosion of upstream face,
- Erosion of downstream face and
- Erosion of toe of the dam

Seepage failure (30%)

- Piping (internal erosion)
- Uplift
- Sloughing and subsequent failure of the dam

Structural failure (25%)

- Embankment slope failures
- Foundation slide

Others (5%)

2.3 PURPOSE OF DAM BREACH ANALYSIS AND ITS INUNDATION MAPPING

According to FEMA (FEMA, 2013), Dam breach inundation studies are utilized for various uses, including:

- Assessing and determining the dam hazard potential classification
- Estimating the potential for loss of life
- Evaluating dam safety risk and prioritizing dams within a dam safety portfolio
- Selecting the appropriate SDF or IDF for dam and spillway design
- Developing EAPs and exercise planning associated with dam safety permitting
- Developing breach inundation zone mapping for flood warning systems and flood evacuation planning
- Developing breach inundation zone mapping for dam breach consequence studies and for flood mitigation planning
- Developing dam breach inundation zone mapping for risk communication to inform the public of the risk living downstream of dams.

2.4 HAZARD POTENTIAL CLASSIFICATION

Hazard Potential Classification is a system that classifies dams as per the extent of negative increment of consequences such as probable loss of human life and the impacts on economic, environmental, and lifeline interests happened due to a failure or mis-operation (unscheduled release) of dam on the downstream areas or at locations remote from the dam. The hazard potential classification of a dam, beside its size (height and capacity) classification, is used to define all phases of dam life cycle and dam breach modeling.

Hence, a key objective of any classification system is not to analyze the possibility of a failure, but to evaluate the possible negative effects and create appropriate design criteria and operation system of the Dam and its appurtenant structures. In other words, design criteria and safety guidelines will become more conservative as the potential for loss of life and/or property damage from failure increases. However, judgment and common sense must ultimately be a part of any decision on classification since all possibilities cannot be defined.

In general, the following lists the three hazard potential classifications of dams (FEMA, 2013)

Table 2-2: Hazard potential classification for dams

Hazard Potential	Loss of human life	Economic, environmental, lifeline losses

Low	None expected	Low and generally limited to owner
Significant	None expected	Yes
High	Probable, one or more expected	Yes

2.5 DAM BREACH ANALYSIS STUDY APPROACHES

The two most common techniques to dam breach studies in United States of America State governments and Federal agencies are event-based and risk-based approaches. For dam breach analysis, the event-based method has typically been the most common. For dam breach analysis and downstream inundation mapping, the event-based technique is a deterministic method based on specific precipitation and non-precipitation phenomena. Based on a dam's hazard potential classification, both a non-hydrologic "fair weather failure," also known as a "sunny day failure," and a specific hydrologic failure event, such as the Probable Maximum Flood (PMF), are commonly established for the event-based approach (FEMA, 2013).

In the past two decades, risk-based approaches to dam breach analysis have become more acceptable for dam safety and dam design purposes. A risk-based approach is commonly used for dam design purposes to establish the SDF or IDF for a dam. For a risk-based approach, the downstream consequences for a range of hydrologic dam failure events are evaluated (FEMA, 2007).

2.5.1 EVENT-BASED APPROACH

An event-based approach is a deterministic method for evaluating dam failures and downstream inundation mapping that involves the usage of a specific or sequence of specified precipitation and non-precipitation events. Extreme rainfall and runoff events, for example, can result in natural floods of a higher magnitude. The maximum flood for which a dam must be designed or evaluated is frequently determined by its existing hazard potential or size classification (FEMA, 2013).

An event-based dam safety analysis typically evaluates various hydrologic and non-hydrologic (fair weather) events. An extreme flood event, ranging from the 50-year event for low-hazard dams to the PMF for high-hazard dams, is chosen for hydrologic failure events based on the potential for loss of life or major economic and environmental losses due to a dam failure. The extreme hydrologic failure event is usually chosen based on the dam's hazard potential classification. The PMF is the flood that can be predicted in the drainage basin under the most severe combination of key meteorological and hydrologic conditions that are reasonably possible in the drainage basin under study. The Probable Maximum Precipitation (PMP) is a calculation of the maximum potential precipitation depth for a given catchment area for specified time period. (Stedinger et al., 1996).

The greatest advantage to using an event-based approach is that it is a direct approach, is less complicated to perform and regulate, and produces more conservative breach inundation zone mapping when compared to a risk-based approach. High-hazard potential dams are typically evaluated using a full PMF, and significant- or low-hazard potential dams are evaluated on a percentage of a PMF or some more frequent storm event (FEMA, 2013).

a. Fair Weather (Non-Hydrologic) Failure

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

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

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

During fair weather conditions, a breach at the reservoir's usual pool elevation is utilized to estimate the volume and accompanying breach discharge that would flow from a failure event. This type of event is modeled as piping/internal erosion failure in an embankment dam, whereas a monolith collapse in a concrete dam is modeled as sliding, foundation instabilities, or a seismic event.

- **Invert of Auxiliary Spillway (lowest uncontrolled spillway)**

To simulate a breach during the disoperation of the principal outflow works, a breach of the dam with the reservoir water level set at the auxiliary spillway (also known as an emergency spillway) is usual practice. Dam failure usually assumes to begin at the same time as the reservoir level reaches normal pool.

- **Top of Dam / Maximum High Pool**

The reservoir level is set at the dam crest to represent the maximum amount of volume that can be retained. This criterion can be used to assess the most conservative non-hydrologic event. Dams without appropriate spillways or pump storage facilities, where the water level is kept at the top of the dam during non-hydrologic events, are unusual conditions subjected to this conservative assumption. When the water level is at the top of the dam, a breach event can be modeled as a piping/internal erosion failure or an overtopping failure with the water level just above the top of the dam invert.

Various United States of America Federal agency publications deliver guidance for determining the initial water surface elevation of a pool during a fair weather failure event. Each of these elevations is used to describe various failure types and also the pool's potential volume at the time of failure. For fair weather failure, the normal pool elevation is considered as the default volume.

b. Hydrologic Failure

Hydrologic breaches that occur with extreme precipitation and runoff are termed “rainy day” or hydrologic failures. Hydrologic failures that cause dam breach events are generally analyzed based on the IDF established by the dam’s hazard potential and hazard size classification, typically a PMF for high-hazard potential dams. For significant-hazard potential dams, the breach event may include a breach of the PMF and IDF that could range from the 1-percent-annual-chance flood event (often called the 100-year flood) to a percentage of the PMF (FEMA, 2013).

However, it is still difficult to get certain deterministic values due to the inherent uncertainty in breach process. In this regard, Risk informed decision making based on a probabilistic approach to dam breach modeling can be taken in to account as additional supplement to the conservative deterministic approach. Historically, this uncertainty was assessed by performing a sensitivity analysis on a variety of hypothetical dam breach parameter sets to determine the full range of possible dam breach outcomes and how sensitive those outcomes were to the inputs. Each of the undetermined dam breach parameters should be assigned a probability density function (PDF) in a probabilistic dam breach parameter evaluation. A simple uniform distribution (for example, the piping initiation elevation, where all elevations are equally likely) or a more typical normal (Gaussian) distribution could be used. Means and variances can be approximated to define the PDFs by examining at the breach parameter predictive equations that apply to the subject dam, considering potential failure modes and site conditions, and applying reasonable engineering judgment.

2.5.2 RISK-BASED (CONSEQUENCES-BASED) APPROACH

To account for the downstream impacts of a possible dam failure, a risk-based approach to dam design and dam safety evaluations has been established. The consequences assessment is focused on the potential loss of life or increase in economic damages caused by a projected dam failure, rather than the likelihood of failure.

2.5.3 TIERED DAM BREACH ANALYSIS

To develop an initial dam hazard potential classification as well as obtain dam breach inundation zone mapping for EAPs, a tiered method of dam breach analyses can be applied. The three-tiered dam breach analysis structure isn't appropriate for dam design.(FEMA, 2013).

For this study, Tier 3 –Advanced is selected as per the table shown below

Table 2-3: Tiered Method for Dam Breach Inundation Mapping (FEMA, 2013)

Tier Level	Applicable to	Breach Parameter Prediction	Peak Breach Discharge Prediction	Downstream Routing of Breach Hydrograph
Tier 1 –Basic level Screening and Simple Analysis	• Low-hazard potential / small size • First level screening for significant- or high-hazard dams	Empirical Equations	Simplified Models (SMPDBK, GeoDam-BREACH, or Technical Release [TR]-66) or HECHMS	GeoDam-BREACH, SMPDBK, DSAT,1D HECRAS Steady State, or HEC-HMS Hydrologic Routing

Tier 2 – Intermediate	• Significant hazard potential / intermediate size • High-hazard dams with limited population at risk	Empirical Equations	HEC-HMS or HEC-RAS Unsteady Model	HEC-RAS (Steady or Unsteady Modeling) 1-D or 2-D models
Tier 3 –Advanced	• High-hazard potential / large size dams with sufficient population at risk to justify advanced analyses	Empirical Equations, NWS BREACH, or WinDAM	HEC-RAS Unsteady Model	HEC-RAS Unsteady Model or 2-D models

Tier 1 and 2 analyses are most appropriate for low-hazard potential / small sized and significant-hazard potential / intermediate-sized dams with a limited number of structures.

More detailed surveying or modeling may be warranted for Tier 3 analyses for high-hazard potential / large-sized dams, those with a large population in the evacuation area, or those with significant downstream hydraulic complexities.

2.6 DAM BREACH ANALYSIS

In dam breach analysis, there are two basic perspectives that could be viewed by the modeler (Wahl, 2010):

1. Reservoir outflow hydrograph prediction and
2. Routing of that hydrograph through the downstream areas in order to assess the dam failure negative impact.

Furthermore, the following four key tasks to conduct a dam break analysis study are suggested in the dam breach analysis guidelines (Colorado, 2013):

1. Breach parameter estimation (breach size ,shape and time of failure)
2. Breach peak discharge and breach hydrograph estimation
3. Breach flood routing, and
4. Estimation of the hydraulic conditions at critical locations or Inundation Mapping.

2.6.1 BREACH PARAMETER

A significant factor in computing a dam breach hydrograph in respect of said dam is the estimation of dam breach modeling parameters related to the geometry and timing of the breach formation (for example, width, depth, shape and failure time) (FEMA 2013). Even commonly used dam breach analysis programs require estimates of certain geometric and temporal characteristics of the dam breach as inputs for the model. Dam type and hydraulic loading are major factors that affect the rate and size of breach formation. Furthermore, design, construction quality, material used and the state of material can all also have a significant effect on breach progression.

2.6.1.1 BREACH PARAMETER DEFINITION

The term breach parameter is commonly used to describe the factors that define the time required for breach initiation and development (breach depth, breach width, and side slope angles), as well as the parameters that define the physical description of the breach (breach depth, breach width, and side slope angles).

Those are as described as below.

- **Breach formation time (tf)**: the duration of time between the first breaching of the upstream face of the dam (breach initiation) and when the breach has reached its full geometry.

  In depth, a dam breach is normally occurring in two different phases beginning with the breach initiation then by the breach formation.

  Breach initiation: Flow through the dam is small during the breach start period, thus the dam is not regarded to have failed. If the flow is regulated during this time, a dam breach may be avoided.

  Breach formation: When the flow through the dam has increased and moved from the upstream face to the downstream face of the dam, breach formation (as indicated above) occurs. It will be uncontrollable and will cause dam failure (FEMA, 2013).

- **Breach depth (hb):** the breach depth is the vertical extent of the breach measured from a specific elevation to the invert of the dam breach.
- **Breach width (Bave)**: is the average of the final breach width, typically measured at the vertical center of the breach.
- **Bottom width (Wb)**: is the breach width developed when the breach formation reaches its maximum.
- **Breach side slope (zH: 1V):** the breach side slope is a measure of the angle of the breach sides represented as z horizontal to 1 vertical.

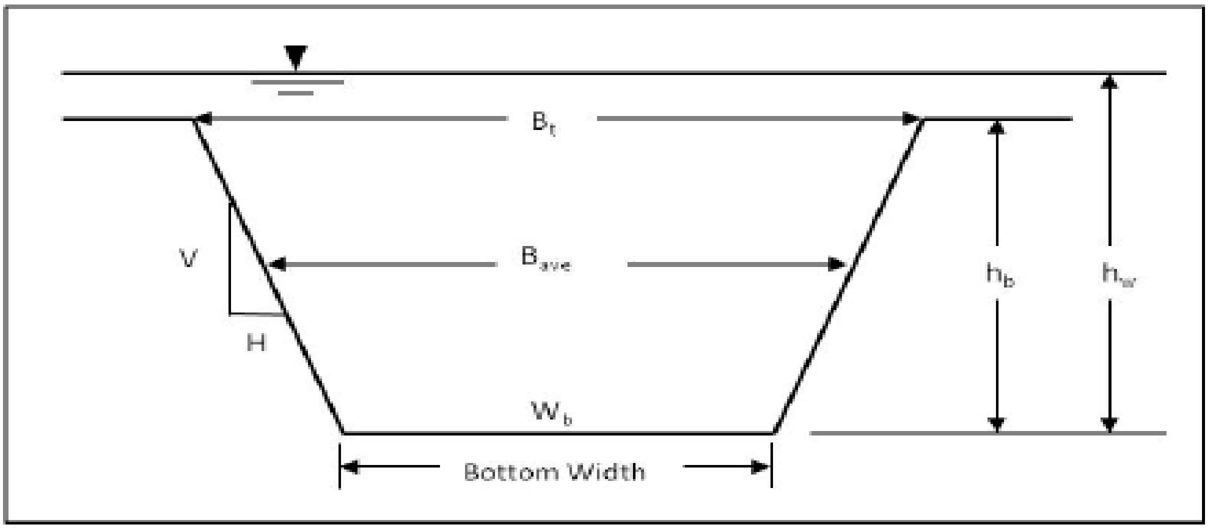


Figure 2-1: Dimensions of Dam Breach

2.6.1.2 BREACH MECHANISMS FOR EMBANKMENT DAMS

In general, the breach forming mechanisms can be classified as (Colorado, 2010):

1. Breaches formed by the sudden removal of all or a portion of the impounding structure as a result of some over-stressing of the structure (for concrete or other rigid dams), and
2. Breaches formed by erosion of embankment material (overtopping and piping failure).

The breach mechanism and dam type affects the hypothetical dam breach geometric shape in which usually approximated as rectangle, triangle, trapezoidal or parabola. For embankment dams, a trapezoidal shape is common and for concrete arch dams usually the shape of the breach will be the same as the shape of the dam (Wahl, 1998).

While breaking in embankment dams can occur for different reasons as specified in section 2.2, breaches in embankment dams are most often categorized and simulated as overtopping or piping failure modes. In general, failure mode is a start-and-growth process of a breach in which starting of the overtopping failure from the top of the dam and increasing to full extent and piping failure can start at any height or place and expand to the maximum extent.

Three distinct categories of material typically used to build flood embankments or dams are non-cohesive fill, cohesive fill and rock fill and their breach behavior is shown in the figure below (Morris, 2009).The breaching process such as head cutting, progressive surface erosion and interlocking can be seen simply as a result of the use of different construction materials. Despite the fact that the main modes of failure have been identified as piping or overtopping, the actual failure mechanics are not well and fully understood for either earthen or concrete dams yet (Y.Xiong, 2011).

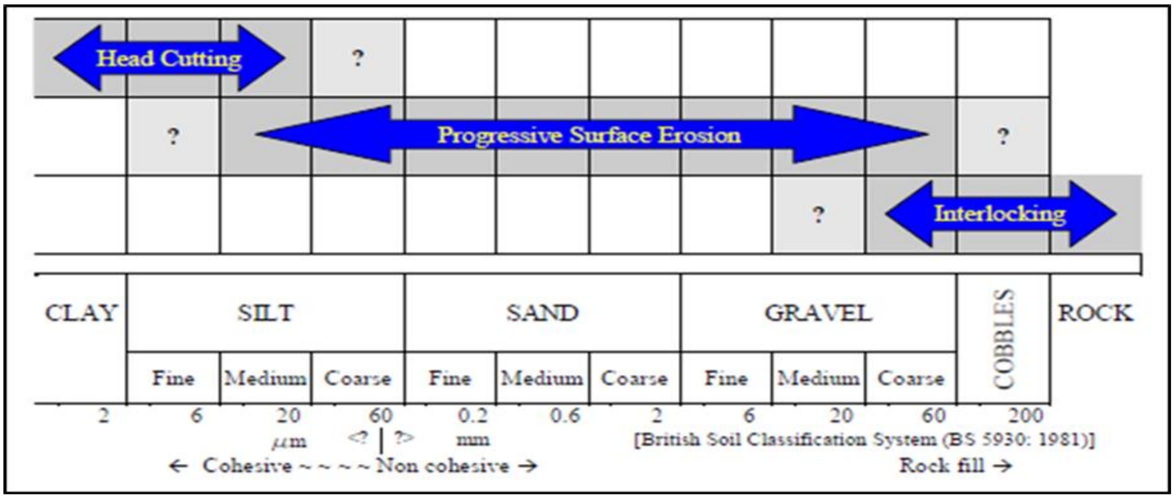


Figure 2-2: Broad division of breach behaviour by material type (Morris, 2009)

2.6.1.2.1 OVERTOPPING FAILURES

As specified above, overtopping failures will occur quite in a different way conditional on the dam composition. Possibly the most straightforward overtopping failure to talk over is a failure of a cohesive soil embankment. A study by Ralston (1987) shows that a small head cut naturally forms on the downstream face of a cohesive soil embankment and develop upstream as shown in Figure 2.3(FEMA, 2013).

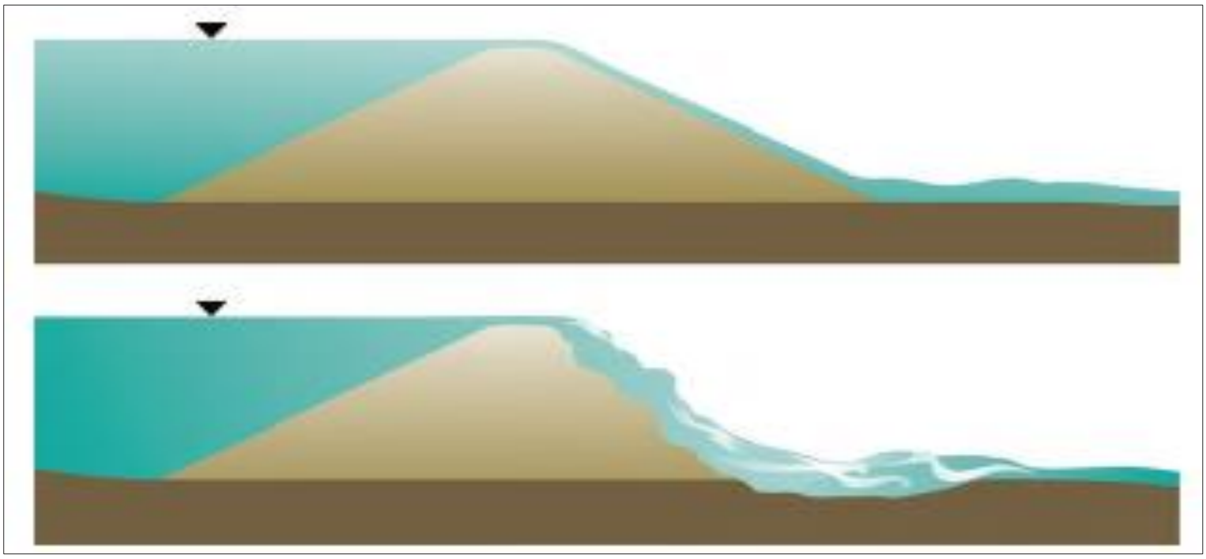


Figure 2-3: Cohesive soil embankment dam erosion

According to FEMA, the dam break is considered to commence when erosion takes place over dam crest width. Following the initiation of the break at the top of the dam crest, it extends to its ultimate range as shown in Figure 2.4(FEMA, 2013).

An assumed trapezoidal breach progression is illustrated in Figure 2.4.

Figure 2-4: Assumed Trapezoidal Breach Progression

The breach progression may be considered and modeled as either a linear progression or a sine wave progression:

- Linear progression: rate of erosion remains the same for the duration of erosion development)
- Sine wave progression: breach grows very slowly at the beginning and end of development and rapidly in between

In a study by the State of Colorado Department of Natural Resources, no significant difference were found between linear and sine wave progression models when comparing one overtopping case study in HEC-Hydrologic Modeling System (HMS) and HEC-RAS (2010).

The progression using the more conservative results should be assessed and used (FEMA, 2013).

In fact, the dam break may stop increasing if the reservoir has emptied and no further water is there to erode the dam, or the dam has entirely eroded or reached the foundation of the dam (Gee, 2009).

In this regard, in overtopping failure or flood induced failure, the reservoir's water level is at or near the dam crest (i.e. fairly above the maximum water level) in which the build-up of the water cannot be held later. This usually happens during an extreme flood event which is greater than what the dam can safely pass. Conversely, this type of failure can be anticipated from advance extreme meteorological forecasts which could apparently produce flooding. Thus, advance warning may be issued and required emergency actions can be planned ahead.

2.6.1.2.2 PIPING / INTERNAL EROSION FAILURES

Piping happens when a concentrated seepage advanced within an embankment dam. The seepage erodes the dam over time, creating huge voids in the soil. The piping generally starts near the downstream toe of the dam and moves to the upstream face. As the voids grow, erosion increases alarmingly. As erosion worsens, the flow of water through the embankment will become muddy. When erosion approaches the reservoir, the pipe hole can become larger, causing the dam crest to collapse. Figure 2.5 illustrate a schematic of a fully formed piping hole.

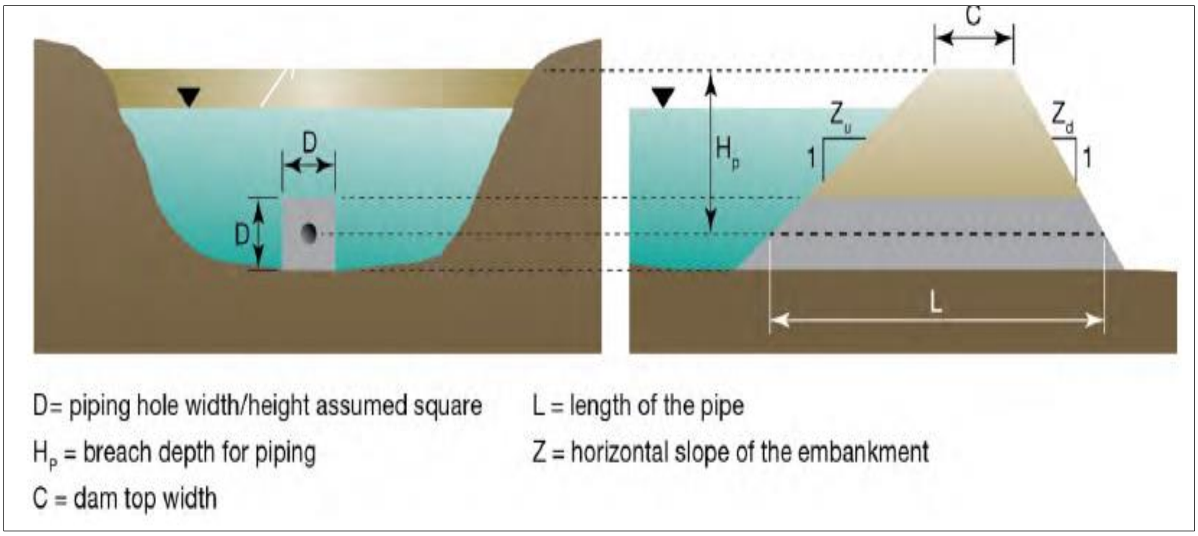


Figure 2-5: Schematic Diagram of piping hole

In this regard, dam breach analysis due to piping / Internal Erosion failures with reservoir at normal pool level (NPL) is termed as “sunny day failure”. This is a sudden dam failure that occurs during normal operations with water level at NPL and the water released during failure bringing a significant change in flows. The causes of failure can be foundation failure, earthquake, or related events. Basically, sunny day failure is usually sudden and leaves no or little time as an indication of the onset of failure.

2.6.1.3 FAILURE LOCATION

The break failure location is determined by various factors such as type and shape of dam, failure type, mode and driving force for the failures. If no physical reason exists to assume the embankment will fail at a particular location based on the assessment of any historical knowledge of seepage and foundation problem or any likely location for each failure type, the break analysis shall be modeled as initiating at the maximum part commonly situated at the downstream main channel centerline.

2.6.1.4 BREACH DEVELOPMENT TIME

The time of failure most commonly depends on the size of the dam, types of materials used for construction, and the structural strength of the embankment in which generally for earth dams ranges from six minutes to three hours. However, Concrete dam failures are much more rapid, and have a time range of six to eighteen minutes.

There are various viable alternatives to find the breach initiation time. The commencement of a breach associated with a hydrologic event can be deemed to begin when the water level of the reservoir surpassed a specific elevation, or if for a certain period it tops a certain elevation. However, an initiation time will be established regardless of the pool elevation for fair weather dam break analysis (Gee, 2010).Moreover, the breach development time will be counted until the breach is completely formed and major erosion ends.

2.6.1.5 BREACH WEIR AND PIPING FLOW COEFFICIENT

The breach weir and piping flow coefficient is a function of many parameters such as dam height, impounded storage volume of water (including the inflow hydrographs), materials that the dam is constructed of depth, depth and duration of overtopping, protective cover of the embankment and other parameters.

Overtopping failures are modelled as the weir flow only whereas piping failures are typically simulated in two stages, before and after the dam crest slump. Before the dam crest falls, water flow through the piping hole is modeled as orifice flow, and after the dam crest falls, it is modeled as weir flow. For small cohesive dams, the reservoir might fully empty before the dam crest slump (State of Colorado Department of Natural Resources, 2010). However for the large dams, the breach width and height grow until the elevation of the top of the breach is the same as the crest elevation, at which point the breach is identical to an overtopping failure (FEMA, 2013).

Guidelines for selecting breach weir and piping flow coefficient are provided in table 2.4

Table 2-4: Dam breach weir and piping coefficients

Dam Type	Overflow/Weir Coefficients (US unit)	Piping/Pressure flow coefficients
Earthen clay or Clay core	2.6-3.3	0.5-0.6
Earthen sand and gravel	2.6-3.0	0.5-0.6
Concrete Arch	3.1-3.3	0.5-0.6
Concrete Gravity	2.6-3.0	0.5-0.6

2.7 DAM BREACH PARAMETERS ESTIMATION

Accurate prediction of breach parameters is decisive to make a close predication of the outflow hydrographs and downstream flood inundation maps and consequently a lead to effective development of emergencies action plans, design of early warning system, and characterization of threats to lives and property. In particular, the prediction is very critical when the population centers are located close to dams.

The breaching process comprises a complex interaction between hydraulic, geotechnical and structural processes; hence a combination of knowledge and skills cutting across these different technical disciplines is therefore required if reliable models are to be developed for breach prediction. In this aspect, different models have been developed to estimate the breach parameters and magnitude of outflow hydrograph. However, so far it is evident that reliable breach prediction remains quite crude and the existing ones still comprehends the highest uncertainty in all sides of dam breach analysis. Thus, a cautious assessment and knowledge of the associated break parameters is required.

In this regard, models can be classified as non-physical, semi-physical and physical models, as explained below (Morris, 2009).

I. Non-Physically Based, Empirical Models.

Non-physically based or empirical models are methods usually based upon data collected from a series of documented breach events (Morris, 2009). These include guidelines in which ranges of breach parameters are specified and also a regression equations in which developed equations as best fit to available data of historically failed dams are presented.

- **Federal Agency Guidelines**

Many Federal Agency Guidelines have circulated guidelines with possible range of value for breach width, side slopes, and development time as shown in table 2.5 based on the data collected on historic dam failures. Thus, whenever there is estimating of the dam breach parameters, this shall be utilized as a minimum and maximum limit. (TD-39, 2014). Nevertheless, especially dams with large volumes of impoundment water, and lengthy dam crest length, will last to erode for long extended periods as considerable volume of water flows through breach. Hence, it might have a longer breach widths and times than what is shown in table

Table 2-5: Breach characteristics range guidelines

Dam Type	Average breach width (B_{avg})	Horizontal component of breach side slope (H)(H:V)	Failure time(Hours)	Agency
Earthen/Rockfill	(0.5 to 3.0)*HD (1.0 to 5.0)*HD (2.0 to 5.0)*HD (0.5 to 5.0)*HD	0 to1.0 0 to1.0 0 to1.0(slightly larger) 0 to1.0	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	Multiply monoliths Usually≤0.5L Usually≤0.5L Multiply 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 Arch	Entire Dam	Valley wall slope	≤0.1	USACE 1980

	Entire Dam (0.8*L) to L (0.8*L) to L	0 to valley walls 0 to valley walls 0 to valley walls	≤0.1 ≤0.1 ≤0.1	FERC NWS USACE 2007
Slug/Refuse	(0.8*L) to L (0.8*L) to L	1.0 to 2.0	0.1 to 0.3 ≤0.1	FERC NWS

HD=Height of dam L=Length of dam crest

- **Regression equations**

Two varieties of methods are categorized in regression equations (FEMA, 2013) in which dam breach geometry and time summarized below, and its peak discharge as stated in section 2.8.1.

Parametric Regression Equations – These equations, established from case study data, are utilized to estimate the ultimate break geometry and failure time. The breach will then be modelled to continue as a time-dependent linear process for the calculation of breach outflows hydrograph using principles of hydraulics.

The advantage of these equations is their simplicity and it possesses some control over the dam breach parameters, and thus allowing adjustment for site-specific circumstances. However, this simplicity is also one of their main weaknesses, in that there can be considerable uncertainty within the predictions. Moreover, users often have little knowledge of the data set upon which the Equations were developed and hence any constraints for application and the suitability for application to site specific cases are unknown.

Several scholars have established regression equations for estimating the dam break dimensions (Width, side slopes, volume of eroded etc.) as well as failure time. The regressions equations mentioned below have been utilized for a numerous dam safety studies and literatures except the Xu and Zhang equations in which presented due to their broad range of historical datasets. Moreover, these regression equations have been applied in a number of dam breach studies and shown out to give a reasonable range of values for earthen, zoned earthen, earthen with core wall(i.e. clay),and rockfill dams(TD-39,2014).

- **Mac Donald and langridge-Monopolis**

Mac Donald and langridge-Monopolis used 42 data sets (predominantly earth fill dams, earth fill dams with clay core, rockfill dams) to establish a relationship called the "Breach formation factor". The breach formation factor is calculated by multiplying the volume of water coming out of the dam and the height of water above the dam and the breach development factor was then linked to the volume of material eroded from the dam's embankment. The ranges of data utilized by Mac Donald and Langridge-Monopolis for regression analysis were as follows:

Height of the dams: 4.27 -92.96 meters

(With 76 %< 30meters, and 57 %< 15meters)

Volume of water at breach time: 0.0037 – 660.0 m3 x 106

(With 79 %< 25.0m3 $*10^6$, and 69 %< 15.0m3 $*10^6$)

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

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

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

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

$$t_f\,(hr) = 0.0179(Ver)^{0.364}$$

$$Wb = \frac{V_{eroded} - h_b^2\,(CZ_b + h_b\,Z_b\,Z_3/3)}{h_b(C + h_b Z3/3)}$$

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

Veroded = Volume of water eroded from the dam

Vout= Volume of water that passes through the breach, Initial estimate will be the volume of water before breaching

hw= depth of water above the bottom of the breach

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

C = Crest width of the top of the dam

Z3 = Z1+Z2

Z1 = Upstream Slope of the dam

Z2 = Downstream Slope of the dam

Zb = Side Slope of the Breach

According to MacDonald and Langridge-Monopolis, the dam break shape should be trapezoidal and have a side slope of 0.5H: 1V.

- **Froehlich(1995a)**

Froehlich used 63 earthen, zoned earthen, earthen with a core wall (i.e. clay) and rockfill data base to derive a number of equations in order to estimate the average breach width, side slopes and failure time.

The data that Froehlich used for his regression analysis had the following ranges:

Height of the dams: 3.05 -92.96 meters

(With 90 %< 30meters, and 76 %< 15meters)

Volume of water at breach time: 0.0133 – 660.0 m3 x 106

(With 87 %< 25.0m3 *10^6, and 76 %< 15.0m3 *10^6)

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

$$\mathbf{Bave = 0.1803\, K_o\, V_w^{0.32}\, h_b^{0.19}}$$

$$\mathbf{tf = 0.00254Vw^{0.53}\, hb^{-0.90}}$$

Where: Bave= average breach width (meters)

Ko= constant (1.4 for overtopping failures, 1.0 for piping)

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

hb = height of the final breach (meters)

tf = breach formation time (hours)

Froehlich states that the average side slopes should be:

1.4H: 1V for overtopping failures

0.9H: 1V otherwise (i.e. piping/seepage)

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

- **Froehlich, 2008**

In 2008 Dr. Froehlich updated his breach equations based on the addition of new data. Dr.

Froehlich used 74 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rock fill data base to derive a number of equations in order to estimate the average breach width, side slopes and failure time. The data that Froehlich used for his regression analysis had the following ranges.

Height of the dams: 3.05 -92.96 meters

(With 93 %< 30meters, and 81 %< 15meters)

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

(With 86 %< 25.0m3 *10^6, and 82 %< 15.0m3 *10^6)

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

$$\mathbf{Bave = 0.27\ K_o\ V_w^{0.32}\ h_b^{0.04}}$$

$$\mathbf{Tf = 63.2\sqrt{Vw}/ghb^2}$$

Where: Bave= average breach width (meters)

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

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

hb = height of the final breach (meters)

g = gravitational acceleration (9.80665 m/sec2)

Tf = breach formation time (seconds)

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

1H: 1V for overtopping failures

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

While not clearly stated in Froehlich's paper, the height of the dam break is computed by assuming that the break extends from the top of the dam to the natural ground elevation at the breach location.

- **Von Thun and Gillette(1990)**

Von Thun and Gillette used 57 dams from both the Froehlich (1987) paper and the Mac Donald and langridge-Monopolis (1984) paper to develop their methodology. The method proposes to use breach side slopes of 1.0H: 1V except for dams with cohesive soils, where side slopes should be on the order of 0.5H: 1V to 0.33H: 1V.The data that Von Thun and Gillette used for their regression analysis had the following ranges.

Height of the dams: 3.66 -92.96 meters

(With 90 %< 30meters, and 75 %< 15meters)

Volume of water at breach time: 0.027 – 660.0 m3 x 106

(With 89 %< 25.0m3 *10^6, and 84 %< 15.0m3 *10^6)

Von Thun and Gillette presented the following formulae for the average breach width and failure time:

Bavg (m) =2.5hw+Cb

tf (hr) =0.015*hw for highly erodible dam

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

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

Cb is function of reservoir size and given in the table below

Table 2-6: Cb Values

Size of reservoir (m3)	Cb (m)
<1.23*10^6	6.3
1.23*10^6-6.17*10^6	18.3
6.17*10^6-1.23*10^7	42.7
>1.23*10^7	54.9

- **Xu and Zhng**

The data base gathered by Dr’s Y.Xu and L.M Zhang contained 182 earth and rockfill dams from the United States of America and china, with nearly 50 percent of the dams greater than 15meters in height.

The data that Xu and Zhang used for their regression analysis had the following ranges:

Height of the dams: 3.2 - 92.96 meters

(With 78 %< 30meters, and 58 %< 15meters)

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

(With 80 %< 25.0m3 *10^6, and 67 %< 15.0m3 *10^6)

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

Bave/ hb =0.787(hd/hr) $^{0.133}$(Vw$^{1/3}$/hw) 0.652 e^{B3}

Where:

Bave = average breach width (meters)

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

hb = height of the final breach (meters)

hd = height of the dam (meters)

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

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

B3 =b3+b4+b5 coefficient that is a function of dam properties

b3 = -0.041, 0.026 and -0.226 for dams with core walls, concrete faced dams, and homogenous/zoned –fills dams, respectively

b4= 0.149 and -0.389 for overtopping and seepage/piping respectively.

b5 = 0.291,-0.14, and -0.39, for high, medium, and low dam erodibility respectively.

The Xu and Zhang study does not directly derive side slope estimations. Instead, they present an equation for estimating the breach's top width, which may then be utilized to calculate the corresponding side slopes with the average breach width.

$$\mathbf{Bt/\ hb = 1.062(hd/hr)^{0.092}\,(Vw1/3/hw)^{0.508}\,e^{B2}}$$

Where: Bt = Breach top width (meters)

B2 = b3+b4+b5 coefficient that is a function of dam properties

b3 = 0.061, 0.088 and -0.089 for dams with core walls, concrete faced dams, and homogenous/zoned –fills dams, respectively.

b4= 0.299 and -0.239 for overtopping and seepage/piping respectively.

b5 = 0.411,-0.062, and -0.285, for high, medium, and low dam erodibility

Breach side slopes can be computed with the following equation

$$\mathbf{Z = \frac{Bt - Bavg}{Hb}}$$

Furthermore, Xu and zhang data used in the equation derivation for break formation time includes more of the initial erosion period and post erosion period than what is commonly used in HEC-RAS. This equation generally results in dam break formation times that are higher than the other four equations listed above. As a result, the Xu and Zhang equation for dam break formation time should not be used in HEC-RAS. However it is shown here for the completeness of their method.

$$\mathbf{Tf/\ Tr = 0.304(hd/hr)^{0.707}(Vw^{1/3}/hw)\ 1.228\ e^{B5}}$$

Where: Tf = Breach formation time (hours)

Tr = 1hour (unit duration)

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

hd = height of the dam (meters)

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

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

B5 = coefficient that is a function of dam properties

b3 = -0.327, -0.674 and -0.189 for dams with core walls, concrete faced dams, and homogenous/zoned –fills dams, respectively.

b4= -0.579and -0.611 for overtopping and seepage/piping respectively.

b5 = -1.205,-0.564, and 0.579, for high, medium, and low dam erodibility respectively.

II. Semi-Physically Based, Analytical and Parametric Models.

The large range of uncertainty associated with the non-physically based methods on one hand and the complexity of the physically based methods lead to this model.

The following assumptions are usually made in such models (Morris, 2009):

1. A weir equation can adequately present the flow over the embankment.
2. Critical flow conditions exist on the embankment crest.
3. The breach growth process is time dependent.

These models require the user to provide an erosion rate of the breach versus with velocity or final dimensions of the breach and time of failure of embankment. The model simply predicts a growth pattern to fit these parameters and hence produces a flood hydrograph. However, these parameters cannot be easily identified and they can differ significantly from one case to another. Hence, whilst these models appear to provide a more accurate prediction of the flood hydrograph, as compared to empirical equations, they simply reflect the data provided by the user and can also include the large degree of uncertainty within this data (Morris, 2009).Furthermore, so far there is no developed guidelines for the erosion rates based on analyzing historic levee and dam breaches.

III. Physically Based Models.

For rectangular, triangular, and trapezoidal shaped breaches, equations are developed using a reservoir water-mass depletion equation, broad-crested weir hydraulics, and a breach-erosion relation.(Singh, 1998).Hence, Several numerical models have been developed to model the dam break process governed by sediment transport principles, soil mechanics including side slope stability and hydraulics. Some of these models are summarized in table 2.4.

Table 2-7: Physically Based Computer Models

Model and year	Sediment Transport	Breach morphology	Parameters	Other features
Cristofano(1955)	Empirical formula	Constant breach width	Angle of repose, others	
Harris and Wagner(1967);BRDAM(Brown and Rogers,1977)	Schoklistch formula	Parabolic breach shape	Breach dimensions, Sediment	
Lou(1981);Ponce and Tsivoglou(1981)	Meyer-Peter and Muller formula	Regime type relation	Critical Shear stress, Sediment	Tail water effects
BREACH(Fread,1988)	Meyer-Peter and Muller modified by Smart	Rectangular, Triangular, or Trapezoidal	Critical Shear, Sediment	Tail water effects, Dry slope stability
BEED(Singh and Scarlatos,1985)	Einstein Brown formula	Rectangular, or Trapezoidal	Sediment, others	Tail water effects, Saturated slope stability
FLOW SIM1 and FLOW	Linear predetermined	Rectangular, Triangular, or	Breach dimensions	

SIM2(Bodine,undated)	erosion, Schoklistch formula option	Trapezoidal	sediment	

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

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

However, the model has drawback since it has only been calibrated with a small number size.

In addition to the presented models, there is a simplest approach to dam breach characteristics estimation named as comparative analysis. This method compares a specific dam with those in a well-documented dam failure case history database. In this technique, particular dam geometry, height, slope angles, and reservoir areas and volumes are compared with a list of similarly sized dams that have failed and documented and then to apply the dam breach characteristics to the dam in concern (Colorado, 2010). However, this method is not highly reliable for large dams because of lack of accurate and comprehensive case study data on large dam (Wahl, 1998).

2.8 BREACH PEAK DISCHARGE AND BREACH HYDROGRAPH ESTIMATION

The key features of dam break outflow hydrograph in affecting the loss of life and impacting flooding areas are the magnitude of the peak discharge. Moreover, the time taken to attain the peak rate, relates to the warning time available (Wahl1998).In this aspect, the peak discharge including hydrograph computation in the dam breach analysis is priority to determine the peak flow, flood arrival time, and the depth of flood at downstream areas. The shape of the peak breach outflow hydrograph is dependent on the impounded storage volume and inflow at breach formation time, dam size, and significantly, the dam type’s erodibility and/or mode of assumed failure. In contrast to the overtopping failure of a massive, coherent, well-compacted and well-vegetated embankment, a non-ductile concrete structure will have a significantly rapid period of breach development (FERC, 2014).

2.8.1 BREACH PEAK DISCHARGE PREDICTION

Several scholars have developed predictor regression equations for the breach peak discharge. These equations predict the dam breach peak outflow discharge on the basis of case studies collected on peak discharge. When using peak discharge equations, it should be recognized that the worst case flood conditions do not always relate to peak discharge flooding since flooding is a function of the volume and rate of water release, combined with topographic features. The additional limitation of these equations from the parametric equations is that the peak discharge is predicted by the equations rather than the whole hydrograph predication. Hence, additional uncertainty will also be added in using a hydrologic-hydraulic model to compute the break hydrograph from the peak discharge. The calculated peak discharge from the model can be checked against the hydrograph developed form the parametric regression equation through models as a test for reasonableness even though the data for the regression have to be checked.

Some of common predicator regression equations for peak discharge are listed below:

Mac Donald and langridge-Monopolis recommended peak flow discharge.

$$\mathbf{Q = 3.85(Vwhw)\ 0.411}$$

Frohelich has suggested the peak flow as follows;

$$\mathbf{Q = 0.607Vw^{0.295}hw^{1.24}}$$

Xu and Zhng

The peak out flow discharge is suggested as follows,

$$Q/\sqrt{\mathbf{gVw5/3}} = \mathbf{0.175}\left(\frac{hd}{hr}\right)\mathbf{0.199}\left(\frac{Vw^{\frac{1}{3}}}{hw}\right) - \mathbf{1.274}\, e^{B4}$$

Where,

hw= depth of water above the bottom of the breach

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

hd = height of the dam (meters)

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

B4=b3+b4+b5 coefficient that are a function of dam properties b3=-0.503,-0.591, and -0.649 for dams with core walls, concrete faced dams, and homogenous /zoned –fill dams, respectively.

b4=-0.705 and -1.039 for overtopping and seepage/piping, respectively.

b5=-0.007,-0.375 and -1.362 for high, medium, and low dam erodibility, respectively.

2.8.2 BREACH HYDROGRAPH ESTIMATION

The breach outflow hydrograph is vitally essential for the assessment of flooding characteristics in the downstream areas such as depth, velocity, discharge and the area of inundation with time function. Predicating dam breach hydrographs necessitates selecting on the size, shape, and timing of the breach's formation.

Different scenarios triggering dam failure such as variable inflow, different gate operations and sunny day failure are set and the selected model is simulated to predict the outflow hydrograph. All hydrograph will be considered and compared however; the highest flood of the different scenarios is selected as a catastrophic one. For relatively small reservoirs, the dam break peak outflow usually occurs before the breach is completely developed, resulting from a consistent drop in reservoir levels during the formation of the rupture, while in large reservoirs the dam break peak outflow occurs as soon as the breach has extended to its maximum size (Colorado, 2010).

2.8.3 DAM-BREAK MAXIMUM BREACHING OUTFLOW VERIFICATION

After the hydrographs developed from different models, the maximum breaching outflow must be checked for reasonableness against the envelope curves of historic failures. When comparing computed results to the envelop curve shown below keep in mind that this envelope curve was developed from only fourteen data sets, and may not be a true upper bound of peak flow versus hydraulic depth(TD-39, 2014).

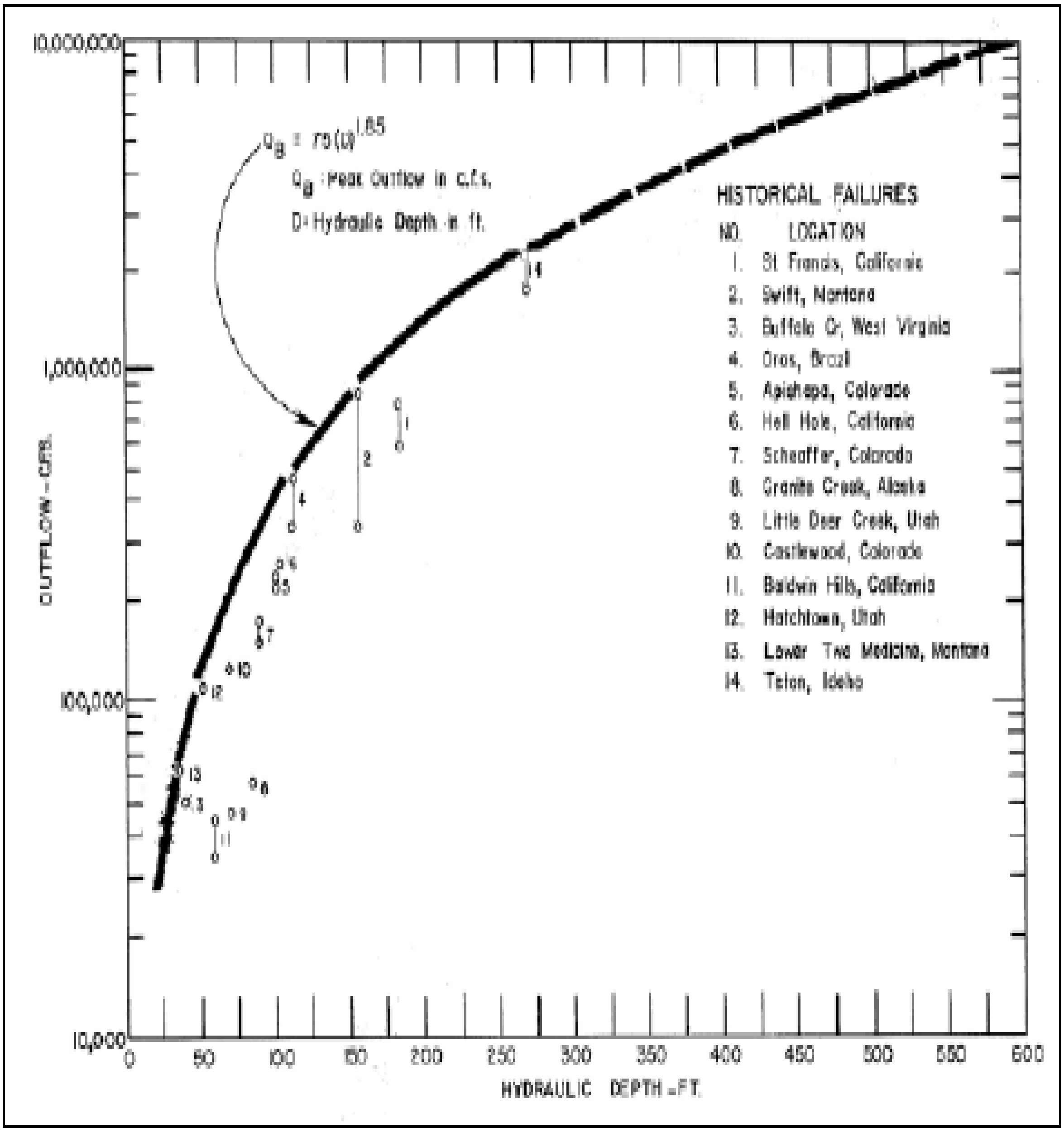


Figure 2-6: Envelope of Experienced Outflow Rates from Breached Dams (USACE, 2014)

2.9 DAM BREACH FLOOD ROUTING

The breach analysis or modeling typically involves estimation of the ultimate downstream discharge from a hypothetical breach of a dam, and then this flood wave is routed downstream throughout the river network under consideration. The analysis determines the peak flow, flood arrival time, and the depth of flood at downstream areas. Routing of the break outflow hydrograph along the flood area has a significant advantage of assessing and studying of the potential consequence and downstream risk area identification of dam failure. Routing of the breach hydrograph is performed to evaluate the attenuated or reduced peak flow at crucial areas located at downstream of the dam. Besides to computing the attenuation, estimating the flood wave arrival time, depth and velocity of flow at those important locations are also very essential parts of the study (Garcia-Martinez et al., 2009). However, once these hydrographs are routed downstream, they will tend to converge towards a common result. How close they get to each other will depend on the distance they are routed, the steepness of the stream, the roughness of the river and floodplain, and the amount of floodplain storage available for attenuating the hydrograph. If the populated areas below the dam are quite a distance away (say 20 miles or more), then the resulting hydrographs from the various dam breaches may be very similar in magnitude by the time they reach the area of interest . However, if the areas of interest are close to the dam, then the resulting breach hydrographs could produce a significant range in results. In this situation, the selection of the breach parameters is even more crucial (HEC-RAS, 2016).

The flood wave's propagates in a natural water way is a varied unsteady flow process, which is guided by conservation of mass and momentum equations. The result of these equations in a distributed mode is mentioned as distributed routing of flood waves. Flow routing by distributed system methods is known as hydraulic routing and the flow is computed as a function of space and time. If no space variability is taken into consideration and the channel reach or reservoir is regarded as a black box, the appropriate routing technique is called a bumped routing. Routing by lumped system methods is called hydrological routing. These methods analyze and compute the flow as function of time only (Chow, 1964).

Various hydrologic and hydraulic models of varying complexity have been proposed and used in recent decades to accurately predict river water levels and discharges. Dam breach analysis and flood routing are conducted using numerical models with one-dimensional, two-dimensional, or a combination of both. However if the flow channel section increases or the flow becomes out of the water way bank, accuracy of one-dimensional models lessens and two-dimensional models are preferred for conducting a flood area study. In order to select model, the modeler has to be aware of physical description of the hydraulic system, identification of the water path for the full range of events, accuracy of the data, duration of the events to be modelled, required model outputs and experience of the modeler. The following listed models are used for dam break analysis and flood inundation mapping (FEMA, 2013)

Table 2-8: Models Used in Breach Study and Inundation Mapping

Model	Application	Strength	Limitations	Governing equations
NRCS TR-66	Dams with small height and storage reconnaissance-level work value of the parameter K* is less than or equal 1.0	Fast and easy to use	Provided but no longer supported by NRCS	Peak breach process-based methods or empirical relationships Hydraulic routing A simplified form of the Att-kin routing method
NRCS SITES	Analysis of erosion in an earthen and vegetative spillway To determine discharge capacity of the principal and auxiliary spillway		Does not consider full erosion /failure of an embankment dam	
Win Dam B	Analysis of erosion in an earthen and vegetative spillway to determine the discharge capacity of the principal and auxiliary spillway .Analysis overtopping erosional breach using physical parameters	Erosion estimation based on geotechnical inputs parameters and conditions of vegetation	Does not consider breach flow through erosion/failure of the auxiliary spillway	**Routing** Does not route breach hydrograph downstream and uses level pool routing for dam breach simulation.
NWS SMPDBK	For use in emergency situation	Fast and easy to use	Nota nationality accepted, FEMA-supported by hydraulic model neglects back water effect	**Routing** Dimensions curves distinguished by the ratio of the volume in the reservoir to the average flow volume in the downstream channel governed by the Froude number travel time of the peak flow Kinematic wave(steady-state)velocity

NWS FLDWAV	Modelled for piping and overtopping breaches and also can analysis flows in mixed regimes in a system of interconnected water ways	Consider effects of downstream obstruction such as back water effects	Calibration is time consuming not adequate for all complex river condition	**Routing** One-dimensional St. Venant equations User select implicit dynamic wave ,explicit dynamic wave implicit diffusion wave or level pool solution of the St. Venant equations of unsteady flow
USACE HEC-1			No longer supported by USACE replaced by HEC-HMS	**Routing** Steady-state theory **Dam-break simulation** Level pool routing
USACE HEC-RAS	Recommended for detailed analysis and routing of the breach hydrograph	Considers effects of downstream Obstruction such as back water effect Interface with GIS to make flood inundation map Possible to use dynamic reservoir routing	More Labor demanding and time taking Instability issue may occur	The Governing equation differs based on the intended function HEC-RAS can conduct Steady flow routing Unsteady flow routing Movable boundary flow for sediment transport analysis Water quality
FEMA Geo-Dam BREACH Toolset	Simplified technique used in preliminary analysis and non-regulatory studies	easy and fast to usage	Only undertake sunny weather scenario break analysis	Uses principle laid out in the NWS SMPDBK **Routing** Undimensional curves characterized by the reservoir volume to the average flow volume in the Froude number

				The peak flow travel time Steady –state Velocity

In addition to the above summarized models by FEMA, there is also FLO-2D and MIKE 11/21 models which are also commonly used in different parts of the world.

2.10 INUNDATION MAPPING, FLOOD HAZARD, VULERNIBILITY AND RISK ASSESSEMNT

An inundation map's principal function is to represent the areas that would be flooded, as well as travel durations for wave fronts and flood peaks at important places, if a dam fails or if there are operational releases during flooding conditions. In fact, depending on both environmental conditions and the form of failure, dam failure floods can be extremely varied, and these distinctions are likely to be very important in determining suitable emergency response. In consequence, a different failure modes and environment will create quite different planning challenges.

Inundation maps have a many types of uses including EAPs, mitigation planning, emergency response, and consequence assessment. Each application has its own set of information requirements and can be applied in a variety of ways. (FEMA, 2013).Ideally the plans that are built up must obviously generate an ability to respond effectively irrespective of the environment or the exact nature of the flood. As each location below the dam for which evacuation operations are necessary exists with a plan, the environmental problem will necessarily be considered case-by-case to solve the plan problem. However, Variation by failure mode presents more serious complications (Chas keys, 1992).Thus; the planner should identify the most credible failure modes and develop arrangements sufficiently flexible to encompass the possibility of less likely kinds of failure.

Based on the inundation and hazard map, emergency action plans, emergency response, hazard mitigation planning and dam breach consequence assessment.

A Flood hazard is an agent that can cause harm or damage to humans, property, or the environment. Risk is the probability that exposure or vulnerability to a hazard will lead to a negative consequence, or more simply, a hazard poses no risk if there is no exposure and vulnerability to the level of hazard. In this regard, Risk is a combination of hazard, exposure and vulnerability (Skakun et al, 2014). In fact, the thesis paper is limited only to flood inundation mapping.

2.11 REVIEW OF SELECTED MODEL

All models, numerical or scale physical, are simplified representation of the real world (Prototype). The purpose of the model, and the expected level of accuracy required, can significantly dictate the model and modelling approach selection and the required level of accuracy of the source data.

2.11.1 DAM BREACH MODEL

As a summary of the comparison, the model HEC-RAS is selected as it uses several regression equations to predict a range of breach sizes and failure times for each failure mode and hydrologic event. Additionally, it will give a chance to compare the breach parameters to the government agency ranges provided. Furthermore, the selected HEC-RAS modeling tools are available without licensing costs and are well integrated for both flood hazard mapping and risk analysis.

As specified above, The Hydrologic Engineering Center's River Analysis System (HEC-RAS) is simulation software developed by the US Army Corps of Engineers and has been developed to manage rivers and other public works under their jurisdiction.

HEC-RAS is designed to execute one-dimensional (1D), two-dimensional (2D), or combined 1D and 2D hydraulic computations for a full network of natural or man-made channels (HEC-RAS, 2016).

The following is a list of the major hydraulic capabilities of HEC-RAS:-

(1) Steady flow water surface profile computations;
(2) Unsteady flow simulation;
(3) Movable boundary sediment transport computations; and
(4) Water quality analysis.

One of the most important aspects is that all four components share a common geometric data format and common geometric and hydraulic computation routines. The HEC-RAS system is consist of a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, and graphing and reporting facilities (HEC, 2016).

In addition to analyzing hydraulics of river flow, it is capable of modeling breaching process of inline structure such as dam, or a lateral structure such as a levee and predicting the catastrophic outflow hydrograph for both overtopping and piping .HEC-RAS utilize the listed below methods to route an inflowing flood hydrograph through a reservoir:

- One-dimensional unsteady flow routing(Full Saint Venant equations);
- Two-dimensional unsteady flow routing(Full Saint Venant equations or Diffusion wave equations);or
- With level pool routing

In general, both full unsteady flow routing is more accurate methods for reservoir routing with and without breach scenarios. The unsteady flow routing method recognizes and record the water surface slope along the reservoir as the inflow arrives, as well as the change in water surface during a break of the dam. Reservoirs with long narrow pools will exhibit greater water surface slope upstream of the dam than reservoirs that are wide and short. Therefore, the most accurate modelling technique to capture pool elevation and outflows of long narrow reservoirs is full dynamic wave (unsteady flow) routing. For wide and short reservoirs, level pool routing may be appropriate.

In level pool routing, the engineer must enter an elevation-volume curve as part of the storage-area data describing the reservoir. The minimum elevation of the two upstream cross sections should be roughly equal to the minimum elevation specified for the storage area in order to prevent any instability once the storage area is emptied as the layout shown in figure 2.7.

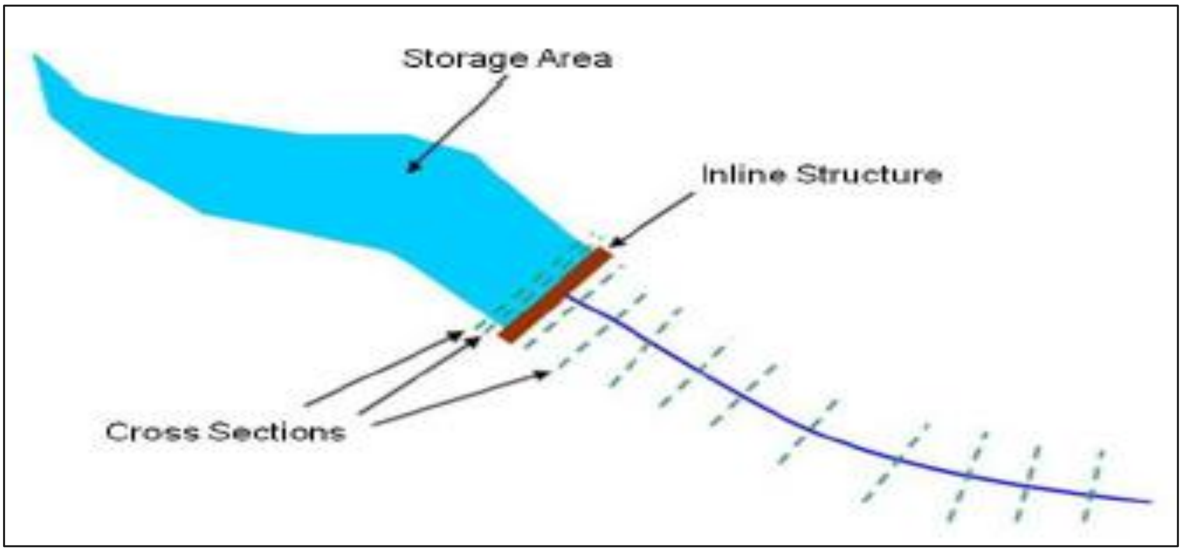


Figure 2-7: Storage Area and Cross Sections Layout for Level Pool Routing

When a dam break is modelled, the breach discharge will be computed by using the same equations as the full dynamic wave method. The only difference is that the water supplied to the dam will come from the storage area, and the storage area elevation will drop as a level pool as water flows out of the breach. As noted above, when a rapidly forming breach occurs, the water surface upstream of the dam will often have significant slope to it. With the level pool routing method, the water surface in the reservoir is horizontal. This may or may not produce significant differences in the outflow hydrograph, depending on many factors.

Moreover, a criteria called draw down is used to decide in selection of reservoir routing methods. The drawdown number, Dn, is a measure to determine if the level pool reservoir technique is acceptable routing for the assumed dam break events.

The drawdown number equation is:

$$Dn=Fc*Ft$$

Where, Fc = Compaction factor,

Ft= Translation factor.

In which a compaction factor measures compactness of a reservoir and the equation is:

$$Fc=H/L$$

Where, H = Breach height,

L= Reservoir length.

And also the translation factor which measures the rate at which the water replenish the drawdown effect. The equation for translation factor is:

$$Ft=ct/L$$

Where, c = Wave celerity, the equation for wave celerity= $(gD)^{0.5}$ Which D=H/2

t= Breach formation time.

L= Reservoir length.

Hence, based on the draw down number and the graph below, it is possible to know and decide the error if level pool reservoir routing is used.

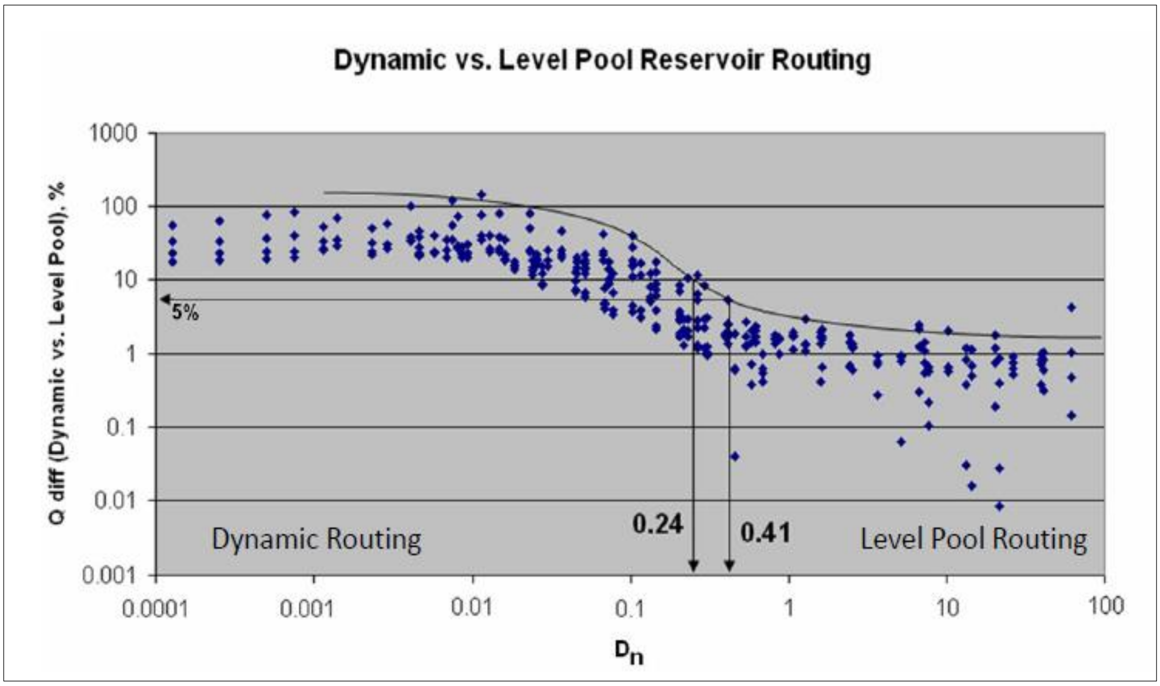


Figure 2-8: Reservoir Routing Error for Level Pool Assumption

Furthermore, the dam crest is modeled as an inline weir and either a piping failure or overtopping failure is simulated with enlargement of the breach occurring over time as defined by a specified breach progression. Flow through the piping hole is calculated as orifice flow and flow through the breach is calculated as weir flow. The water surface profile upstream of the dam is back-calculated using unsteady momentum and hydraulic principles for each time step and the resulting drawdown through the hole and/or breach produces an outflow hydrograph. Resulting water levels for each time step downstream of the dam are used to model potential backwater effects and the weir and orifice coefficients are automatically adjusted for submergence, if necessary. It can also model a piping failure that does not progress to the point of collapsing the crest. In this scenario, the piping hole is simulated as a sluice gate (HEC, 2016). The model gives different outflow hydrograph for each breach parameter estimated. These various hydrographs are then routed downstream to identify the most hazardous flood.

Moreover, if all the breach estimates, for a given event/pool elevation, end up converging to the same flow and stage before getting to any population at risk and potential damages areas, then the final selection of a final set of breach parameters should not affect the computations and a simple mean value should be used. However, if the various set of breach parameters produce significantly different flow and stage value at downstream locations (populations at risk locations and potential damages zones), then engineering judgment will need to be used to pick a set of values that are considered most likely. Conservatively high and low values should not be used, as this will bias the overall results (HEC, 2016).

Another check for reasonableness will be done by evaluating the breach flow and velocities through the breach, during the breach formation process. This can be accomplished by reviewing the detailed output for the inline structure (Dam) and reviewing the flow rate and velocities going through the breach. This output is provided on the HEC-RAS detailed output table for the inline structure. There are two things to check here:

- If the model reaches full breach development time and size, and there are fill very high flow rates and velocities going through the breach ,then this is a sign that your breach is too small ,or your development time is to short(unless there are some physical constraints limiting the size of the breach)
- If the flow rate and velocities through the breach become very small before the breach has reach its full size and development time, then this is an indicator that your breach size may be too large or your breach time may be long. Additional factors affecting this could be your breach progression curve and the hydraulic coefficient (weir and piping) you used.

When you get in to the situation described above in either scenario 1 or 2, the breach size and development time should be re-evaluated to improve the estimates for that particular structure (HEC, 2016).

Moreover, the computational interval is used in the unsteady flow calculations. This is probably one of the most important parameters entered in the model .The interval should be close enough to precisely define the rise and fall of the routed hydrographs. A general rule of time to use a computation interval is equal or less than the time of rise of hydrograph divided by 20.

Therefore

$$\Delta t < Tr/20$$

Where, Tr= Time of rise of hydrograph to be routed

A second way of computing the appropriate time step is by applying a numerical accuracy criterion called the courant condition. The fundamental idea is that the computing interval should be equal to or less than the time it takes for water to move from one cross section to the succeeding one. Use of a time step based on the courant condition will give the best numerical solution, but it may cause the model to take a lot longer time to run.

The estimated courant number is:

$$Cr = (Vw\ \Delta t)/L \leq 1.00$$

Therefore

$$\Delta t \leq (\Delta x/Vw)$$

Where, Vw= Flood wave speed, which is normally greater than the average velocity

Cr=Courant Number, A value =1.0 is optimal.

Δx=Distance between cross sections

Δt=Computational time steps

Furthermore, McBreach is used which is a companion software application to the hydrologic engineering center's river analysis system (HEC-RAS) that performs probabilistic dam breach modelling. Using Mc breach, the hydraulic modeller has the ability to randomly sample about the predefined statistical distribution, all breach parameters for inline structures, lateral structures and SA/2D area connection. In addition to breach parameters, the user can include the model inflow's hydrograph in the probabilistic analysis randomly sampling hydrograph flow and duration multipliers.

Through a Monte Carlo simulation, Mc breach produces exceedance probability peak discharge and their respective sampled breach parameters that can be used to produce exceedance probability inundation (EPI) maps. Ultimately a design exceedance probability peak discharge is selected for decision making. Several metrics are available for the user to gage statistical convergence during the probabilistic simulation.

Using the techniques presented by Goodell (2014), McBreach communicates with HECR-RAS and exchange information at the realization level. The connection with HEC-RAS is managed through the use of HECRAS controller as well as programmatic reading/writing of the HECRAS input files (McBreach 507 user manual, 2019).

McBreach satisfies a need in the dam safety community to augment the overlay conservative deterministic approach to a dam breach modelling with probabilistic approach that quantifies uncertainty in the analysis. McBreach allows for decision making based on risk and uncertainty and compliments dam safety jurisdictional desires to move towards risk informed decision (McBreach 507 user manual, 2019).

2.11.2 MODELING THE PROPAGATION OF FLOOD

Hydraulic models are broadly categorized into two types: steady flow models and unsteady flow models. Time dependent flow changes are explicitly analysed as a variable in unsteady flow, while a steady flow analysis models does not consider time all together (Colorado, 2010).

Steady flow analysis can estimate a water surface elevation and flow velocity at a specified cross section for a known flow using Manning's equation under the assumption of gradually varied flow conditions whereas, unsteady flow analysis can be used to evaluate the downstream attenuation of the flood wave, providing a more accurate estimate of flood magnitude and velocity at critical locations. The full unsteady flow equations have the capability to simulate the widest range of flow situations and channel characteristics. Unsteady flow analysis can be more divided into two types based on the lateral distribution flow: one-dimensional and two-dimensional(Purvang, 2013).

Inundation maps show the flood contour for natural floods of certain return periods, in most cases up to PMF, and flood contours of potential dam break floods caused by a "sunny day failure" and a failure superimposed over certain natural "base flood conditions". (ICOLD, 1998).HEC RAS is selected as a model to route the most catastrophic outflow hydrograph over the downstream channel and flood plain and prepare an inundation map. Because of its comprehensive channel flow analysis capabilities and capacity to model unsteady flood wave propagation and identify flood prone locations, the HEC-RAS software has gained widespread popularity among hydraulic engineers and researchers. HEC-RAS is the most widely used hydraulic model for dam safety analyses in the United States and can be utilized for steady and unsteady flow analyses (Colorado, 2010). The HEC-RAS computational program has the ability to model extreme flow dynamics in the downstream of the reach due to a dam break flood waves and produce water surface profiles over the length of the modeled area (Asburry and Goodell, 2009). In particular, the recent versions of HEC RAS are equipped with RAS mapper in which the outputs like water surface profiles can easily be converted to flood inundation to visualize the Identification of the inundated areas, inundation depth, speed and duration, as well as the impact that flood water without using another software. Furthermore, many studies indicate that the solution found by HECRAS is stable and trustworthy (Ackerman and Brunner, 2006) even though Instability problems may arise due to the complexity of solving routines.

HECRAS can also perform one dimensional or two dimensional flood routing analysis for Dam break flood waves by using unsteady flow equations as flow depth, velocity and discharge change with time. Hydraulic routing (unsteady flow routing) solves the continuity and momentum equations together, in order to route the hydrographs and compute the water surface elevations. Moreover, HEC-RAS is able to take into account hydraulic effects of bridges, culverts, weirs, and other structures in the river and floodplain on water surface calculations This reasonable approach is based on the assumption that the water is incompressible, pressure is hydrostatic, vertical acceleration is negligible, and waves are non-dispersive.

One-Dimensional Equation (1D)

The one dimensional continuity equation and momentum equations can be written in partial differential equation form, with respect to depth (h) and velocity (u), and shown as follows:

Continuity Equation:

$$(\delta h/\delta t)+ (\delta\ (hu)/\delta x)-q=0$$

Momentum Equation:

$$(\delta u/\delta t)+ (u\delta u/\delta x) +g\ (\delta h/\delta x-So+Sf+Sh) =0$$

Where:

U=velocity in the x-direction

h=Depth of water

g=Gravity

t=time

x=distance in the direction of flow(x plane)

q=Lateral inflow term (Source/Sink)

So=Bed slope

Sf=Friction slope, from manning’s equation

Sh=added force term (additional minor losses)

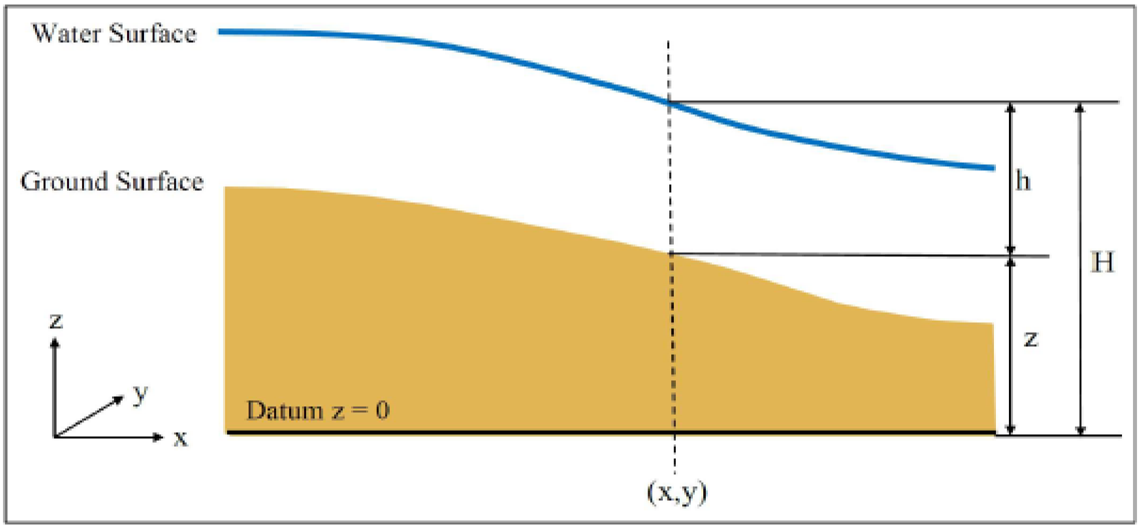


Figure 2-9: Definition of symbols used in the 1D and 2D equations of motion

One-Dimensional Equation (2D)

The two dimensional continuity equation and momentum equations can be written in partial differential equation form, with respect to depth (h) and velocity(u, v, U,V) assume that the vertical velocities are much smaller than the horizontal velocities, which leads to hydrostatic pressure distribution in a channel cross section, and shown as follows:

Vertically averaged Continuity Equation:

$$(\delta h/\delta t)+ (\delta\ (hu)/\delta x) + (\delta\ (hv)/\delta y)-q=0$$

Vertically averaged Momentum Equation:

$$(\delta u/\delta t)+ (u\delta u/\delta x) + (v\delta u/\delta y) =-g\ (\delta H/\delta x) +Vt\ ((\delta^2 u/\delta x^2) + (\delta^2 u/\delta y^2))-c_f\, u+ f_v$$

$$(\delta v/\delta t)+ (u\delta v/\delta x) + (v\delta v/\delta y) =-g\ (\delta h/\delta x) +Vt\ ((\delta^2 v/\delta x^2) + (\delta^2 v/\delta y^2))-c_f\, v+ f_u$$

Where:

v=velocity in the y-direction

y=Distance in the lateral direction (y plane)

H=water surface elevation (Z+ depth)

Vt=horizontal eddy viscosity coefficient

Cf=Bottom frication coefficient

f=Coriolis parameter

t=time

x=distance in the direction of flow(x plane)

q=Lateral inflow term (Source/Sink)

So=Bed slope

Sf=Friction slope, from manning's equation

Sh=added force term (additional minor losses)

Another form of momentum equation is a diffusion wave form which is a simplification of the 1D and 2D forms of the equations. The diffusion wave approximation is often used in both 1D and 2D solutions forms. To get the diffusion form of the equations, simply drop the acceleration terms (changes in velocity with respect to time and space) in the momentum equations. This form of the momentum equation only contains gravity, friction, and hydrostatic pressure forces. The diffusion form of the momentum equations can be combined with the continuity equations, and written in terms of solving for only the water surface elevation. This makes the equation easier to solve, generally more stable for a wide range of problems, and will require much less computational time. However, without the acceleration terms in the equations, the diffusion wave equations are less accurate than the full equations, and then are less applicable to the full range of problems that the modeler may need to solve.

Furthermore, characterization of hydraulic system requires parameterizations, often purely empirical, to describe physical process that can not directly determined from the solution of the above stated basic conservation equation. The use of manning's –n value to define the hydraulic roughness is one of classic example for empirical parameterization in hydraulic modelling. The value of Manning' n is highly variable, depending on a variety of factors such as surface roughness, vegetation cover, channel irregularities, channel alignment, scour and deposition, obstruction, channel size and shape, stage and discharge, seasonal changes, temperature, suspended and bed load materials, and so on.(HEC, 2016).

There are definite advantages the 2D modeling approaches (TD-41):

2D Modeling Advantages:

- The flow path of the water, for all events, does not have to be known to develop the model. However, the extent of the flooding does need to be correctly defined.
- The direction of the flow can change during the event. Water can move in any direction, based on energy and momentum of the flow.
- Velocity, momentum, and the direction of the flow are accurately accounted for with 2D modeling. This accountability is especially true for flow going over roads, levees, barriers, structures, around bends, and at flow field. Around bends, 2D models produce accurate water surface elevations, but velocity distributions might be erroneous due to the existence of helical flow.
- Energy and force losses due to contractions and expansions, etc. are directly accounted for, and do not require empirical coefficients, increased roughness, or user defined ineffective flow areas.
- The mapping of the inundated are, as well as velocities, and flood hazards (depth x velocity) is more accurate.
- Detailed modeling of hydraulic structures, in a full 2D modeling approach, can provide more insight into the distribution approaching, going through, and coming out of a structure.

As per the USACE HEC Training document -TD41, Modeler Application Guidance for Steady vs. Unsteady, and 1D Vs. 2D vs. 3D Hydraulic modelling, recommends 1D and 2D for Dam breach modelling.

3 GENERAL DESCRIPTION OF THE STUDY AREA AND DATA COLLECTION

3.1 GENERAL variable OF THE STUDY AREA

3.2 LOCATION

The Ribb Dam and Irrigation Development Project is located on the eastern side of Lake Tana Sub Basin, Nile Basin and within the administrative boundary of South Gondar Zone of the Amhara National Regional state, Ethiopia. The study area is located between Latitude of 11^0.53' to 12^0.8' north and a Longitude 37.31' to 38.36' east within an altitude ranges from 1784 to 1950m m.a.s.l. The Ribb river sub basin including the study area is shown in the figure 3.1. The Dam axis is located in between the geographic grid reference UTM E 392253 and N 1330434 at right bank, E 391445 N 1330504 at central part with a curve, and E 390907 and N 1330225 at left bank, with an original ground elevation ranging from1773 to 1945.50 m.a.s.l and a dam height of 72.50m.

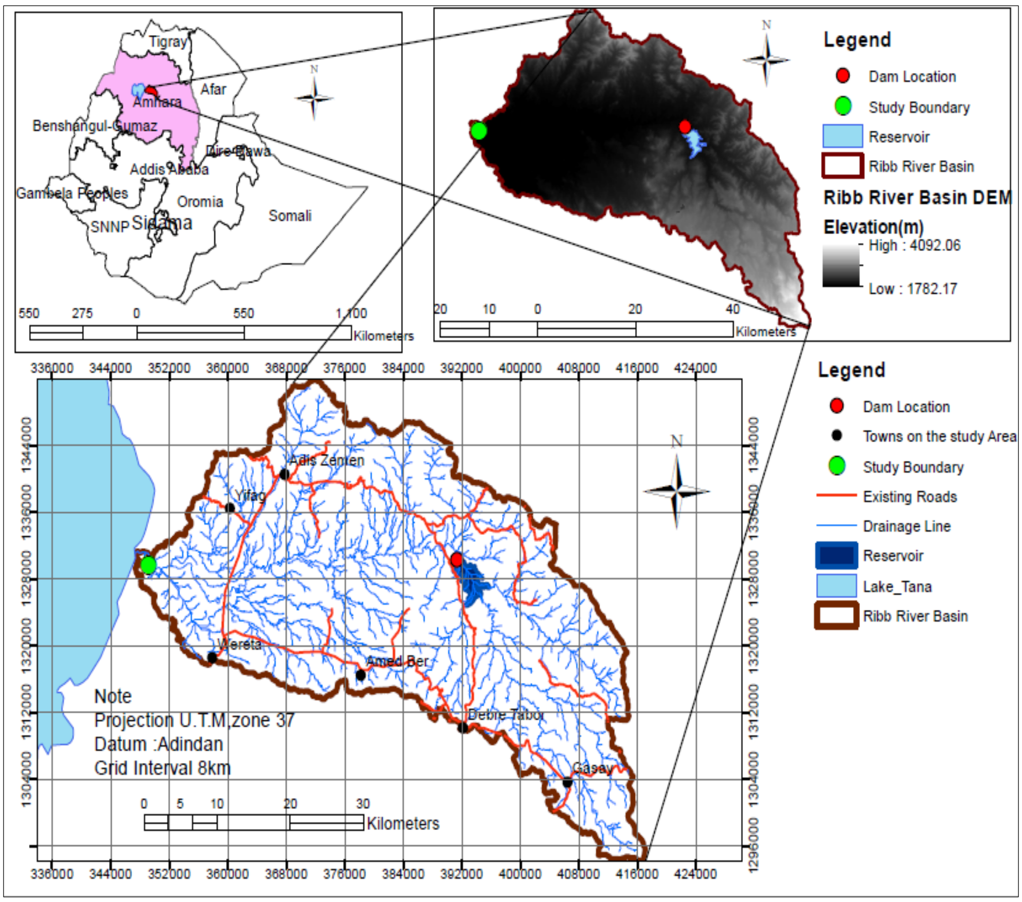


Figure 3-1: Location map

Access to the dam site is possible from the town of Addis Zemen using the existing dry weather road, which is about 40 km long.

3.2.1 CLIMATE

Ethiopia is generally characterized as residing in a tropical to sub-tropical climate, with the primary rainfall season typically occurring between the months of June to September. A secondary rainfall season usually occurs from February to May. The main synoptic influences on the hydrometeorology in Ethiopia include the monsoon effects from the Indian and Atlantic Oceans, the impacts of the Inter-Tropical Convergence Zone seasonal movements (to the north during the summer months), and the influences of the low-level jet stream. During the main rainy season, precipitation amounts are strongly influenced by sea surface temperatures and the active jet stream movement northward (WWDSE, 2007).

The project region around Lake Tana tends to be hydro meteorologically homogeneous with minor orographic impacts on precipitation evident near Debre Tabor in the east and Gish Abay in the south. Annual Isohyets over Tana Sub basin are shown in Figure 3.2.The mean annual precipitation is around 1,400 mm in the upper part and around 1,200 mm in the lower part. The Ribb Basin's climate is characterized by a rainy season from May to September, with monthly rainfall ranging from 65 mm in May to 411 mm in July. The dry season, from October to April, has a total rainfall of about 8% of the mean annual rainfall. Dependable rainfall varies from less than 13 mm during the dry season to 80–275 mm/month during the period of June to July/August, equivalent to 40–80% of the average values.

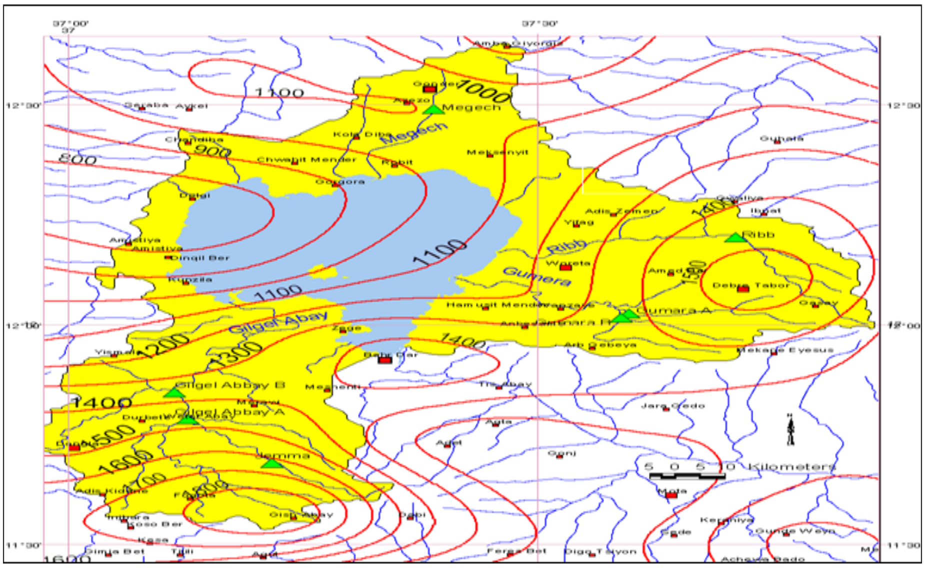


Figure 3-2: Mean Annual Isohyets (mm) over Lake Tana Sub-Basin

Temperature variations throughout the year are minor (19 ºC in December to 23 ºC in May), with maximum and minimum temperatures of 30 °C and 11.5 °C, respectively. Humidity varies between 70% in Dec and 88% in August. Wind speed is low, thus minimizing potential evapotranspiration values between 95 mm/month in December and 140 mm/month in April. Sunshine duration is reduced to 6.0–6.5 hours during July and August.

3.2.2 WATERSHED

The Ribb River, which is 112 km long, has a total drainage area of about 1,790 km² and a catchment area of about 685 km2 at the dam site with an average annual and base discharge of 11.6m³/s and 0.4 m³/s. The Upper Ribb watershed is characterized by a mountainous, wedge-shaped and average steep-sloped (3.6%) watershed as shown in figure 3.3 (WWDSE,2007). According to the river area morphology, it has a slope range from 0% to 9.8% .The highest elevation of the watershed is about 4,100 m in its southeastern part. In the low and middle reaches of the river, especially in the extensive alluvial plains bordering Lake Tana, the river meanders its way and flows slowly, causing sediment deposits, high water table and overflowing of riverbanks during the rainy season, due to insufficient riverbed conveyance. Consequently, major problems of flooding and waterlogging are frequent in the area and causing problem for the settlements and agriculture practice.

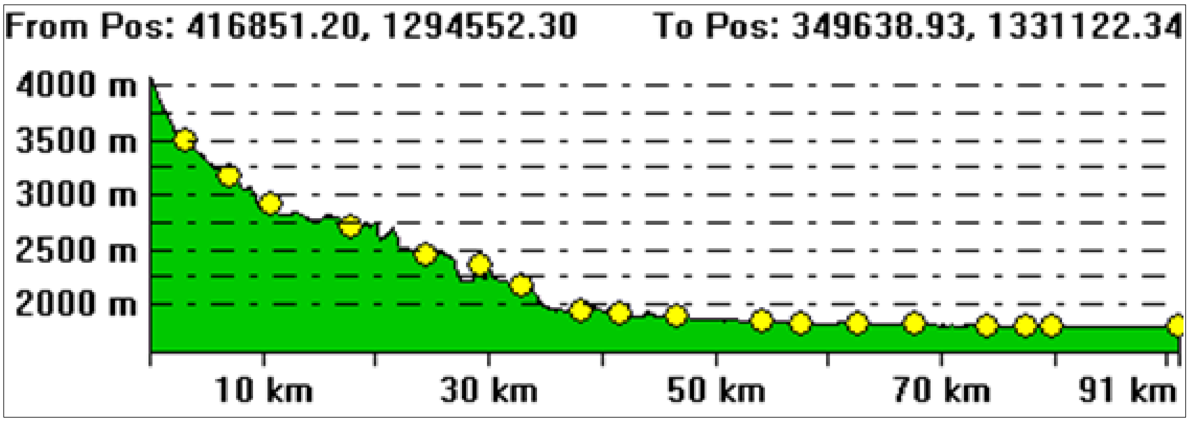


Figure 3-3: Gradient of Upper Ribb River from the sources to near the dam site up to 56km, and from 75km to Lake Tana.

3.2.3 DAM AREA GEOLOGY

The dam site and surrounding area is composed of overburden materials on strong to very strong volcanic rocks. Moreover, the overburden materials are of varying thickness along the dam axis and in the reservoir area. They are suitable as the impermeable-core section of the embankment (WWDSE, 2010).

3.2.4 LAND USE

The Land use for all sub basins was primarily agricultural, agro-pastoral, and pastoral. The overall land use of Ribb river basin is presented in figure 3.4. The dam site area is extensively used for farming and settlements also being denser at the upper reservoir slopes and top of the hills. Moreover, the floodplains soil is largely black cotton soil and suitable for irrigation development (WWDSE, 2007).

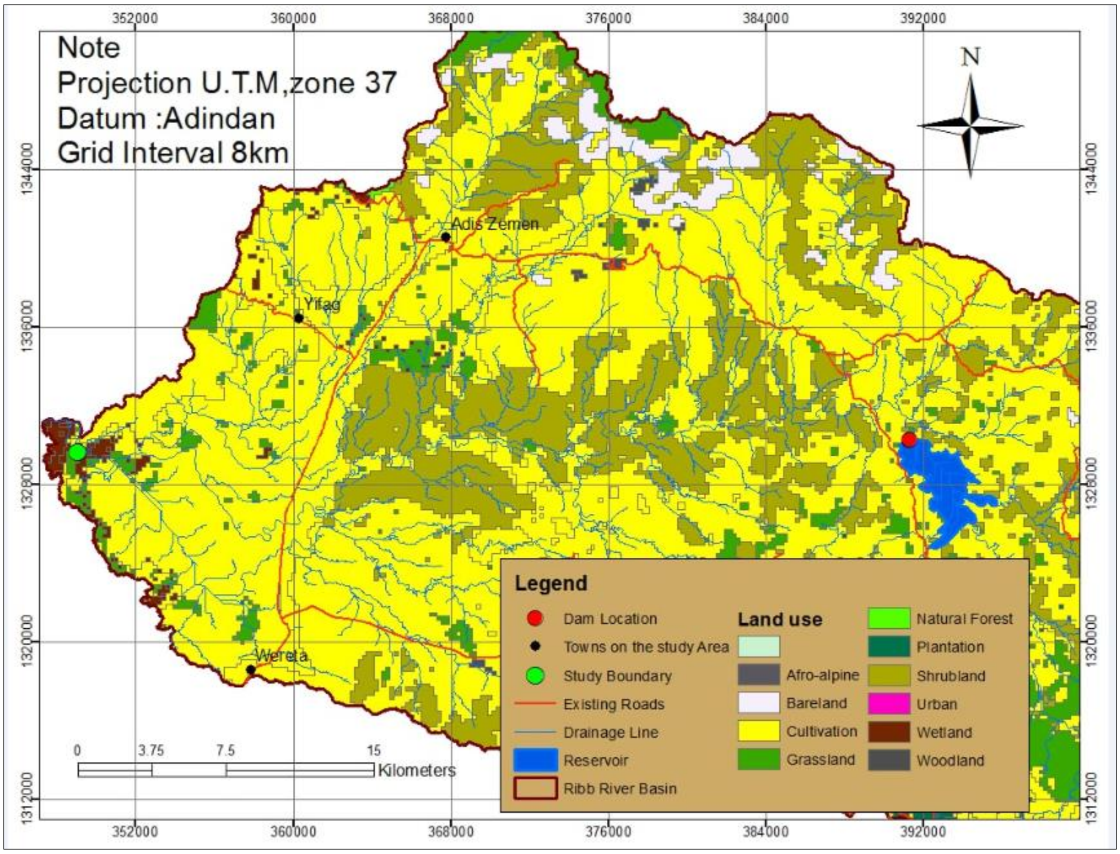


Figure 3-4: Ribb river basin Land use

3.3 DATA COLLECTION

The modeling domain is composed of Ribb dam and its reservoir at the upstream boundary and the river cross-sections up to end of Lake Tana, west direction, at the downstream and the vast flood plain on both sides of the river banks. Furthermore, the pickup weir and trunk highway are located at the downstream boundary with the reach length of 29.74 Km and 50.98km respectively. Furthermore, the planned command area for the irrigation is located at downstream of the pickup weir up to Lake Tana. All these places or routes are key location s in which important infrastructures, irrigation commands or center of populated villages are existed.

The hydraulic modelling of the Ribb Dam breach requires the following sets of data.

- Topographic data
- Salient features of the Dam
- Hydrologic and reservoir storage data
- Downstream civil infrastructures
- Population and households at flood risk
- Downstream boundary conditions and other hydraulic parameters.

3.3.1 TOPOGRAPHIC DATA

Topographic data collection is an essential part of dam breach modeling. Dam breach modeling and flood mapping require an accurate digital elevation model (DEM) from which to extract cross sections and also map the flood surface. The DEM must be of large enough extent to cover the possible limits of flooding as well as contain enough spatial detail within the river channel to enable accurate hydraulic modeling.

In this aspect, an orthophotos and DEM points with 12.5 meter grid cell from the Amhara region rural land administration and use bureau were generated for Fogera and Libo Kemkem woreda by LiDAR (Light Detection and Ranging) technology. The LiDAR instrument principally consists of a laser, a scanner, and a specialized GPS receiver and generates precise, three-dimensional information about the shape of the Earth and its surface characteristics. Hence, the data found to be accurate enough for defining the alignment and cross-section of river and the terrain features up to the flood plain area as shown the figure below.

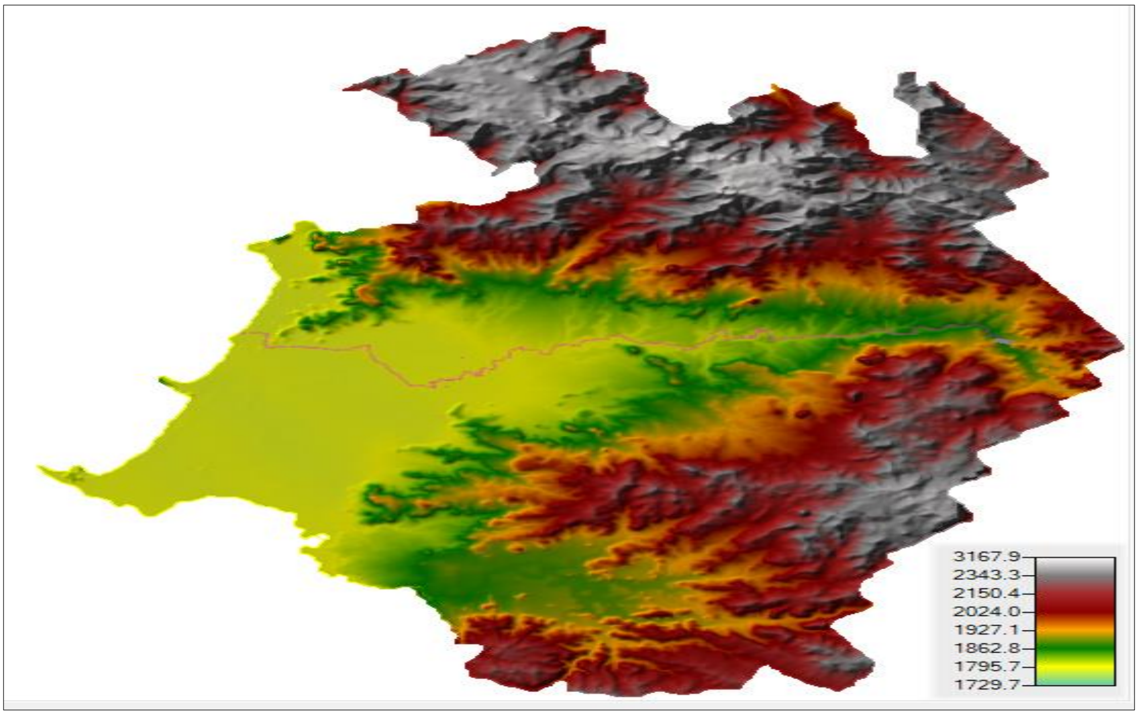


Figure 3-5: Study Area Digital Elevation Model

Furthermore, high resolution satellite images of the study area is captured and stored from Google earth and used to assess the detail information of the study area including the important locations. Besides, the satellite images data will be underlined to visualize the river network for delineation and inundation mapping work.

3.3.2 SALIENT FEATURE OF THE DAM

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

- Location: UTM E 392253, N1330434 and E 390907, N 1330225
- Dam Type: Earth-Rockfill Dam
- Embankment Core: Compacted Clay Material
- Embankment Inner Material: Compacted Shell Material
- Embankment Outer Material /Upstream Slope Protection: Compacted rockfill
- Dam Crest Elevation: 1945.50m.a.s.l
- Crest Length: 800.00m
- Crest Width: 10.00m
- Spillway Length 107.00m
- SpillwayHeight 5.50m
- Dam Height above the River Bed Level: 72.50m
- Dam Bed Width (Maximum): 445.00m
- Normal Water Level: 1940.00 m.a.s.l
- Maximum Water Level: 1943.00 m.a.s.l

- Upstream Face Slope: 3H:1V
- Downstream Face Slope: 3H:1V
- Reservoir Capacity (NWL): 235.00Mm3
- Reservoir Capacity (MWL): 270.00Mm3

All the specified geometry and technical data are used in the development of cross section of the dam body and spillway in the HEC RAS model and then to the predication of the breach parameters. Furthermore, Layout and geometry of the Ribb dam is given in the following figure 3.6 and 3.7 respectively.

Moreover, there is saddle dam, 629 m long, 18 m maximum height, is closing a topographic depression on the ridge, southwest of the left abutment. There are also other small saddle dams; Saddle Dam B and Saddle Dam C closing ridge depressions. Saddle dam B is 295 m long and about 2.2 m high to the South Eastern of the main dam. Saddle dam C, 203m long with same height to Saddle dam B, is further South Eastern of the main dam next to Saddle dam B.

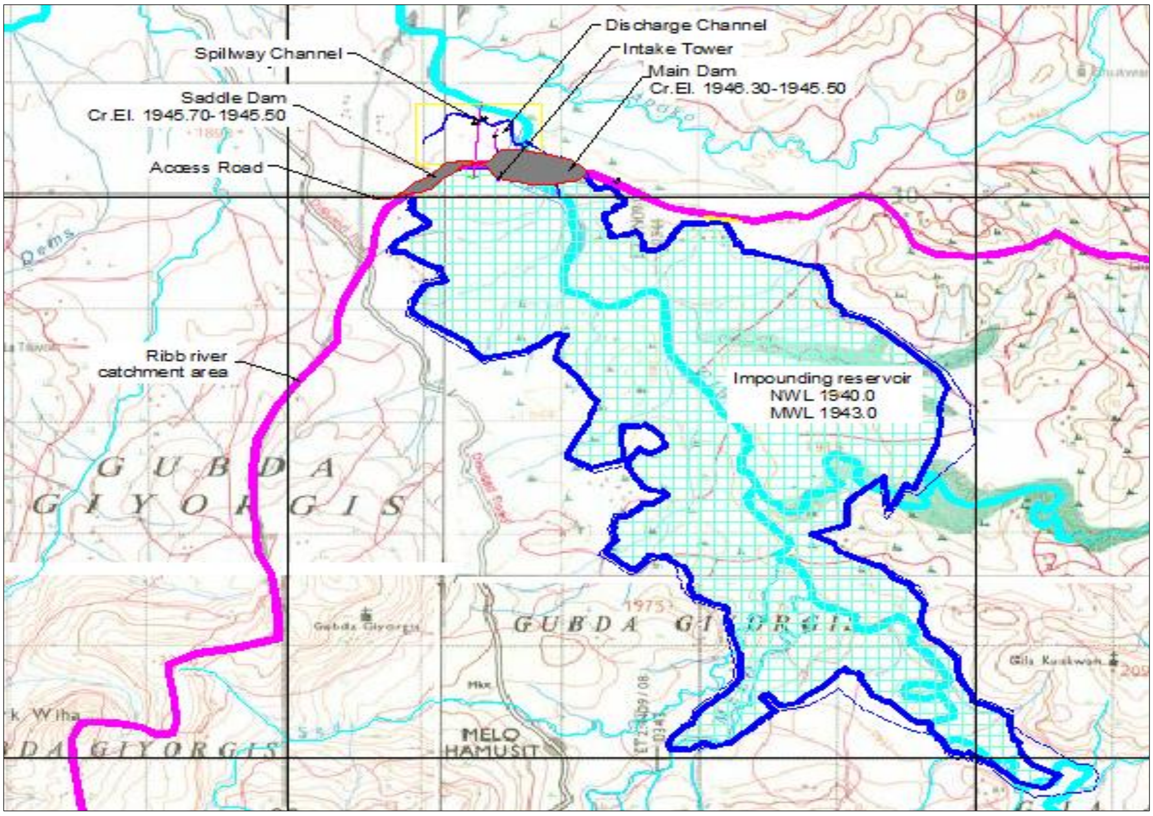


Figure 3.6: General layout of impounding reservoir and dam

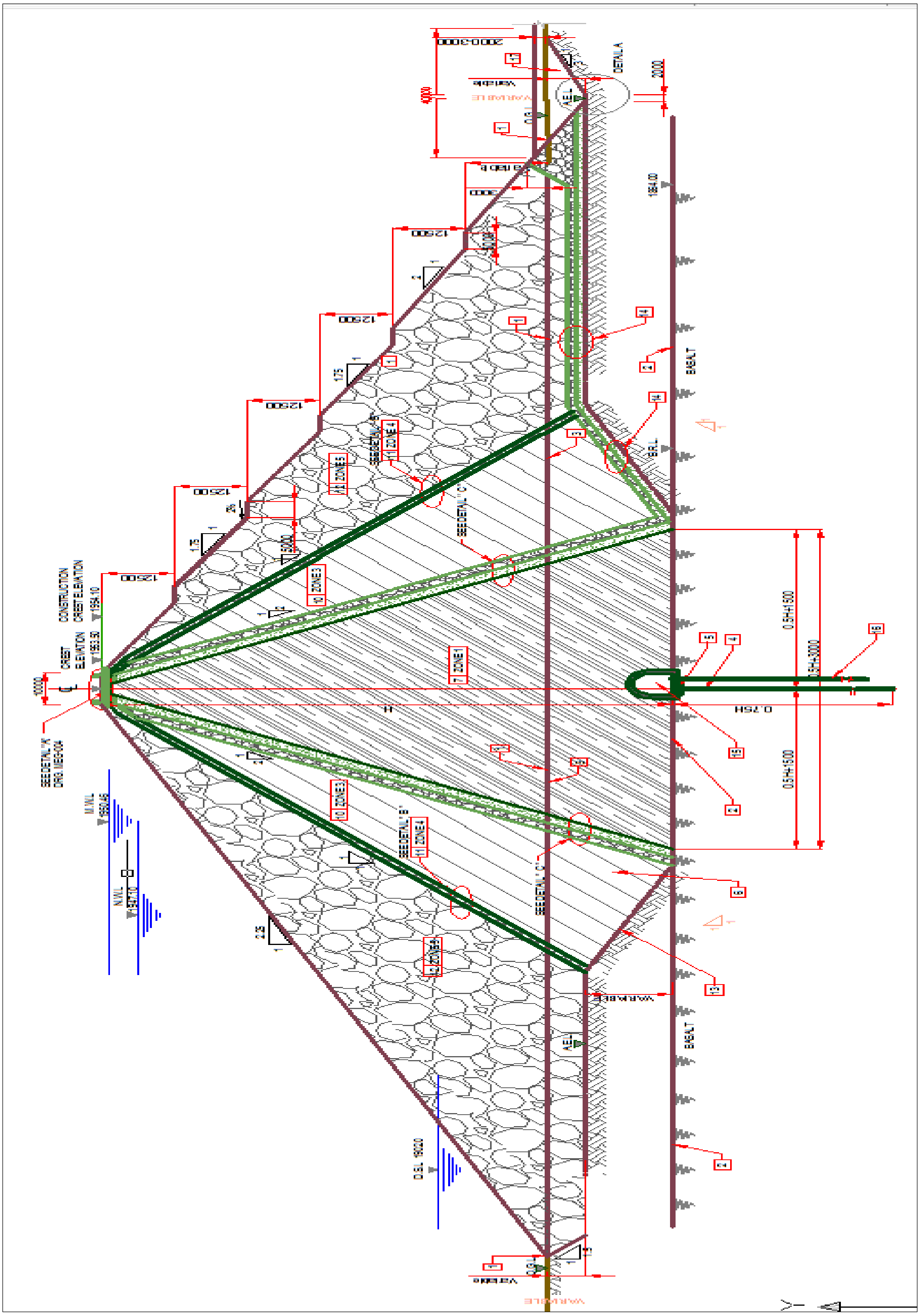


Figure 3.7: Main Dam cross section drawing

3.3.3 HYDROLOGIC DATA AND RESERVOIR STORAGE CAPCITY

The hydrologic data category includes the flood events hydrograph (inflow hydrograph, PMF) and reservoir capacity as provided by Water Works Design and Supervision Enterprise (WWDSE, 2007). Based on the data, The Author generate 1.45PMF, 1.5PMF and 1.55PMF Inflow hydrograph

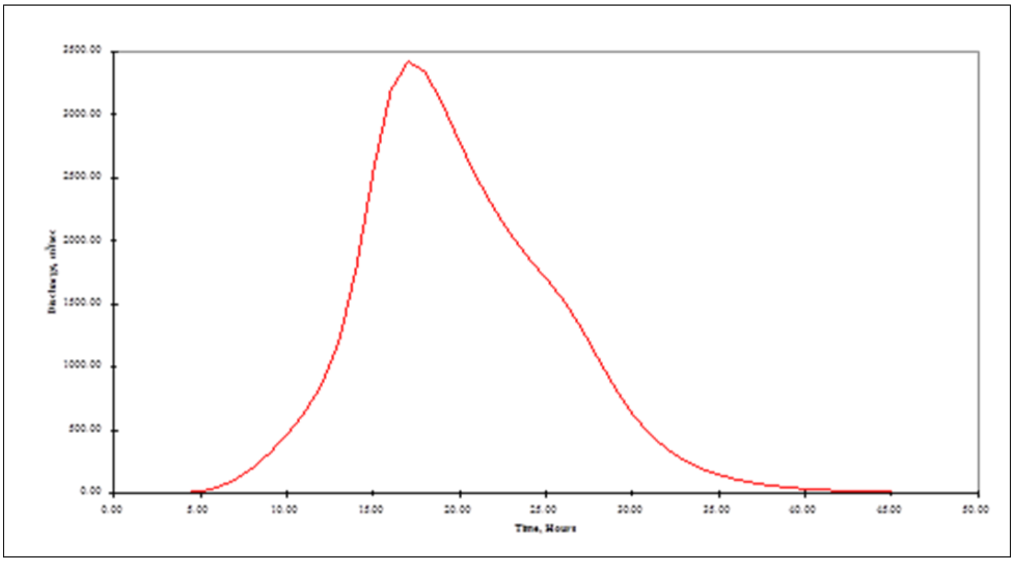


Figure 3.8: The 24-hour PMF hydrograph at the Ribb Dam site

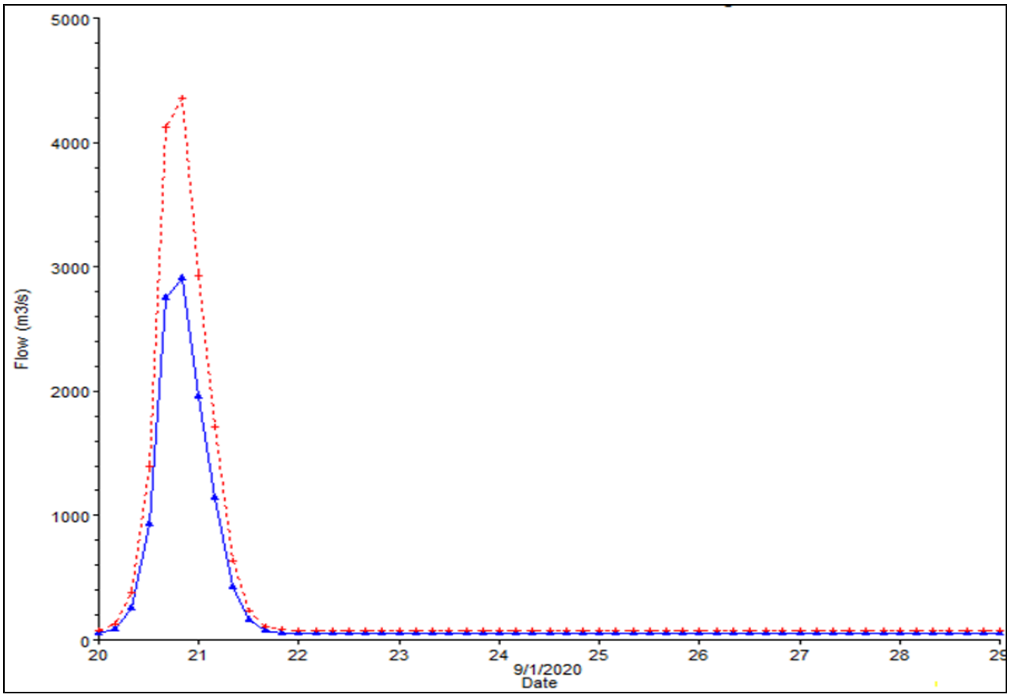


Figure 3.9: The modified 1.5PMF hydrograph at the Ribb Dam site

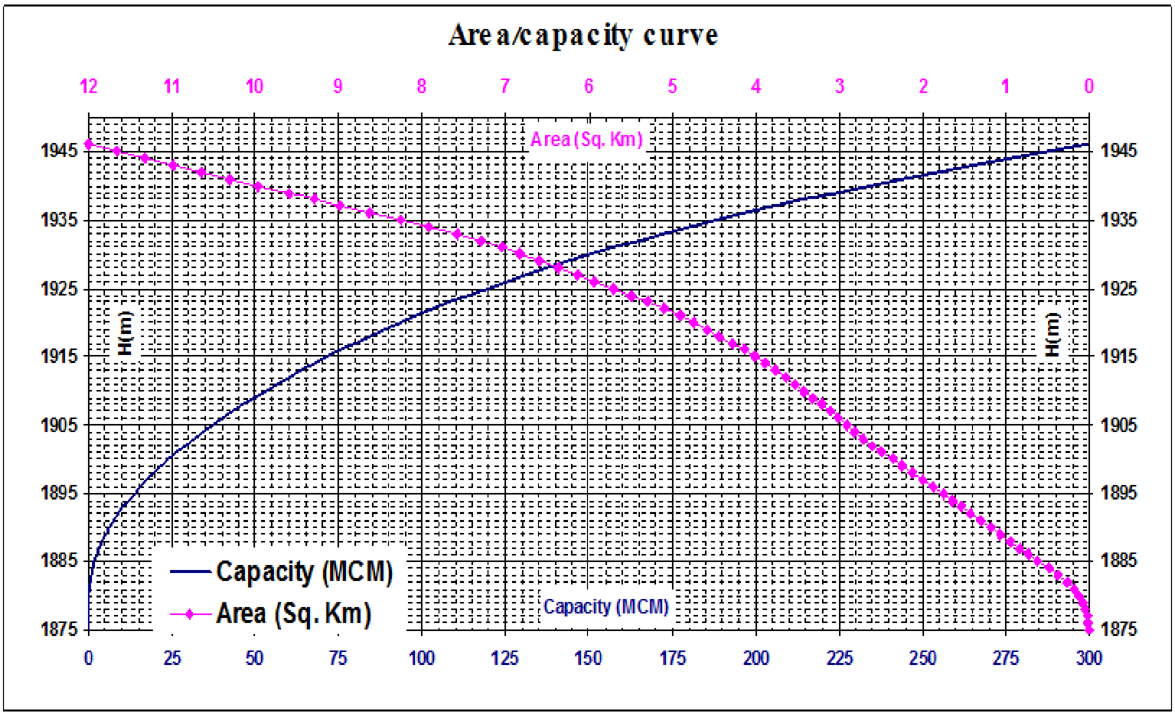


Fig. 3.10: Area/capacity curve

3.3.4 DOWNSTREAM INFRASTRUCTURES

3.3.4.1 IRRIGATION AND DRAINAGE SCHEMES

According to the Ethiopia Ministry of Water, Irrigation & Energy (BRL Ingenerie, 2015), the Ribb Irrigation Project is intended for about 14,460 ha and an additional of 2865ha and 2601ha respectively as future development, on both sides of Ribb River, using water released from the constructed Ribb Dam. The project is under progress in order to complete the first phase of the work.

Moreover, the pickup weir across the river, located downstream from the dam, will regulate the head for the irrigation water abstraction.

The main data of the diversion weir and intake structures are as follows:

- Location: UTM E E373028 N1329828
- Bed level at weir site: 1,798.50 m.a.s.l
- Pond level: 1,802.50 m.a.s.l
- Crest level: 1,802.50 m.a.s.l
- Type of crest: Ogee-shaped
- D/S high flood level: 1,805.32 m.a.s.l

- Length of weir 64.0 m

Moreover, the pickup weir is represented as internal model boundary condition even though its effect compared to the flow depth is minimal and also there is a high probability of being washed away.

3.3.4.2 TRUNK HIGHWAY

There is also a Trunk Highway connecting Bahr Dar with Gonder traverses across the Ribb River. The road is equipped with a cross drainage structures for uninterrupted access at all times. There is a major Bridge across Ribb River and more than 50 box and pipe culverts of various sizes. Most of these crossing culverts openings are currently partially or entirely blocked (BRL Ingenerie, 2015).

Hence, for these studies the Author consider only the big crossing structures as mentioned in detail below as the other remaining structures passing percentage of the flood water is minimal (Michael A. Ports, 2016).

3.3.4.2.1 RIBB BRIDGE

Ribb is a three span (2*15 m+18 m) clear span bridge. The maximum clear height is 4.5 m. The longitudinal length of the Bridge is 8.5 m.

Plate 3-1: Flooding at Ribb Bridge and Road Side Flood Plain (BRL Ingenerie, 2015).

3.3.4.2.2 SHENI BOX CULVERT

The Box culvert provided for road crossing across Sheni River consists of eight box opening. Each rectangular opening is 2m wide and 1.8high. The length of the box culvert is 13.0 m. Currently 4 openings are totally clogged as shown below.

Plate 3-2: Sheni River road crossing box culvert (BRL Ingenerie, 2015).

3.3.4.2.3 MARZA BOX CULVERT

The Box culvert provided for road crossing across Marza River consists of eight box opening. Each rectangular opening is 3m wide and 3m high. The length of the box culvert is 13.0 m. The openings of the box culvert are partially and entirely blocked as shown below.

Plate 3-3: Marza River road crossing box culvert (BRL Ingenerie, 2015)

3.3.4.2.4 NACHURIT BOX CULVERT

The Box culvert provided for road crossing across Nachurit River consists of ten box opening. Each rectangular opening is 2m wide and 2m high. The length of the box culvert is 13.7 m. Most of the openings are partially and entirely blocked as shown below.

Plate 3-4: Nachurit River Box Culvert partially to entirely blocked (BRL Ingenerie, 2015)

3.3.5 POPULATION AND HOUSEHOLDS AT FLOOD RISK

The target population for the project constitutes the people living in the command as well as the reservoir areas, which comprise mainly two woredas namely Libo Kemkem and Fogera woredas. The flood might affect twenty five kebeles from both woredas namely Agid Qirgna, Bira Abo,Agita, Estifanos,Shamo Godguadit, Birkutie, Bura Egaziabher Ab,Angot, Kidist Hana, Wagetera, Nabega, Bambik tsiyion,Genda wuha,Teza Amba, Kab, Tibaga, Shina Tsion, Wetneb, Addis Bete Christian, Rib Gabreal, Diba na Sifatira, Worta town zura, Abu Kokit, Shaga and Shina. Actually, most kebele villages are scattered far away from each other. Moreover, Woreta town is also located in the Fogera woreda. Moreover, the floods also affect the Farta and Ebenat Woredas which are the location of the dam area and reservoir area.

Based on the 2007 national census conducted by the Ethiopia Central Statistical Agency of Ethiopia (CSA, 2007), the two woreda's total population and household are presented in the following table:

Table 3-1 Population and Household in the study area

Woreda Naming	**Number of population**		**Total**	**Population density per Km^2**	**House hold number**	**Average persons to house hold**
	Male	**Female**				
Libo						

Kemkem	100,987	97,448	198,435	198.48	45,399	4.37
Fogera	116,509	119,400	228449	205.55	52,905	4.32

Downstream risk analysis is not the focus of this study but in order to categorize and asses the dam's hazard level it's vital to study downstream population size, main facilities and infrastructures. The large population size, the downstream irrigation structures, trunk highway and the different infrastructures provided for the community alone can't be used to classify the dam as per the dam hazard classification category. In fact the above listed points are basic information but a study of emergency action planning reveals the dam hazard classification category.

3.3.6 BOUNDARY CONDITION DATA AND HYDRAULIC PARAMETERS

3.3.6.1 DOWNSTREAM BOUNDARY CONDITION DATA

Tana Lake water level is found to be the best alternative for creating downstream hydraulic boundary condition of the River model; thus the whole river reach all along up to Tana Lake is introduced in to the model. The downstream hydraulic boundary condition is considered as stage hydrograph since impact of the breach compared to Lake Tana is negligible as shown below.

Maximum Reservoir volume 270.00 Mm3

Lake Tana Reservoir area at elevation of 1988 2156 Km2

Increase in height considering the whole water from the reservoir enters to Lake Tana will only be 0.125m even the whole water stored in the dam spill to Lake Tana.

Furthermore, the following table presents the average water surface elevations in Lake Tana during the wet season months of July, August, and September for the sixteen-year period beginning in 1999 and ending in in 2014 (Michael A. Ports, 2016). For the sixteen-year period of record, the average wet season water surface elevation in Lake Tana is 1786.53.

Table 3-2: Lake Tana water surface elevation

Year	July	August	September	Mean

2014	1786.53	1787.22	1787.75	1787.17
2013	1786.19	1787.28	1787.82	1787.09
2012	1786.21	1787.19	1787.69	1787.03
2011	1786.35	1787.13	1787.70	1787.06
2010	1785.30	1786.26	1787.23	1786.26
2009	1785.13	1785.80	1786.43	1785.79
2008	1785.46	1786.47	1787.08	1786.34
2007	1785.76	1786.52	1787.08	1786.45
2006	1785.44	1786.44	1787.26	1786.38
2005	1785.52	1786.10	1786.55	1786.06
2004	1785.43	1786.15	1786.66	1786.08
2003	1785.00	1785.80	1786.63	1785.81
2002	1785.49	1786.10	1786.54	1786.04
2001	1786.03	1787.01	1787.58	1786.87
2000	1786.49	1787.24	1787.72	1787.15
1999	1786.21	1787.05	1787.62	1786.96
Mean	1785.78	1786.61	1787.21	1786.53

The downstream boundary condition of the model environment is taken as the recorded high water level of Tana Lake which is 1786.53 masl; this represents the worst downstream boundary condition.

3.3.6.2 ROUGHNESS COEFFICIENT (MANNING'S 'n')

Manning roughness estimates for the rivers is calculated following procedures outlined in Water Supply Paper 2339 of USGS (U.S department of the interior, 1984). The method as outlined in the study introduces components of the roughness coefficient to integrate specific river properties that are known to affect hydraulic roughness property.

Manning's n compositing as recommended by Cowan (1956) is outlined in the paper, it reads;

$$n = (nb + n1 + n2 + n3 + n4)$$

Where:

nb = a base value of n for a straight, uniform, smooth channel in natural materials

n1 = a correction factor for the effect of surface irregularities

n2 = a value for variations in shape and size of the channel cross section,

n3 = a value for obstructions

n4 = a value for vegetation and flow conditions

m = a correction factor for meandering of the channel

Table below presents the construction of the manning's n estimate for the river modelling.

Table 3-3: Manning Roughness Estimate; estimated after Cowan (1956)

Parameter	Adopted	Remark
Base n, nb	0.025	Typical value for firm bed
Surface Irregularties,n1	0.0025	Minor Irregularity
Variation in cross section,n2	0.000	Uniform minor river bed cross section all along
Flow obstruction,n3	0.0015	Minor flow obstruction
Vegetation effect,n4	0.001	Estimates are relevant after few years after training
Channel meandering, m	1.5	Significantly meandering rivers
Estimated, n	0.045	Estimated after Cowan(1956)

Moreover, in order to consider the mud flow at the immediate downstream of the dam breach, the Manning's n compositing value recommended is 0.06.

4 MODELLING TOOLS, MODELLING APPROACH AND ANALYSIS

4.1 TOOLS

HEC-RAS 5.0.7, a 1D-2D hydraulic modelling tool developed by the Hydrologic engineering Center of the USACE is applied. The model provides option for modelling steady and unsteady flow simulations and introduction of various civil/hydraulic infrastructures existing or planned.

McBreach is also used which is companion software application to the hydrologic engineer's center river analysis system (HEC-RAS) that performs probabilistic dam breach modelling.

Moreover, Global Mapper Version 16 with Google Earth software are used for assessing and editing study area map and for remote sensing of the developed infrastructures like hydraulic structures , Trunk roads and current land use information. In addition, Microsoft EXCEL 2010 was used to analyze HEC-RAS outputs. Moreover, ARC GIS also used to export the map in order to make it in scale.

4.1.1 MODEL ASSUMPTIONS

Assumptions considered for the modelling are:-

- Only overtopping and piping modes of failure considered
- The modeling incorporates only the hydraulic aspects of the flood in which the flood flow is assumed free of excessive debris flows, mud flow and sediment discharge.

4.1.2 MODEL LIMITATION

The limitations of the model are:-

- Estimation of breach parameters is approximate due to inadequacy of existing knowledge to model the breach phenomena. In fact, the modeling does not incorporate aspects of sediment transport and soil mechanics in which the breach is prescribed not computed.
- A simple trapezoidal breach representation
- The flood plain area DEM quality due to continues changes in the channel geometry due to excessive sedimentation
- Uncertainty in the input data, i.e. DEM, Manning's roughness coeffcient and inflow hydrograph data.
- Only 1D and 2D unsteady flow modelling, not 3D or computational fluid dynamics

4.2 MODELLING APPROACH

The modelling approach to conduct this thesis is;

- Dam breach analysis is done to estimate the breach size and the outflow hydrograph.
- Flood routing is done in the downstream channel and flood plain
- Results of hydraulic flow simulation are exported into RAS MAPPER to produce flood inundation maps.

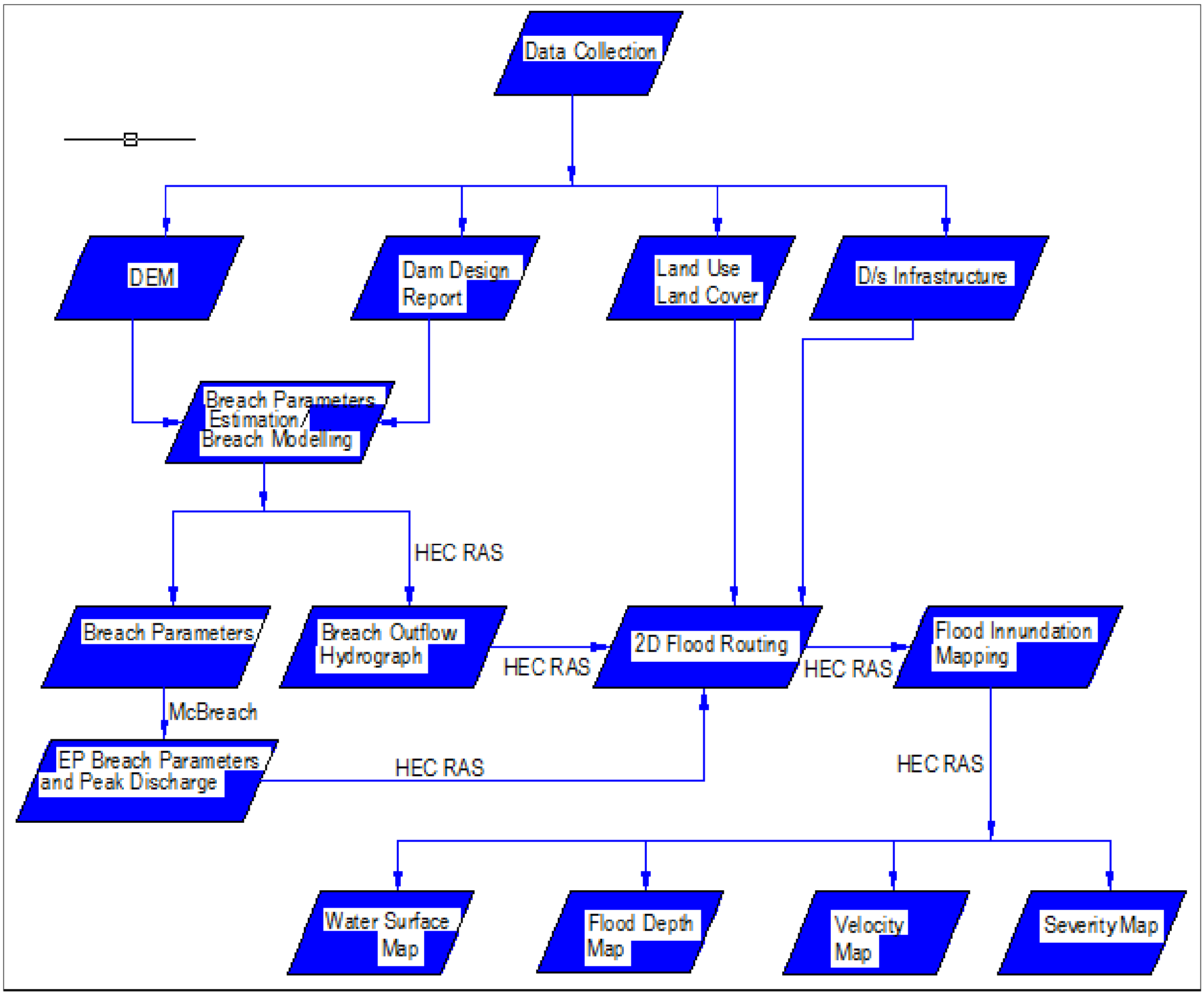


Figure 4-1: shows the conceptual framework of the thesis.

In general, the modelling task can be categorized in to two:-

- Model Schematization
- Internal Data Analysis

4.2.1 MODEL SCHEMATIZATION

4.2.1.1 GEOMETRIC PROCESSING

HEC-RAS presents the possibility of representing and exchange of geo referenced cross-section and alignment graphics in the geometric data editor tool. Moreover the annexed RAS-MAPPER in the current version provides the capability of creating terrain layers from GeoTIFF raster database into a local format HDF. Accordingly using the LiDAR Data, a gridded terrain model is developed and imported for further processing by RAS MAPPER. This resulting terrain layer is a basis for developing 2D mesh cells terrain property.

4.2.1.2 2D MESH GENERATION

HEC-RAS has the ability to represent details of the more detailed under lying terrain into the larger mesh sizes by creating a detailed cell face elevation and area volume capacity curve. Therefore the grid spacing is applicable to limit the computation steps and thus the wider it is the more averaged is the computed water surface slopes. Otherwise all the details of the terrain are applicable in the computation procedure. Therefore an average of 150m by 150m mesh grid on a terrain layer of 12.5m by 12.5m resolution is considered for the current study. Note that the finer the mesh sizes, the computation time and the result file size will be very large. In other words, the model becomes computationally expensive for no or little added advantage in computational stability and accuracy; as such the ability to use larger cell sizes is one of the unique advantages of using HEC RAS 2D.

4.2.1.3 UNSTEADY EQUATION SETS

For the current study, the Diffusive Wave Equation is applied. Two equation sets are applicable in the unsteady solver in HEC-RAS. These are the diffusive wave equation and the full moment equation. Generally the later should give a more accurate result, yet it causes higher numerical instability before it can be made stable. Therefore, as procedure the Hydrologic Engineering Center, USACE, the producer of the model recommends to setup a working model using the Diffusive Wave Equation sets and later to update the result using the full momentum equation. However the Author also noted that significant differences are not observed between results of the two sets of equations where sudden changes in flow and terrain conditions are absent. Likewise, the Diffusive wave equation sets are applied in the current study and the error percentage for the computation is less than 2%.

4.2.1.4 EFFECTS OF TURBULENCE IN THE TWO-DIMENSIONAL FLOW FIELD

HEC-RAS has the option to include the effects of turbulence in the two-dimensional flow field. Turbulence is the transfer of momentum due to the chaotic motion of the water particles as the flow contracts and expands over the surface and around objects (HEC, 2016). Turbulence in

HEC-RAS is modeled as a gradient diffusion process, represented by an eddy viscosity mixing coefficient, Dτ.

The following table presents typical and representative values of eddy viscosity mixing Coefficients that have been found appropriate in numerous practical applications.

Table 4-1: Values of eddy viscosity mixing coefficients

Mixing Intensity	Geometry and Surface	D_T
Minor transversal mixing	Straight Channel, Smooth surface	0.11 to 0.26
Moderate transversal mixing	Gentle meanders, moderate surface irregularities	0.33 to 0.77
Strong transverse mixing	Strong meanders, rough surface	2.0 to 5.0

Moreover, the introduction of turbulence into the project model should provide more accurate simulation results, reduce numerical instability, and further reduce simulation time.

4.2.2 INTERNAL DATA ANALYSIS

Internal data analysis includes the consideration taken and the analyses done for the breach parameter predication and the flood mapping work for both overtopping and piping failure cases of Ribb dam.

4.2.2.1 BREACH PARAMETERS

The following empirical equations and recommendations are used to predict the breach parameters for the hypothetical dam failure. Here are the considerations taken in to account:

- The main dam section is considered for the breach modelling due to its bigger cross section and lowest elevation than others.
- The dam is considered as medium erosion resistant because of appropriate provision of berms, stiff shell material and clay core and also rock ripraps.
- The breach formation growth will continue only in the embankment considering that the natural ground geological characteristics are more resistant for the erosion process than the embankment.

4.2.2.1.1 PREDICATION AND VALIDATION OF BREACH PARAMETERS FOR OVERTOPPING FAILURE CASE

Table below presents the breach parameters value for overtopping failure case in different methods.

Table 4-2: Overtopping case breach width

Method	Breach Bottom Width(m)	Average breach width (Bavg) (m)	Breach Top Width(m)	Average breach width (Bavg) to Dam height Ratio	Fulfill the listed Agency guidelines for average breach width(m)	Requirement comparison between the crest length and Breach top width
Mac Donald and langridge-Monopolis	109	181.5	254	2.50	USACE 1980, FERC, NWS, USACE 2007	<800m
Froehlich (1995a)	191	394	597	5.41		<800m
Froehlich (2008)	141	286	431	3.95	USACE 1980, FERC, NWS, USACE 2007	<800m
Von Thun and Gillette(1990)	200	272.50	345	3.76	USACE 1980, FERC, NWS, USACE 2007	<800m
Xu and Zhng	162	317.15	472	4.38	USACE 1980, FERC, NWS, USACE 2007	<800m

Table 4-3: Overtopping case breach side slope and breach development time

Method	Side Slope (H:V)	Fulfill the listed Agency guidelines for Side Slope (H:V)	Breach development time(hr)	Fulfill the listed Agency guidelines for Breach development time(hr)
Mac Donald and langridge-Mono polis	0.5	USACE 1980, FERC, NWS, USACE 2007	3.65	USACE 1980,USACE 2007
Froehlich (1995a)	1.4		1.66	USACE 1980,USACE 2007
Froehlich (2008)	1	USACE 1980, FERC, NWS, USACE 2007	1.33	USACE 1980,USACE 2007
Von Thun and Gillette(1990)	0.5	USACE 1980, FERC, NWS, USACE 2007	1.70	USACE 1980,USACE 2007
Xu and Zhng	1.07		3.72	USACE 1980,USACE 2007

4.2.2.1.2 PREDICATION AND VALIDATION OF BREACH PARAMETERS FOR PIPING FAILURE CASE

Table below presents the breach parameters value for piping failure case in different methods.

Table 4-4: Piping case breach width

Method	Breach Bottom Width (m)	Average breach width (Bavg) (m)	Breach Top Width (m)	Average breach width (Bavg) to Dam height Ratio	Fulfill the listed Agency guidelines for average breach	Requirement comparison between the crest length and Breach top

					width(m)	width
Mac Donald and langridge-Mono polis	78	114.25	150.50	1.58	USACE 1980, FERC, NWS, USACE 2007	<800m
Froehlich (1995a)	129	194.25	259.50	2.68	USACE 1980, FERC, NWS, USACE 2007	<800m
Froehlich (2008)	102	152.75	203.50	2.11	USACE 1980, FERC, NWS, USACE 2007	<800m
Von Thun and Gillette(1990)	186	222.25	258.50	3.07	USACE 1980, FERC, NWS, USACE 2007	<800m
Xu and Zhng	95	140.68	186.35	1.94	USACE 1980, FERC, NWS, USACE 2007	<800m

Table 4-5: Piping case breach side slope and breach development time

Method	Side Slope (H:V)	Fulfill the listed Agency guidelines for Side Slope (H:V)	Breach development time(hr)	Fulfill the listed Agency guidelines for Breach development time(hr)

Mac Donald and langridge-Mono polis	0.5	USACE 1980, FERC, NWS, USACE 2007	3.31	USACE 1980,USACE 2007
Froehlich (1995a)	0.9	USACE 1980, FERC, NWS, USACE 2007	1.47	USACE 1980,USACE 2007
Froehlich (2008)	0.7	USACE 1980, FERC, NWS, USACE 2007	1.19	USACE 1980,USACE 2007
Von Thun and Gillette(1990)	0.5	USACE 1980, FERC, NWS, USACE 2007	1.59	USACE 1980,USACE 2007
Xu and Zhng	0.63	USACE 1980, FERC, NWS, USACE 2007	3.62	USACE 1980,USACE 2007

Mac Donald and langridge-Monopolis, Froehlich, 2008 and Von Thun and Gillette (1990) equations fulfill the USACE 2007 guidelines. Furthermore, Mac Donald and langridge-Monopolis seems more realistic than other equations as it incorporates the eroded volume from the reservoir volume.

4.2.2.1.3 BREACH PARAMETER SENSITIVITY

Sensitivity analysis on dam breach parameters approach focuses primarily on analyzing the response of the peak discharge upon reduction and increment of time of failure in hours, breach width and other parameters. According to Singh and Snorrason (1984), the peak breach discharge is quite sensitive to breach width than breach development time for a large reservoir dam and the vice versa is true for a small reservoir dam. In this regards, Ribb dam is classified as large dam in which the peak breach discharge is more sensitive to the breach width than breach development time.

However, Petrascheck and Sydler (1984) also demonstrated the sensitivity of discharge, inundation levels, and flood arrival time to the changes of the breach width and breach formation time. For places close to the dam, both parameters can have a significant effect. Whereas, changes in breach formation time can significantly affect only the timing of the flood wave peak for areas far downstream from the dam, while changes in breach parameters have less effect on peak discharge and inundation levels. Hence, evaluation of the relative influences of these two parameters specially, and other parameters on the resulting flood hydrographs shall be carried out as there is no definite equation for accurate prediction. Therefore, the Author proposed to study the effect of breach parameters on maximum breaching outflows and flood hydrographs on all five breach prediction equations even though some are not in the limit specified in the recommended guidelines. A sensitivity analysis of breach parameters and times shall be performed by running all the parameters with in a HEC-RAS model (TD-39, 2014).

4.2.2.1.4 BREACH PARAMETER UNCERTAINTY ANALYSIS

The Monte Carlo technique is used to do the uncertainty analysis in this thesis paper. This will be used to randomly sample the breach parameters in the predefined statistical distribution. Through Monte Carlo simulation by McBreach, the hydraulic modeler will produces exceedance probability peak discharge and their respective sampled breach parameters that can be used to produce probability inundation (EPI) map. The exceedance probability will be conducted on 0.002%, 1%, 5%, 10%, 50%, 90%, 95% and 99% probability and also producing the probability inundation map for 1%and 90%.

Table 4.6 and Table 4.7 show a breach parameters input data and its statistical distribution for overtopping and piping modes of failure in probabilistic approach in which the specific parameters are derived from the result of deterministic approach.

Table 4-6: Breach parameters, sampling mode and distribution for overtopping mode of failure

Breach parameters	Sampling mode	Distribution	Value	Mean Value	Standard deviation Value
Final bottom invert elevation	Deterministic		1973		
Final bottom width	Probabilistic	Normal distribution		153	23.5
Left side slope	Probabilistic	Normal distribution		0.77	0.15

Right side slope	Probabilistic	Normal distribution		0.77	0.15
Breach formation time	Probabilistic	Normal distribution		2.60	0.56
Failure initiation water surface elevation	Deterministic		1945.66		
Breach weir coeffcient	Deterministic		1.45		
Breach progression	Deterministic		Sine		

Table 4-7: Breach parameters, sampling mode and distribution for piping mode of failure

Breach parameters	Sampling mode	Distribution	Value	Mean Value	Standard deviation Value
Final bottom invert elevation	Deterministic		1973		
Final bottom width	Probabilistic	Normal distribution		118.00	34.00
Left side slope	Probabilistic	Normal distribution		0.65	0.13
Right side slope	Probabilistic	Normal distribution		0.65	0.13
Breach formation time	Probabilistic	Normal distribution		2.24	0.54

Failure initiation water surface elevation	Deterministic		1940.00		
Breach weir coeffcient	Deterministic		1.45		
Breach progression	Deterministic		Sine		
Piping coeffcient	Deterministic		0.5		
Initial piping elevation	Probabilistic	Uniform distribution		1873	1940

In most, the normal distribution is used in which it is a symmetric distribution that defines a central tendency. In fact, a normal distribution accurately represents an uncertainty distribution for many of the natural phenomena found in science (McBreach 507 user manual, 2019).However, a uniform distribution is considered for the initial piping elevation as all elevation has equal chance. An envelope of 95.4% confidence interval is considered for conducting the entire anlysis.

HEC RAS dam breach models statistical convergence in a Monte Carlo method run requires between 1000 and 10000 realizations in which one realization means a single HEC RAS simulation. This can be demonstrated with a plot that shows the convergence on the mean and standard deviation of peak dam breach outflow.

4.2.2.2 FAILURE INITIATION CRITERIA

As per HEC-RAS user's manual (HEC, 2016), there are two alternatives to the user as failure initiation criteria. The user can either specify a time at which the dam failure will commence or a certain critical height of water above the crest elevation of dam, which will indicate commencement of dam failure if the elevation of water trapped behind the reservoir, exceeds it. In this study, the option of specifying a critical height of water above the dam crest elevation, is preferred for both modes of failure that is sunny day (piping or non-hydrologic) and overtopping (hydrologic) failure.

For the piping failure case, the failure initiation criteria elevation selected is 1940.0 m, which is equal to Ribb dam's normal water level, as per FEMA Federal guidelines (FEMA, 2013).

For the overtopping failure case, a literature review and needs assessment study conducted by Wahl (Wahl, 2010) makes a quotation from Singh and Snorrason's studies done in 1982 about the critical water surface elevation. They found that the maximum overtopping depth prior to failure ranged from 0.16-0.61 meters (0.5-2 feet). Hence, for the overtopping failure cases, the failure initiation criteria elevation selected is 1945.66 m, which is 16cm higher than Ribb dam crest level as it is conservative option for the safety of the dam.

4.2.2.3 BOUNDARY CONDITIONS

As described in the previous sections, there are two external boundary conditions for the Ribb dam breach analysis. These are upstream and downstream boundary conditions. For unsteady flow models, upstream boundary conditions are typically entered as discharge hydrograph. This is done by establishing an initial reservoir water level and commencing a breach analysis with the PMF for overtopping. However, the dam spillway is designed to accommodate this magnitude of flow and will prevent the overtopping of the dam. Infact, there is uncertainty in the hydrology analysis and a might blockage of the spillway section, in which leads to overtopping of dam crest and hence failure of the dam itself unavoidable. Hence, a 1.4PMF, 1.5 PMF and 1.5 PMF is considered as the inflow hydrograph for the overtopping simulation.

Moreover, the upstream boundary condition for piping is done by creating an initial reservoir water level and initiating a breach analysis with base inflow only.

Furthermore, the major crossing structures located on the Trunk highway are represented as internal model boundary conditions of the model by considering the big cross drainage structures.

Downstream boundary condition is set to be Lake Tana in which the elevation considered will be 1786.53m for both overtopping and piping scenarios entered.

4.2.2.4 INITIAL CONDITIONS

According to the HEC-RAS user's manual (HEC, 2016), the initial conditions include the flow and stage information of the system's defined domain. In this regard, the modeler must establish all the initial conditions of the system at the start of unsteady flow simulation.

According to the HEC-RAS user's manual (HEC, 2016), arbitrary values of initial water surface elevation should be chosen so that their value is between the possible maximum and minimum reservoir operation levels, and an initial flow value equivalent to the modeling reach's base flow hydrograph should be used at each cross-section.

Accordingly, the Author used a reservoir initial water surface elevation value of 1943m for the storage area as the dam break shall be studied for the worst case flood. Moreover, the initial water elevation considered for the 2D flow area is 1786.56m. Furthermore, an initial flow or base flow of 50m3/s is used for inflow hydrograph for both piping and overtopping in order to make the model stable even though the minimum value from the hydrograph is less than the actual value(0.4m3/s). Thereby preventing the retention of critical errors in depth and flow in the vicinity of a rapidly rising wave front such as associated with dam-break waves or any sudden discharge releases from reservoirs. This further prevents occurrence of errors due to calculated water surface elevations lower than the streambed invert elevation. Furthermore, this will prevent the computational errors and increase stability of iterations.

4.2.2.5 FAILURE LOCATION

As there is no seepage and foundation problem known or any likely location for the embankment to breach, the breach will be modeled at the centerline of the downstream main channel.

4.2.2.6 WEIR AND ORIFICE COEFFICIENTS

As Ribb dam has a clay core, it will have much more pronounced head cut process. Moreover, the dam has a relatively low volume of water in comparison to the height of the dam in which peak flow may occur during the phase of the breach by cutting down through the dam. Hence, a weir coefficient of sharp crested weir would be more appropriate in which the value is 1.44.

Furthermore, a value of 0.5 is taken for piping or orifice flow coeffcient as it will not be free flow.

4.2.2.7 BREACH PROGRESSIONS

A detailed review of breach progression was previously made under section 2.6.2.The Author selected sine wave breach progression for both overtopping and piping failure cases.

4.2.2.8 ROUTING METHODS

A draw down number criteria is used to decide in selection of reservoir routing methods. The estimated Compaction factor is:

Fc=H/L Where, H = 72.5m,

L= 6937.5m. Hence, Fc=0.0105

The estimated translation factor is:

Fc=ct/L Where, c = 72.5m, in which g=9.81m/sec^2 and D=36.25m

t= Minimum 1.33 to maximum 3.71hr. Equivalent 4788-13356sec, taken from the breach predication.

L= 6937.5m. ,

Hence, Fc=13.02 minimum and 36.31 maximum

Hence, the Drawdown number, Dn=0.14 minimum and 0.38 maximum

By taking the maximum and the minimum value for whole breach parameters, the percentage error due to level pool reservoir routing will be 14% and 6% respectively.

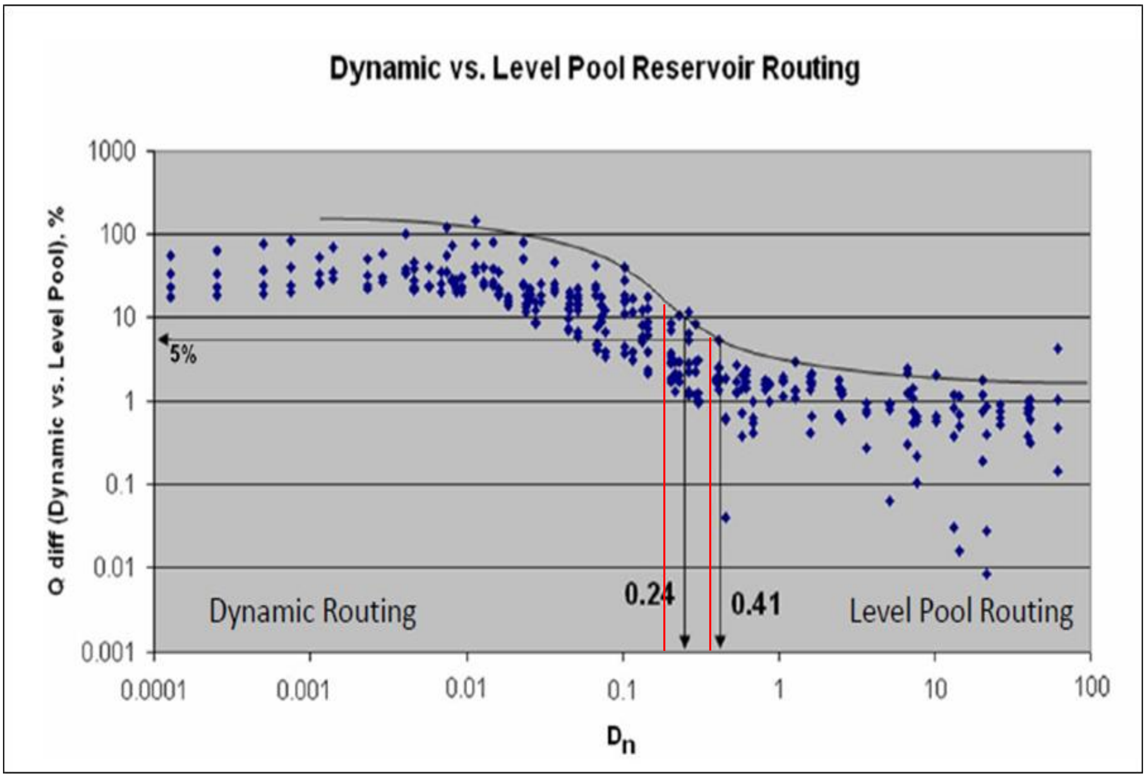


Figure 4-2: Dynamic vs. Level pool reservoir routing discharge diffrence

As the Ribb Dam reservoir is short and compact and the breach will be with slow erosion, a level pool routing is appropriate to be used as routing method in which the combined error will be less than 10% error.

In level pool routing, an elevation-volume curve as part of the storage-area data describing the reservoir is used.

4.2.2.9 ROUGHNESS COEFFICIENT (MANNING'S 'n') SENSITIVITY ANALYSIS

The model has not been calibrated since appropriate measured data does not exist for the watershed. The calibration of the model is not possible, nor practical and possible to attempt to collect all sufficient measured data. In fact, most public infrastructure, as well as much private infrastructure, throughout the world are planned and designed using un-calibrated and un-verified hydrologic and hydraulic computer models. When faced with the lack of sufficient calibration data, modelers should take special care to estimate input parameters as accurately as is reasonably practical. Then, sensitivity Analysis will also be conducted.

It's not easy to assign a definite representative friction slope value even for small areas especially when it comes to natural waterways, where a diverse range of land use and land cover types can be found within short stretch of the boundary. Hence, the other kind of sensitivity Analysis used in conducting a given dam breach analysis is roughness coefficient sensitivity analysis. Sensitivity analysis will determine how the target variables are affected based on changes in other variables or parameters. HEC-RAS model will perform what if or simulation analysis and predict the outcome of a decision given a certain range of variables. These are sensitivity analysis on roughness manning's coeffcient (n) on breach out flow hydrographs based on the percentage reduction and increment of manning's n. The sensitivity analysis was performed by globally adjusting and comparing the baseline values of Manning's n from ± 10% to ± 30%.The author took the ± 30% margin as it is conservative for a specific dam breach parameter.

4.2.2.10 EDDY VISCOSITY MIXING COEFFICIENT

Considering the high discharge value of the flood and the higher manning coeffcient considered, taking in to account the sinuosity, the value of eddy viscosity mixing coeffcient "2" is considered in the simulation.

4.2.2.11 COMPUTATIONAL TIME STEP

The guideline for computational time step is to divide the time of hydrograph rise by 20

Therefore:

$\Delta t < Tr/20 = 60/20$ $\Delta t < 5min$

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:

$$Cr = (Vw\ \Delta t)/L \leq 1.00$$

Therefore the Computational time steps will be

$$\Delta t \leq (\Delta x/Vw)$$

Where, Vw= Flood wave speed, Taking the maximum velocity 13.51m/s

Cr=Courant Number, A value =1.0 is optimal.

Δx=Distance between cross sections, average of 150m by 150m mesh grid on a terrain layer, Take 150m

Hence, $\Delta t = 150/13.51$ $\Delta t = 11.10$

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

4.2.2.12 THETA WEIGHTING FACTOR

When solving the unsteady flow equations, theta weighting factor is one of the weighting factors that is applied to the finite difference approximations. Theoretically Theta can vary from 0.5 to 1.0.However, a practical limit is from 0.6 to 1.0.Theta of 1.0 provides more stability, but less numerical stability (HEC, 2016).Hence, a Theta value of 1.0 is considered in order to keep the model stable.

4.2.2.13 COMPUTATIONAL TOLERANCE

The study assumes a numerical computational tolerance of 0.003 meters for both water surface elevations and for water volumes in each model element and a maximum number of twenty numerical iterations per calculation until computational conversion.

4.2.3 MODELLING SCENARIO DEVELOPMENT

HEC-RAS software can model a dam breach events in a variety of circumstances. The objective of this modeling effort is to evaluate a different catastrophic event, predict the outflow hydrograph just at the outlet of breach for different breach parameter results and to route the peak outflow hydrograph on downstream channel and map the area that will be flooded.

All Dam failure scenarios were analyzed for piping and overtopping (hydrologic and non-hydrologic events) and then with 2D modelling for the routing. These are the breach parameter results from five types of methods namely; Mac Donald and langridge-Monopolis, Frohelich (1995a), Frohelich (2008), Von thun and Gillete and Xu and Zhang in which ten simulation is required for both modes of failure. Moreover, the exceedance probability inundation mapping will also be conducted on 1% and 90% probability for overtopping failure in which two simulation is required. Moreover, two simulations will be performed for sensitivity analysis of manning roughness coeffcient .Hence, in total fourteen simulations will be performed. All simulations shall run of the project models produce no instability errors or warnings and have total mass balance continuity errors less than two percentage.

5 RESULTS AND DISCUSSION

5.1 GENERAL DESCRIPTION

In order to assess the potential risk of Ribb dam breach to the downstream region, the modelling domain was tested for fourteen set of scenarios under overtopping and piping modes of failures. Hence, after completion of Ribb dam breach simulation in HEC-RAS model for each defined scenarios, the following outputs were acquired:

- Discharge hydrographs
- Longitudinal profile showing maximum water surface depth and flow velocity
- Graphical cross section showing maximum water surface elevation, water depth and width, and velocity.
- Time when maximum flow and water surface elevation will occur
- Peak discharge for different exceedance probability breach parameters.
- Flood extent and severity for all scenerios including the specified exceedance probability breach parameters.

Even if the modelling result can be accessed for any given point within the model boundary, simulation cross section results are given and discussed only at key river stations so as to provide concise discussion and interpretation. All distances are measured from dam centreline location.

All simulation results percentage error is less than 2% for mass balance/continuity.

5.2 OVERTOPPING MODE OF FAILURE SIMULATION RESULTS, RESULTS DISCUSSION AND VERIFICATION

As per the overtopping failure mode simulation, the PMF inflow hydrograph could be safely evacuated through the spillway without causing a breach of the dam. Thus, the dam is safe against failure triggered by overtopping for the inflow flood ranging up until 1.5PMF as shown in the table 5.1. In addition, Table 5.2 shows 1.5 PMF event dam breach peak outflows and time to peak values for overtopping mode of failure.

Table 5-1: Different value inflow hydrograph and breach status

Multiplier Factor of PMF	Maximum Head Water Stage	Status
1.45	1945.58	No Breach
1.50	1945.66	Breach

1.55	1945.70	Breach

In all cases, it is assumed that the catchment received a 1.5 PMF flood event starting at 24:00 GMT on 19/9/2020. The breach model results showed exaggerated difference when compared with one another. As per the comparisons’, MacDonald and langridge-Monopolis and Frohelich (1995a) equation result in the minimum and maximum computed peak flows respectively with a percentage difference of 40.3%.

Table 5-2: Overtopping failure case breach peak outflow and time to peak

Methods	Peak flow (m3/s)	Time to peak (hr)
MacDonald and langridge-Monopolis	67,570	20 Sep 2020,20:00
Frohelich (1995)	113,153	20 Sep 2020,18:30
Frohelich (2008)	104,617	20 Sep 2020,18:30
Von thun and Gillete	97,982	20 Sep 2020,18:45
Xu and Zhang	82,601	20 Sep 2020,19:30

5.2.1 OVERTOPPING MODE OF FAILURE SIMULATION RESULTS

5.2.1.1 MAC DONALD AND LANGRIDGE-MONOPOLIS

Table 5.3 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Mac Donald and langridge-Monopolis equation.

Table 5-3: Mac Donald and Langridge Monopolis equation breach peak flow, maximum velocity, Maximum flood depth and time to peak

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth(m)	Time to peak (Day, hr)

Dam	0.26	67,570	10.89	27.62	20Sep 2020,20:00
Pick up weir	29.97	32,526	5.40	16.73	20Sep 2020,21:30
Trunk road from Bahir dar to Gonder	50.98	17,854	3.5	4.67	20Sep 2020,22:45
Lake Tana	75.38	8,954	1.28	9.02*	21Sep 2020,03:15

*Depression area

As shown in the above table, peak flow, flood depth and velocity is decreasing starting from the dam location towards to the Lake Tana entry. Between the dam and Lake Tana the flood attenuated by 87%, resulting in wide spread inundation and showing flood storage capacity of the downstream flat plains. According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 17:48 GMT on 20/9/2020 and 03:15GMT on 21/9/2020 respectively. Thus, the whole flood attenuation in the downstream regions has taken around 9:27 hour in which the travel time is slower than result obtained using the other methods.

Figure 5-1 shows breach outflow hydrographs at selected locations for overtopping by Mac Donald and langridge-Monopolis equation. As shown in the figure above, as the hydrographs moves downstream, the hydrograph will attenuate.

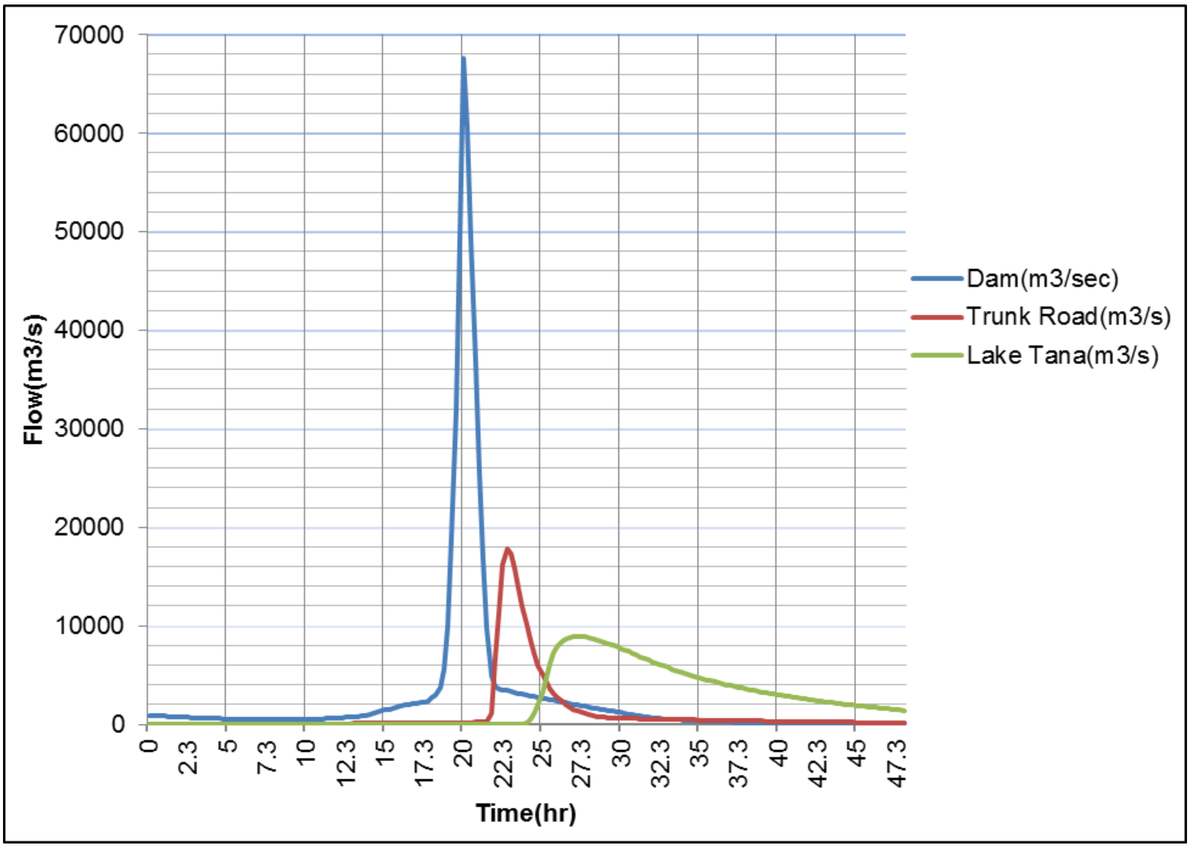


Figure 5-1: Flood hydrograph done by Mac Donald and Langridge Monopolis equation

5.2.1.2 FROHELICH (1995a)

Table 5.4 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Frohelich (1995a) method.

Among the overtopping results Frohelich (1995a) has resulted in the maximum peak flow of, which is 113,153 m^3/s and flood depth of 34 m as shown in the table below.

Table 5-4: Frohelich (1995a) method breach peak flow, maximum velocity, Maximum flood depth and time to peak

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth(m)	Time to peak (Day, hr)
Dam	0.26	113,153	13.51	33.74	20 Sep 2020,18:30
Pick up weir	29.97	68,543	6.41	17.96	20 Sep 2020,19:45
Trunk road from Bahir dar to Gonder	50.98	18,947	3.51	4.75	20 Sep 2020,21:15

Lake Tana	75.38	8,308	1.24	9.03*	21 Sep 2020,01:45

*Depression area

This method's results (i.e. peak flow, velocity and flood depth) have same rising and falling patterns as other methods but with different magnitudes. According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 17:48GMT on 20/9/2020 and 01:45GMT on 21/9/2020 respectively. The whole flood attenuation in the downstream regions has taken around 7:45 hours as shown in the figure below. This is the fastest flood wave advance when compared with the other simulation result.

Figure 5.2 shows flow hydrographs at selected locations, for overtopping by Frohelich 1995a method. Moreover, as shown in the figure below, as the hydrographs moves downstream, the hydrograph will attenuate and converges more quickly than other equations.

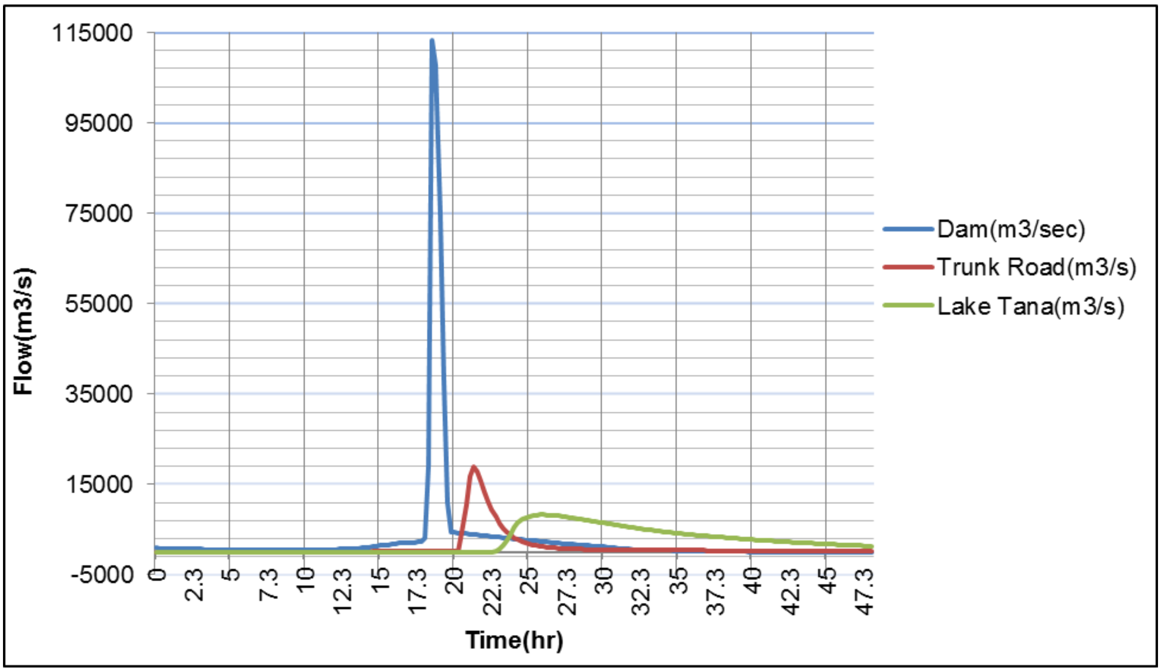


Figure 5-2: Maximum flow hydrographs at selected locations, overtopping by Frohelich 1995a method.

5.2.1.3 FROHELICH (2008)

Table 5.5 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Frohelich (2008) method.

Table 5-5: Frohelich (2008) method breach peak flow, maximum velocity, Maximum flood depth and time to peak

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth (m)	Time to peak (Day, hr)
Dam	0.26	104617.10	12.59	32.60	20Sep 202018:30
Pick up weir	29.97	67345.67	6.23	17.71	20Sep 2020,20:00
Trunk road from Addis Ababa to Gonder	50.98	18499.78	3.52	4.73	20Sep 2020,21:15
Lake Tana	75.38	8362.23	1.28	9.02*	21Sep 2020,02:00

*Depression area

According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 17:48GMT on 20/9/2020 and 02:00GMT on 21/9/2020 respectively. The whole flood attenuation in the downstream regions has taken around 8 hours. The pattern for the rise and fall is similar to Frohelich (1995a)

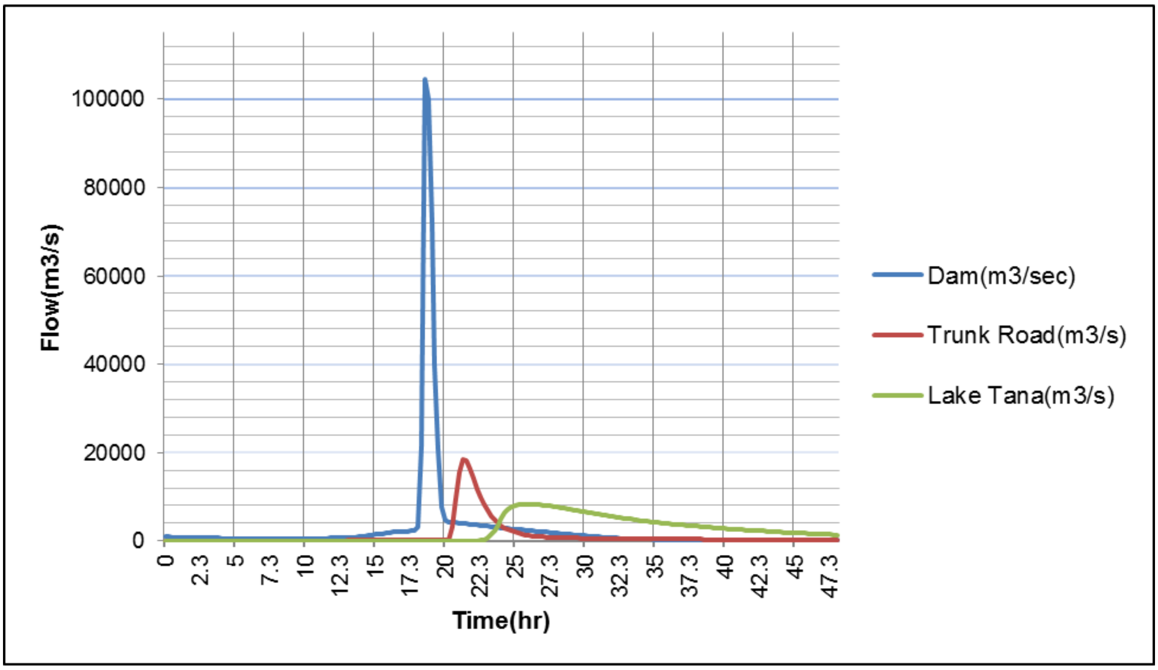


Figure 5-3: Maximum flow hydrographs at selected locations, overtopping by Frohelich 2008 method.

5.2.1.4 VON THUN AND GILLETE

Table 5.6 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Von thun and Gillete method.

Table 5-6: Von thun and Gillete method breach peak flow, maximum velocity, maximum flood depth and time to peak.

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth(m)	Time to peak (Day, hr)
Dam	0.26	97982.24	12.73	32.07	20Sep 2020,18:45
Pick up weir	29.97	49256.35	6.21	11.93	20Sep 2020,20:05
Trunk road from Addis Ababa to Gonder	50.98	18712.82	3.52	4.72	20Sep 2020,21:30
Lake Tana	75.38	8384.41	1.24	9.01*	21Sep 2020,02:00

*Depression area

According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 17:48GMT on 20/9/2020 and 02:00GMT on 21/9/2020 respectively. The whole flood attenuation in the downstream regions has taken around 8:15 hours as shown in the figure below. Figure 5-4 shows the maximum flow hydrographs at selected locations for overtopping case by Von thun and Gillete method.

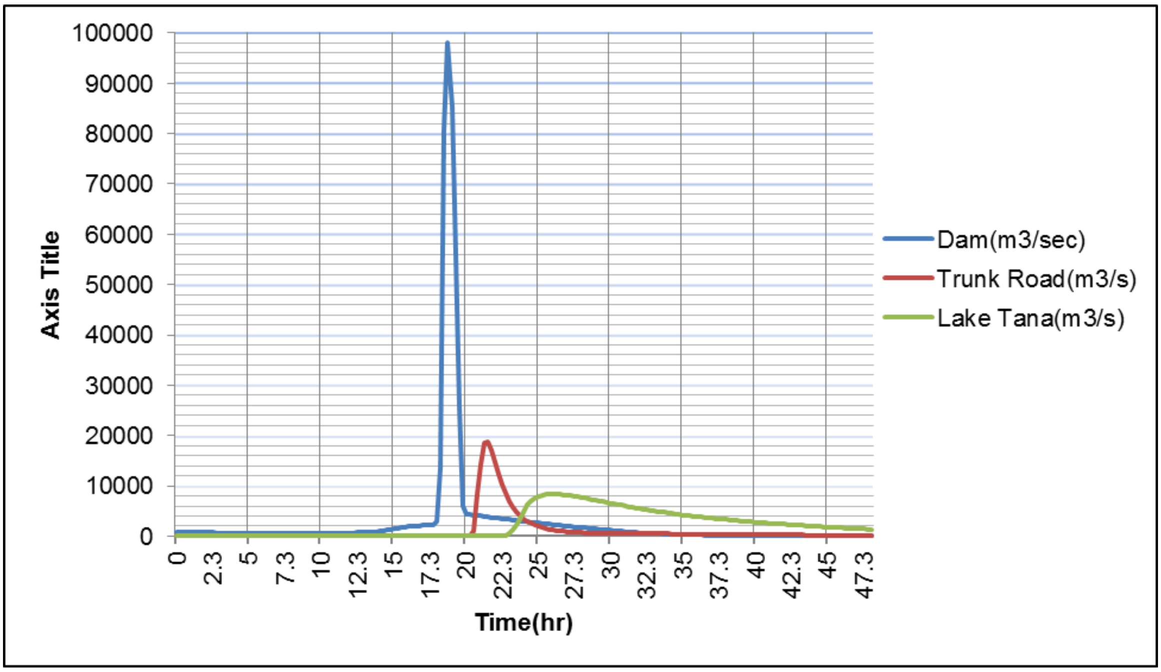


Figure 5-4: Maximum flow hydrographs at selected locations, overtopping by Von Thun and Gillette method.

5.2.1.5 XU AND ZHANG

Table 5.7 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Xu and Zhang method.

According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 17:48GMT on 20/9/2020 and 02:00GMT on 21/9/2020 respectively. Figure 5.5 shows the maximum flow hydrographs at selected locations for overtopping case by Von thun and Gillete method. The whole flood attenuation in the downstream regions has taken around 8:15 hours as shown in the figure below.

Table 5-7: Xu and Zhang method breach peak flow, maximum velocity, Maximum flood depth and time to peak.

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth(m)	Time to peak (Day, hr)
Dam	0.26	82601.52	12.73	32.07	20Sep 2020,19:30
Pick up weir	29.97	44563.21	6.21	11.93	20Sep 2020,20:05

Trunk road from Addis Ababa to Gonder	50.98	18712.82	3.52	4.72	20Sep 2020,21:30
Lake Tana	75.38	8384.41	1.24	9.01*	21Sep 2020,02:00

*Depression area

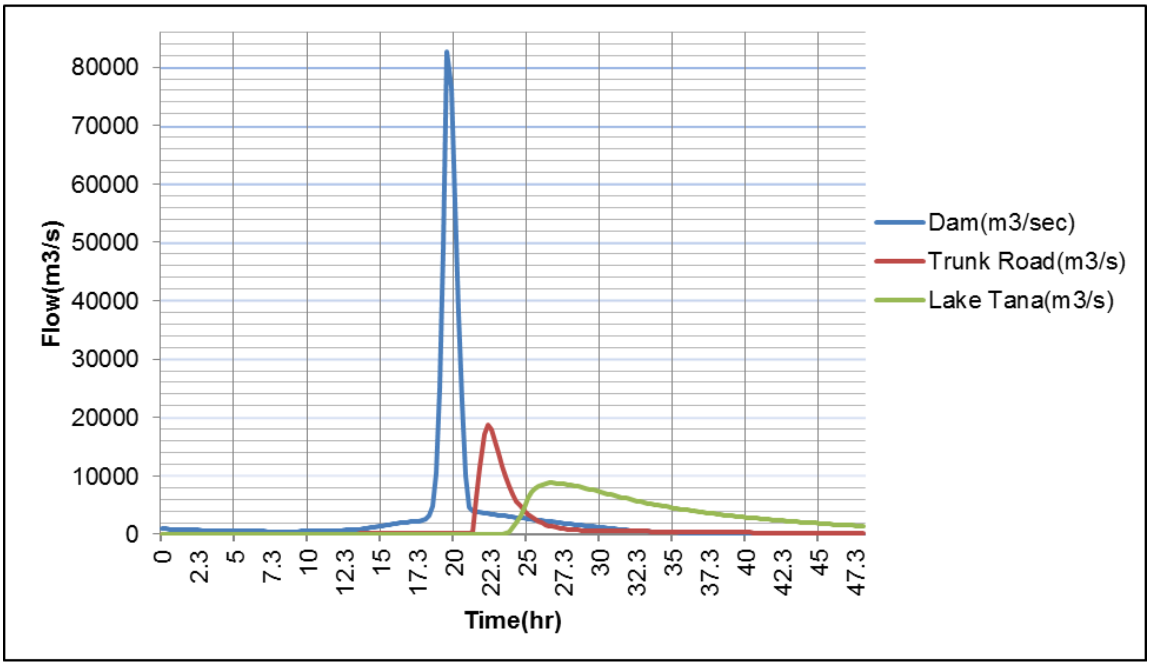


Figure 5-5: Maximum flow hydrographs at selected locations, overtopping by Xu and Zhang method.

5.2.2 DAM BREACH RESULTS COMPARISON

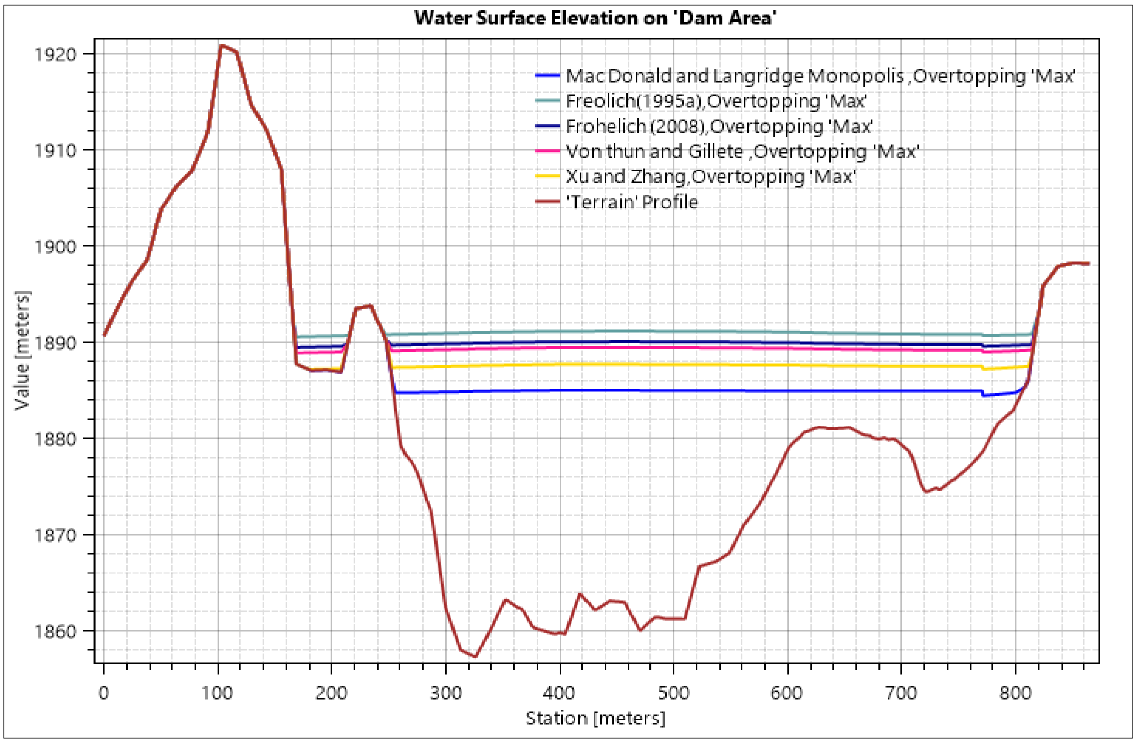


Figure 5-6 Dam area cross section showing water surface elevation for different methods

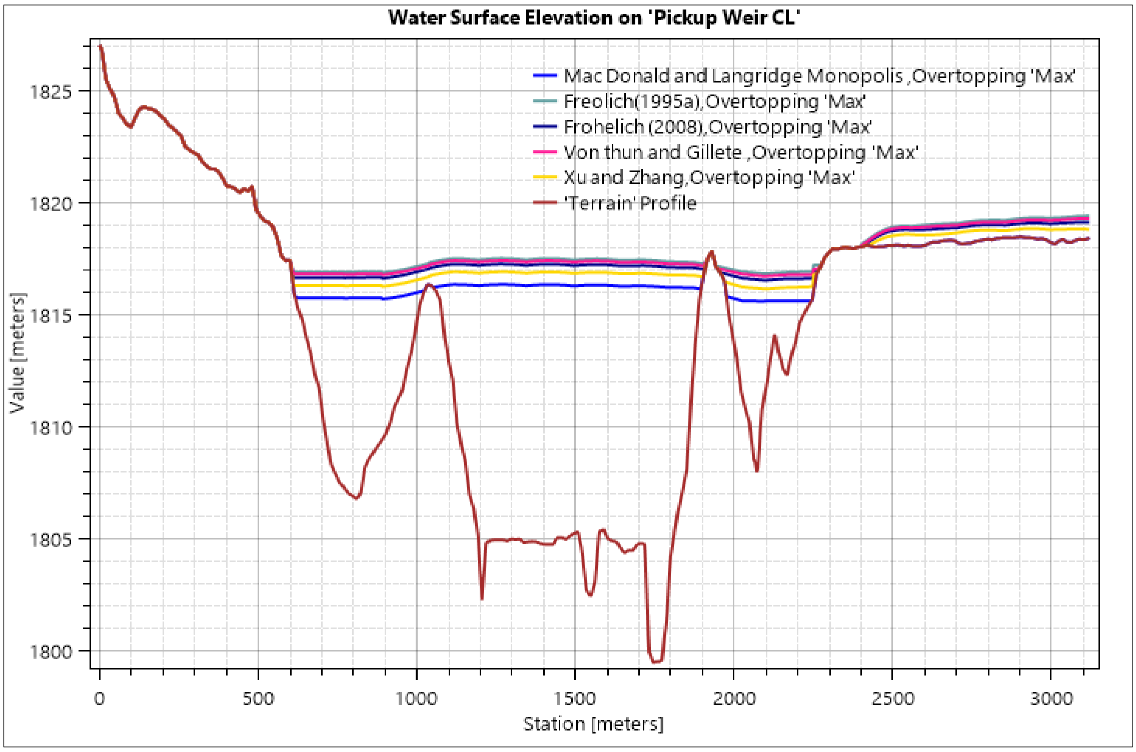


Figure 5-7 Pickup weir area cross section showing water surface elevation for different methods

As shown in Figure 5.6 and 5.7, the water surface elevation and depth for different breach parameters at selected stations is presented in one graph. The selected stations are at a distance of 0.26km and 29.97km from the dam. It is clear that all the graphs follow same trend for different breach parameters along the reaches. In addition, the flow depth difference between different breach equations decrease towards to Lake tana as shown in Figure 5.6 and 5.7 and Table 5.3 to 5.7 and this shows clearly the attenuation and convergance of the flood at the floodplain area.

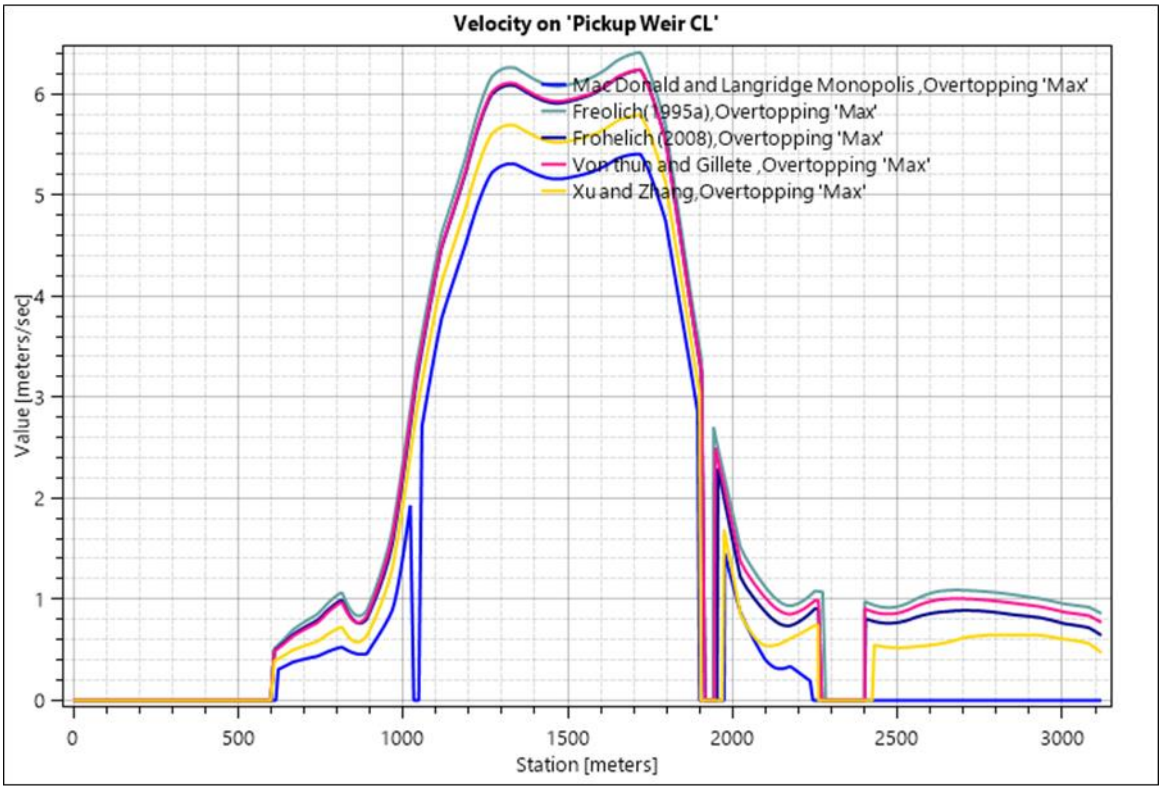


Figure 5-8: Dam area velocity for different methods

The velocity drawing at the pickup weir area is shown in Figure 5.8. The velocity pattern for different breach parameter equation has the same trend. The peak velocity of the flood is experienced at the dam area. This is explained by the fact that the water behind the dam prior to failure is under high hydrostatic pressure and then followed by confined and steeper section than the flood plain area.

As shown below, each set of breach parameters and failure times produce a different outflow hydrograph. Moreover, most of the hydrographs are sharp hydrograph in which tends to attenuate much more quickly. In addition, all the hydrographs are routed downstream and tend to converge towards each other. As the hydrograph move downstream, it substantially converges and the peak flow attenuates significantly around the river station 29.97 km which is the start of the floodplain.

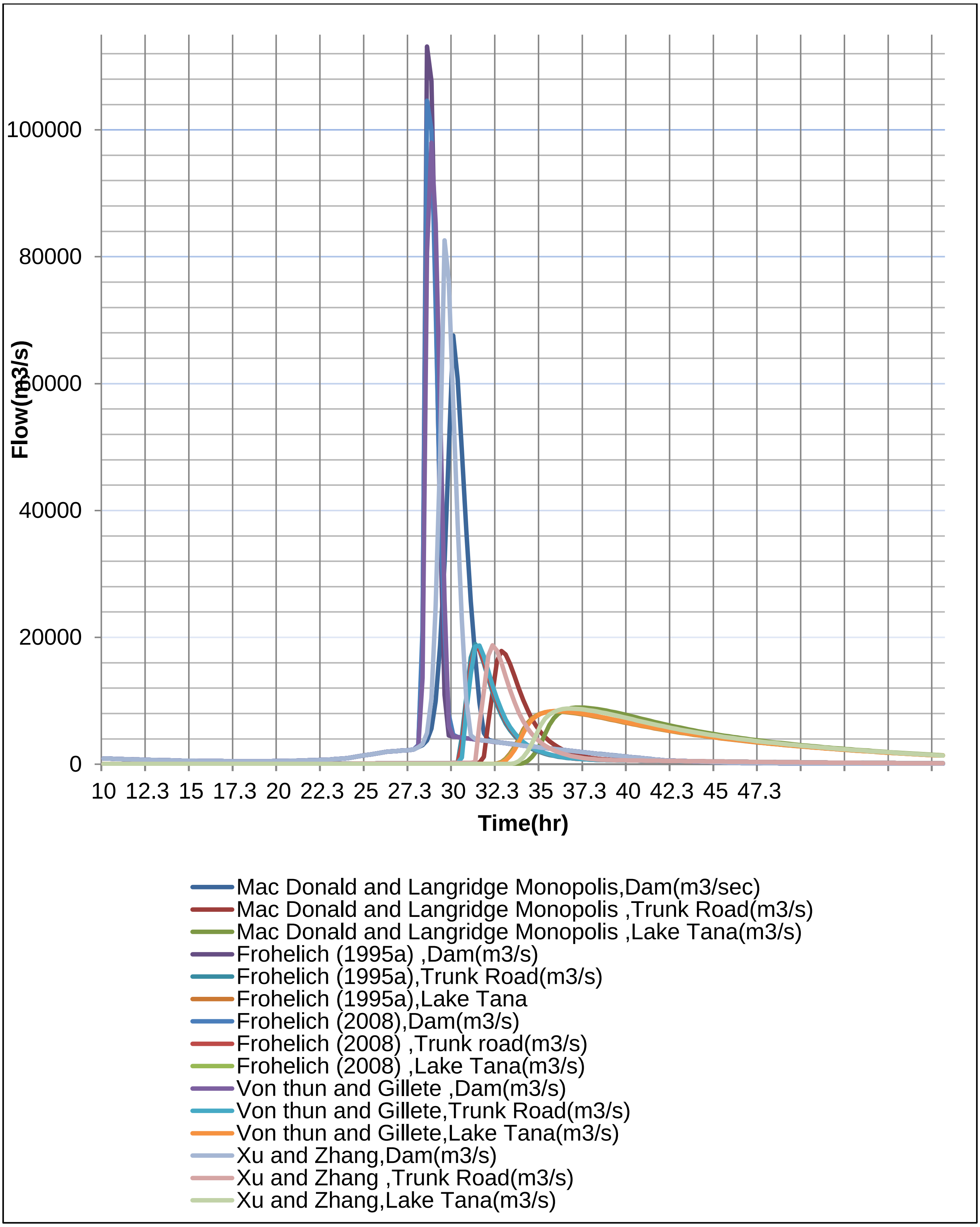


Figure 5-9: Flow hydrographs at selected locations for different breach parameters.

5.2.3 OVERTOPPING BREACH OUTFLOW VERIFICATION

The computed peak outflow from the model compared to the regression equations and the envelope curves of historic failures as test of reasonableness is shown below.

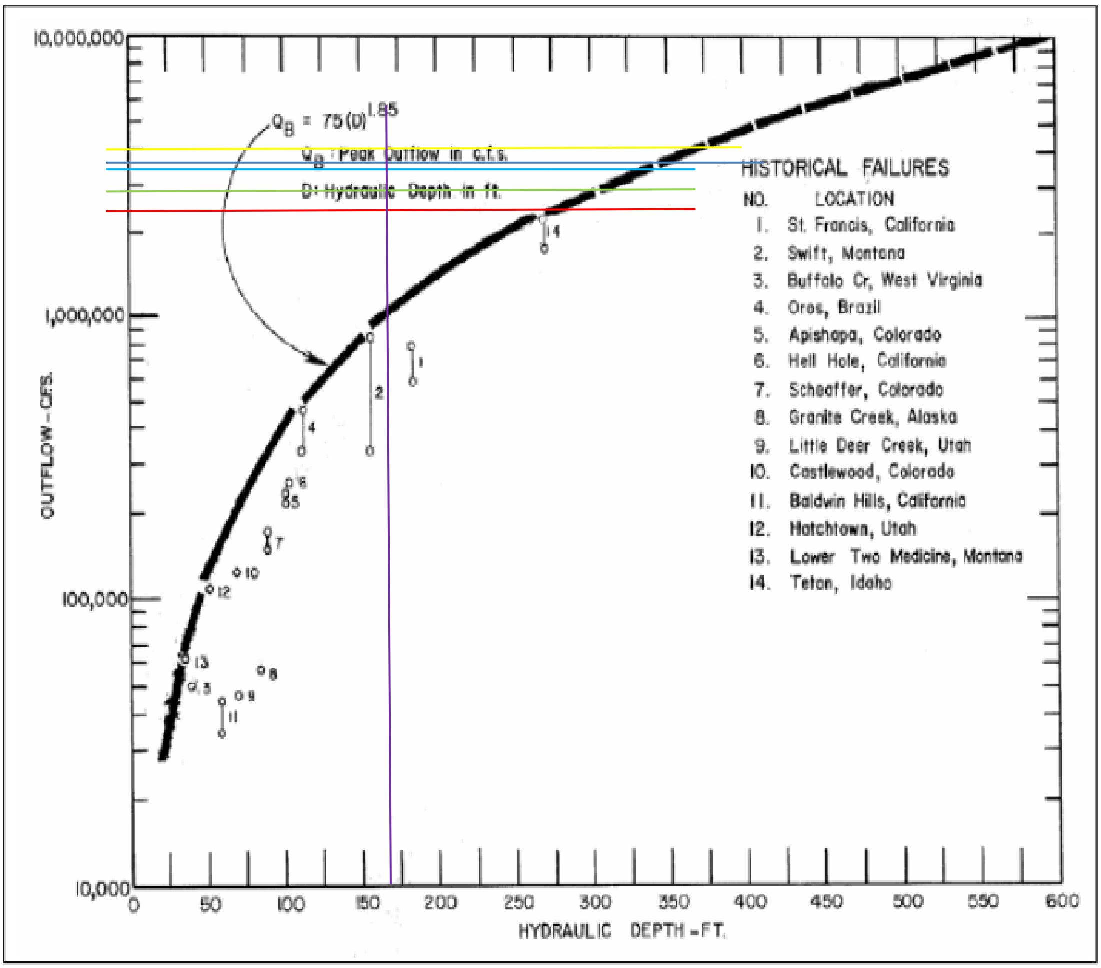


Figure 5-10: Verification of piping outflows using historic outflow rates envelope

Mac Donald and Langridge Monopolis method 67,570m3/s or 2,386,213ft3/s

Frohelich (1995a) method 113,154m3/s or 3,995,986ft3/s

Frohelich (2008) method 104,617m3/s or 3,694,518 ft3/s

Von thun and Gillete method 97,982m3/s or 3,460,210ft3/s

Xu and Zhang method 82,602m3/s or 2,917,045ft3/s

Hydraulic depth 179.86ft

A detail assessment of the graph reveals that Mac Donald and Langridge Monopolis method has resulted in a better result with a difference of 53.3%. Whereas Frohelich's (1995a), Frohelich’s (2008), Von thun and Gillete's and Xu and Zhang's method are found to have 72.13%, 69.86%, 67.82% and 61.83% difference respectively. In fact, the equation and the envelope are developed from limited data sets in which most of the dams considered were small dams. Moreover, the model is conducted by considering the PMF value for the overtopping scenario in which the equation and envelope doesn’t consider. Hence, the author accepted the model result as indicative only.

Moreover, the peak discharge from the HEC RAS run results and peak discharge equation are summarized as below.

Table 5-8: 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 (%)
Mac Donald and Langridge Monopolis	67570.02	67930.03	-360.10	-0.53
Frohelich (2008)	104617.10	38837.85	65779.25	62.88
Xu and Zhang	82601.52	105184.56	22583.04	-27.34

Besides, the Frohelich (1995a) breach parameters are outside the recommended government agency ranges as specified in subsection 3.4.2.1.1.

In addition, another check for the reasonableness is done by reviewing the flow rate and velocities going through the breach in comparison with breach development time and size.

Table 5-9: HEC-RAS results and Equation breach development time comparison.

Methods	HEC RAS Dam breach time (hr)	HEC RAS Minimum Tail water time	Time Difference (hr)	Breach development time from the equation (hr)	Difference (hr)	Percentage

		(hr)				
MacDonald and langridge-Monopolis	17:48	21:45	3.95	3.65	0.30	7.59%
Frohelich (1995)	17:48	19:45	1.95	1.66	0.29	14.9%
Frohelich (2008)	17:48	20:00	2.20	1.33	0.63	27.04%
Von thun and Gillete	17:48	20:00	2.20	1.70	0.50	22.70%
Xu and Zhang	17:48	20:00	3.20	3.72	0.50	15.62%

As stated in the above finding in addition to the previous findings, MacDonald and langridge-Monopolis has smaller difference in which the flow rate and velocities fit with the breach development time and size and then followed by Von thun and Gillete method.

Thus, In accordance with all validation and reasonableness work, MacDonald and langridge-Monopolis equation result seems the most likely and followed by Frohelich (2008) equation. However, for the sake of huge uncertainty in the parameters, the Author took the second bigger value which is Frohelich (2008) method by discarding Frohelich (1995a) in which the breach parameters are outside the recommended government agency ranges.

5.3 PIPING FAILURE SIMULATION RESULTS, DISCUSSION AND VERIFICATION

Piping failure mode is simulated by taking a constant inflow, the base inflow. Table 5.10 shows the dam breach peak outflows and time to peak for piping (Sunny day) failure mode.

In all cases, it is assumed that the catchment received a base inflow starting at 24:00 GMT on 19/9/2020. The breach model results exhibit exaggerated difference when compared with one other. As per the comparison, MacDonald and langridge-Monopolis and Von thun and Gillete equation result with the minimum and the maximum peak flow respectively with a 45.6 percentage difference.

Table 5-10: Piping failure breach peak outflow and time to peak.

Methods	Peak flow (m3/s)	Time to peak (hr)
MacDonald and langridge-Monopolis	22,269	20 Sep 2020,02:45
Frohelich (1995)	35,297	20 Sep 2020,01:30
Frohelich (2008)	31,057	20 Sep 2020,01:15
Von thun and Gillete	40,926	20 Sep 2020,01:30
Xu and Zhang	22,583	20 Sep 2020,03:15

5.3.1 PIPING FAILURE SIMULATION RESULTS

5.3.1.1 MAC DONALD AND LANGRIDGE-MONOPOLIS

Table 5.11 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Mac Donald and langridge-Monopolis method.

Table 5-11: Mac Donald and Langridge Monopolis equation breach peak flow, maximum velocity, Maximum flood depth and time to peak

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth(m)	Time to peak (Day, hr)
Dam	0.26	22,269	4.48	17.70	20Sep 2020,02:45
Pick up weir	29.97	15,269	3.46	11.78	20Sep 2020,04:35
Trunk road from Bahir dar to Gonder	50.98	4,293	3.25	3.42	20Sep 2020,07:30
Lake Tana	75.38	3,353	0.82	8.98*	20Sep 2020,15:00

*Depression area

Like overtopping failure mode, the peak flow, flood depth and velocity are in decreasing pattern from the dam location towards to the Lake Tana entry. According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 24:00 GMT on 19/9/2020 and 15:00GMT on 20/9/2020 respectively. The whole flood attenuation in the downstream regions has taken around 15:00hour in which the travel time is slower than all other equations. Figure 5.11 shows breach outflow hydrographs at selected locations for piping failure by Mac Donald and langridge-Monopolis equation.

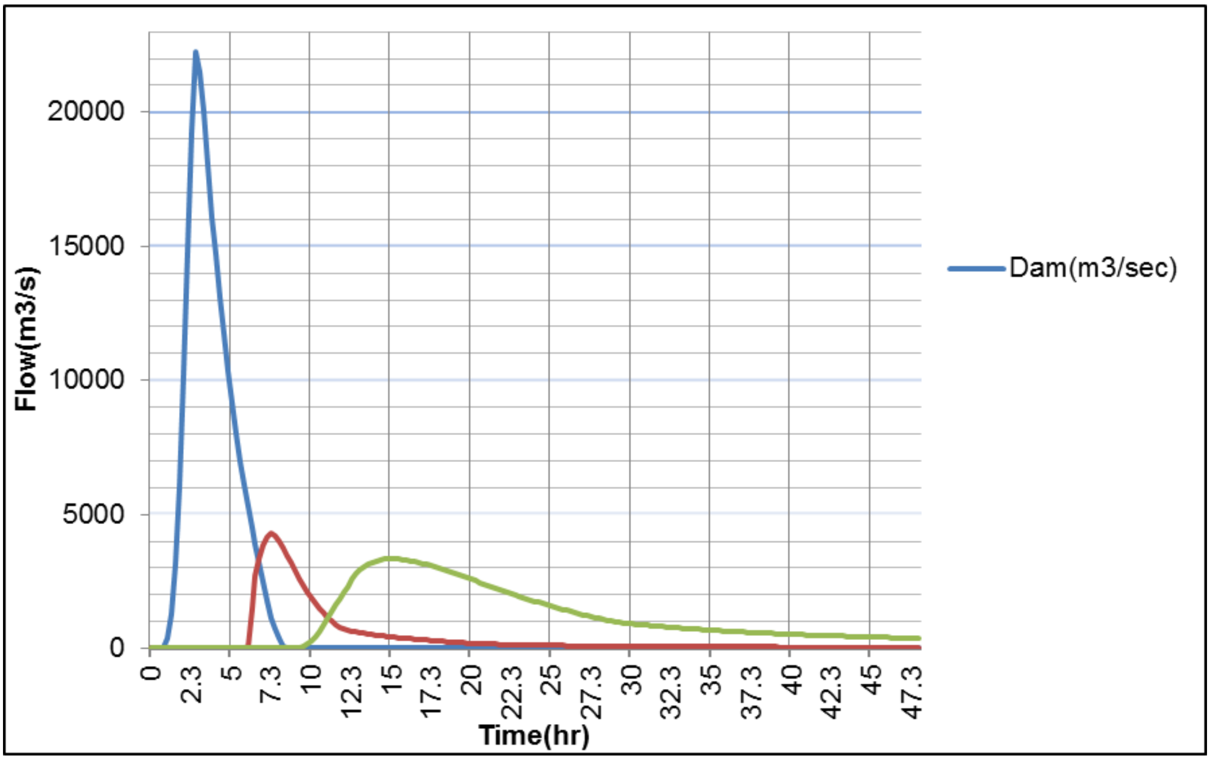


Figure 5-11: Maximum flow hydrographs at selected locations, piping by Mac Donald and langridge-Monopolis method.

As the flow continues to the downstream, the hydrograph attenuate and converges more quickly.

5.3.1.2 FROHELICH (1995a)

Table 5.12 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Frohelich (1995a) method.

Table 5-12: Frohelich (1995a) method breach peak flow, maximum velocity, Maximum flood depth and time to peak

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth(m)	Time to peak (Day, hr)
Dam	0.26	35,297	6.09	21.76	20 Sep 2020,01:30
Pick up weir	29.97	24,115	4.07	13.06	20 Sep 2020,03:30
Trunk road from Bahir dar to Gonder	50.98	5,856	3.33	3.67	20 Sep 2020,05:30
Lake Tana	75.38	3,514	0.84	9.00*	20 Sep 2020,12:15

*Depression area

According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 24:00GMT on 19/9/2020 and 12:15GMT on 20/9/2020 respectively. The whole flood attenuation in the downstream regions has taken around 12:15 hours as shown in the figure below, which is the fastest compared to the other simulation. As per Figure 5.12, the method resulted in maximum flow hydrographs compared to the other breach parameters equation. However, the flood peak attenuates and converges more quickly.

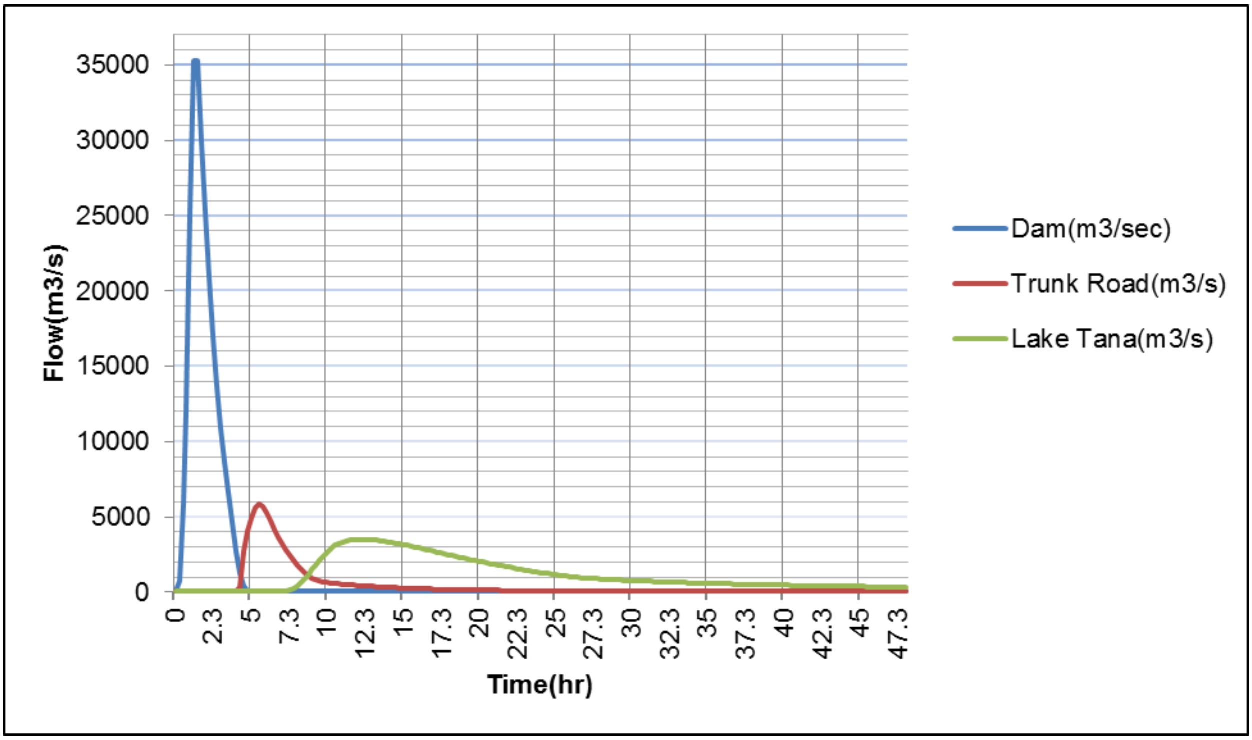


Figure 5-12: Maximum flow hydrographs at selected locations, piping by Frohelich 1995a method.

5.3.1.3 FROHELICH (2008)

Table 5.13 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Frohelich (2008) method.

Table 5-13: Frohelich (2008) method breach peak flow, maximum velocity, Maximum flood depth and time to peak

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth (m)	Time to peak (Day, hr)
Dam	0.26	31,057	5.54	20.41	20Sep 2020 01:15
Pick up weir	29.97	21,856	3.83	12.51	20Sep 2020,03:15
Trunk road from Addis Ababa to Gonder	50.98	5,154	3.30	3.57	20Sep 2020,05:45
Lake Tana	75.38	3,463	0.83	8.98*	20Sep 2020,12:45

*Depression area

According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 24:00GMT on 19/9/2020 and 12:45GMT on 20/9/2020 respectively. The whole flood attenuation in the downstream regions has taken around 12:45hours. The pattern for the rise and fall is similar to Frohelich (1995a)

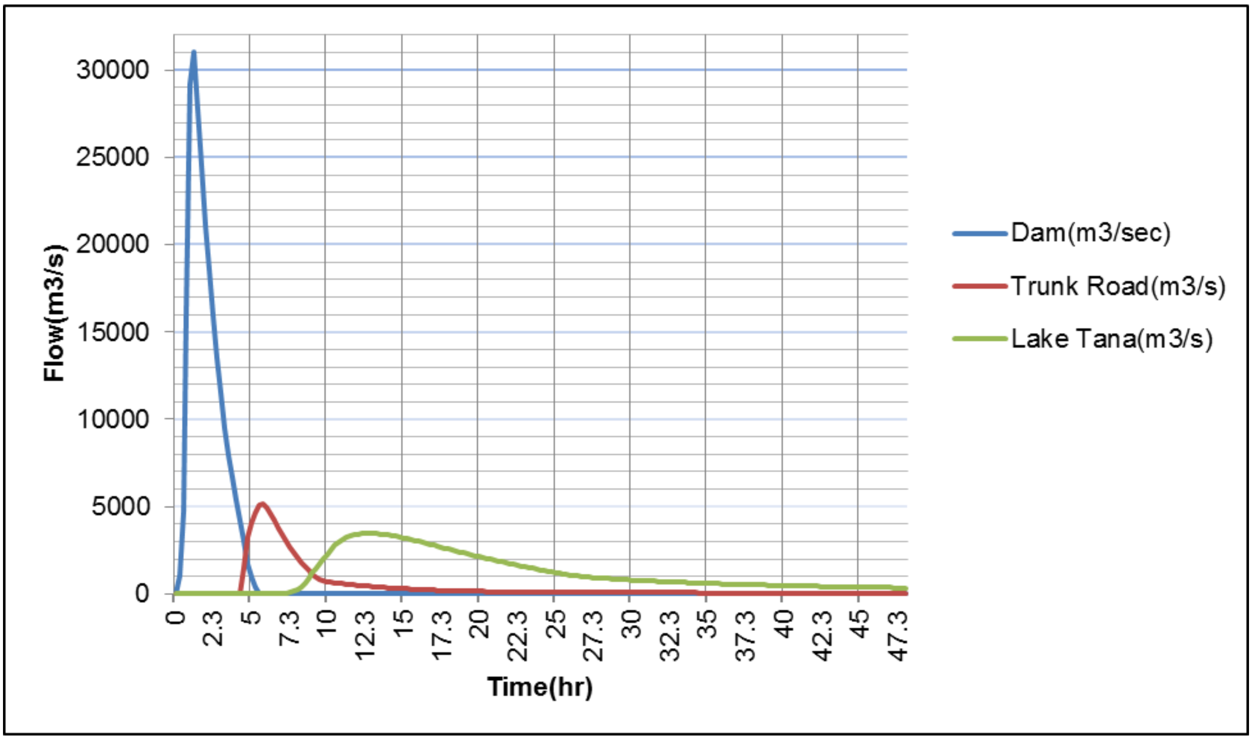


Figure 5-13: Maximum flow hydrographs at selected locations, piping by Frohelich 2008 method.

5.3.1.4 VON THUN AND GILLETE

Table 5.14 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Von thun and Gillete method.

Table 5-14: Von thun and Gillete method breach peak flow, maximum velocity, Maximum flood depth and time to peak.

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth(m)	Time to peak (Day, hr)
Dam	0.26	40,075	6.73	22.59	20Sep 2020,01:15
Pick up weir	29.97	30,623	4.29	13.61	20Sep 2020,03:10
Trunk road from Addis Ababa to Gonder	50.98	6,513	3.39	3.81	20Sep 2020,05:00
Lake Tana	75.38	3,569	0.85	9.00*	20Sep 2020,11:45

*Depression area

Among the piping mode of failure results Von thun and Gillete has result the maximum peak flow of 40075 m3/s and flood depth of 22.59 m as shown in the table above. This method's results (i.e. peak flow, velocity and flood depth) have same rising and falling patterns as other methods but with different magnitudes likewise as observed for overtopping mode of failure. According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 24:00GMT on 19/9/2020 and 11:45GMT on 20/9/2020 respectively. The whole flood attenuation in the downstream regions has taken around 11:45 hours as shown in the figure below.

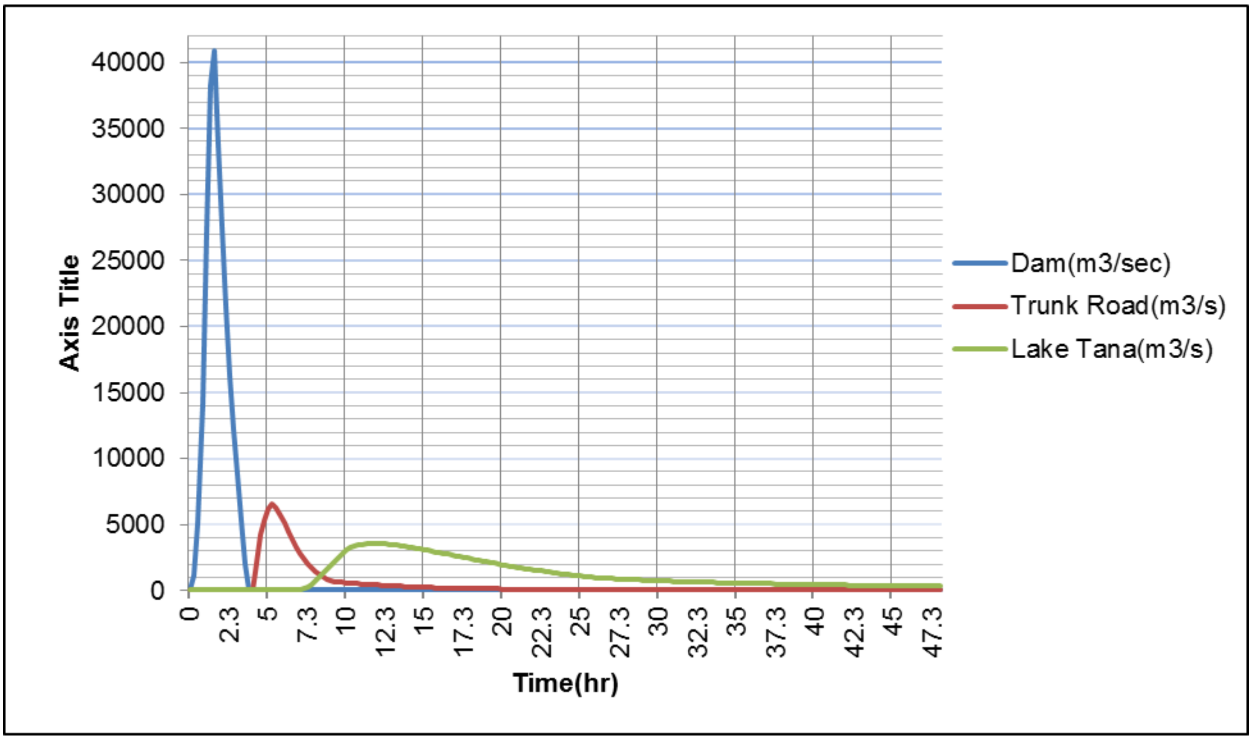


Figure 5-14: Maximum flow hydrographs at selected locations, piping by Von Thun and Gillette method.

5.3.1.5 XU AND ZHANG

Table 5.15 shows the peak flow, time to peak, high flow velocity and maximum flood depth for the selected Ribb river stations as per Xu and Zhang method.

Table 5-15: Xu and Zhang method breach peak flow, maximum velocity, Maximum flood depth and time to peak.

Naming	Stations (Km)	Peak flow (m3/s)	Maximum Velocity (m/s)	Maximum flood depth(m)	Time to peak (Day, hr)

Dam	0.26	22,583	4.40	17.94	20Sep 2020,03:15
Pick up weir	29.97	15,560	3.51	12.14	20Sep 2020,04:15
Trunk road from Addis Ababa to Gonder	50.98	4,845	3.29	3.53	20Sep 2020,07:15
Lake Tana	75.38	3437	0.83	8.98*	20Sep 2020,14:30

*Depression area

According to the dam breach simulation, the fictional dam breach and the peak flood entry to Lake Tana will occur at 24:00GMT on 19/9/2020 and 14:30GMT on 20/9/2020 respectively. Figure 5-15 shows the maximum flow hydrographs at selected locations for piping case by Xu and Zhang method. There are two peaks which will occur at 1:15GMT and 3:15GMT on 20/9/2020 respectively in which the first one is a smaller one. This is due to the decrease in head for the orifice flow and then the rise of the flow rate due to the weir flow condition. The case is recognizable in this case due to higher breach formation time. The whole flood attenuation in the downstream regions has taken around 14:30 hours as shown in the figure below.

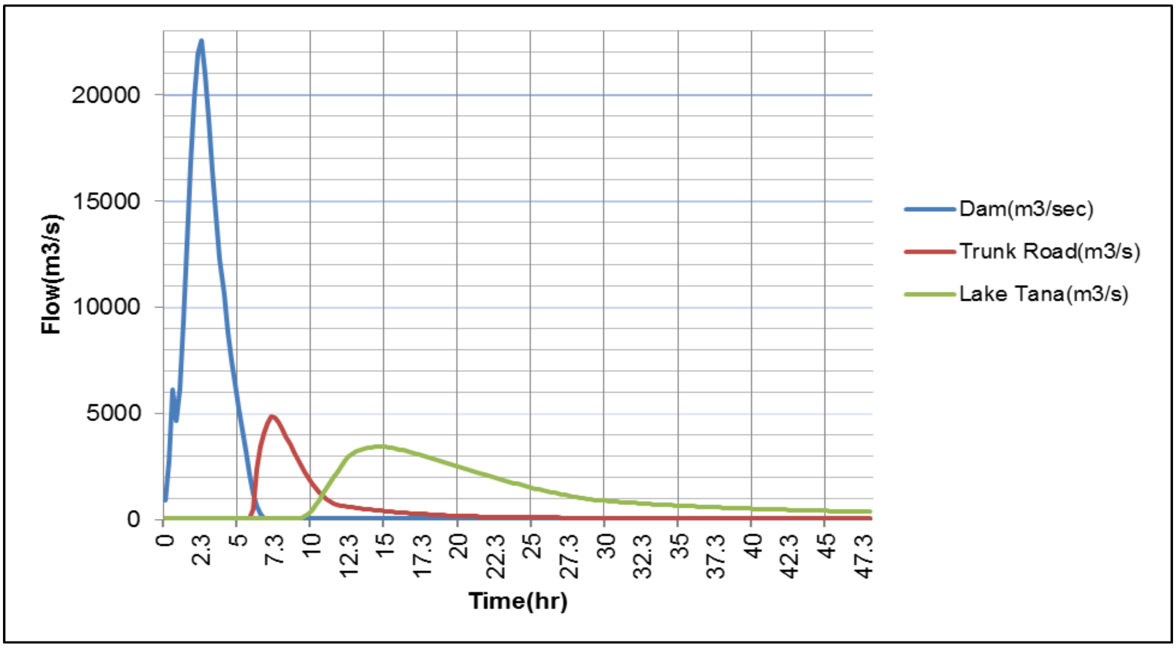


Figure 5-15: Maximum flow hydrographs at selected locations, overtopping by Xu and Zhang method.

5.3.2 DAM BREACH RESULTS COMPARISON

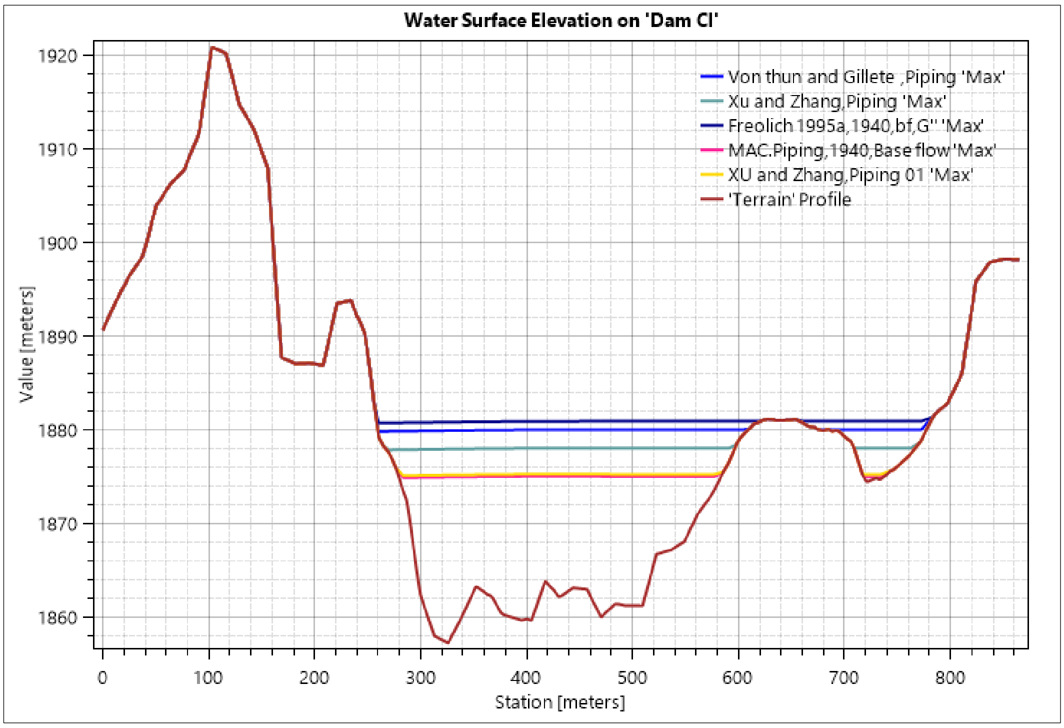


Figure 5-16: Dam area cross section showing water surface elevation for different methods

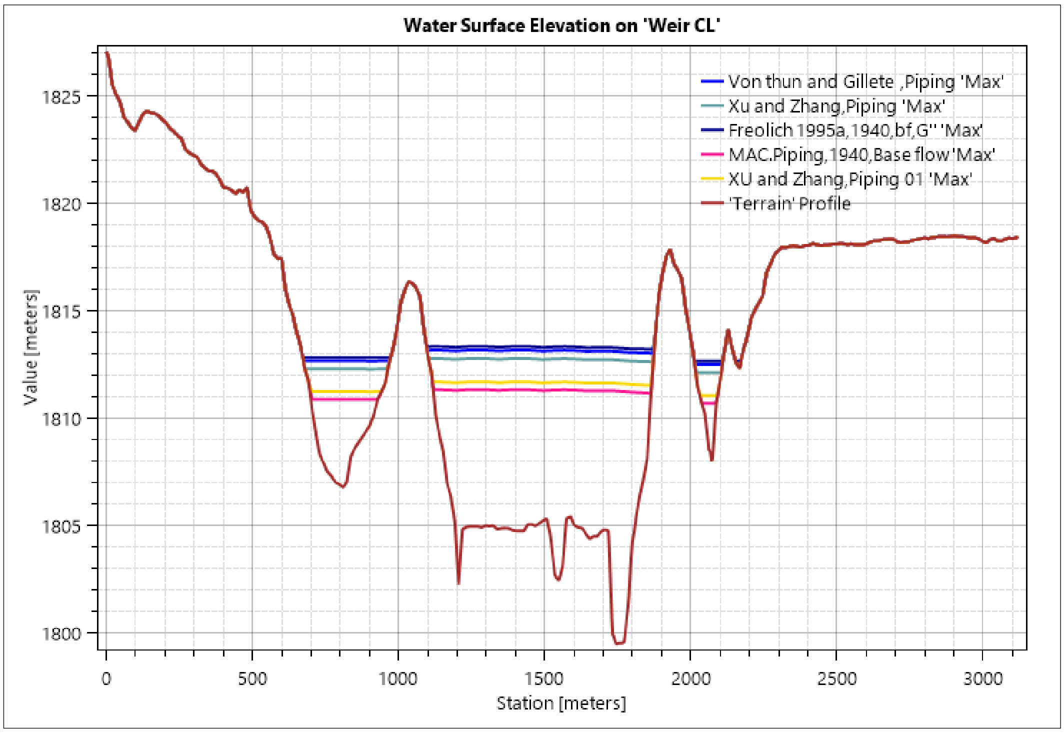


Figure 5-17: Pickup weir area cross section showing water surface elevation for different methods.

As shown in Figure 5.16 and 5.17, the water surface elevation and depth for different breach parameters at selected stations is presented in one graph. The selected stations are at a distance of 0.26km and 29.97km from the dam. Like the overtopping scenerio, it is clear that all the graphs follow same trend for different breach parameters along the reaches. Moreover, the flow depth difference between different breach equations decrease towards to Lake Tana as shown in Figure 5.16 and 5.17 and Table 5.10 to 5.14. This shows clearly the attenuation and convergance of the flood at the floodplain area.

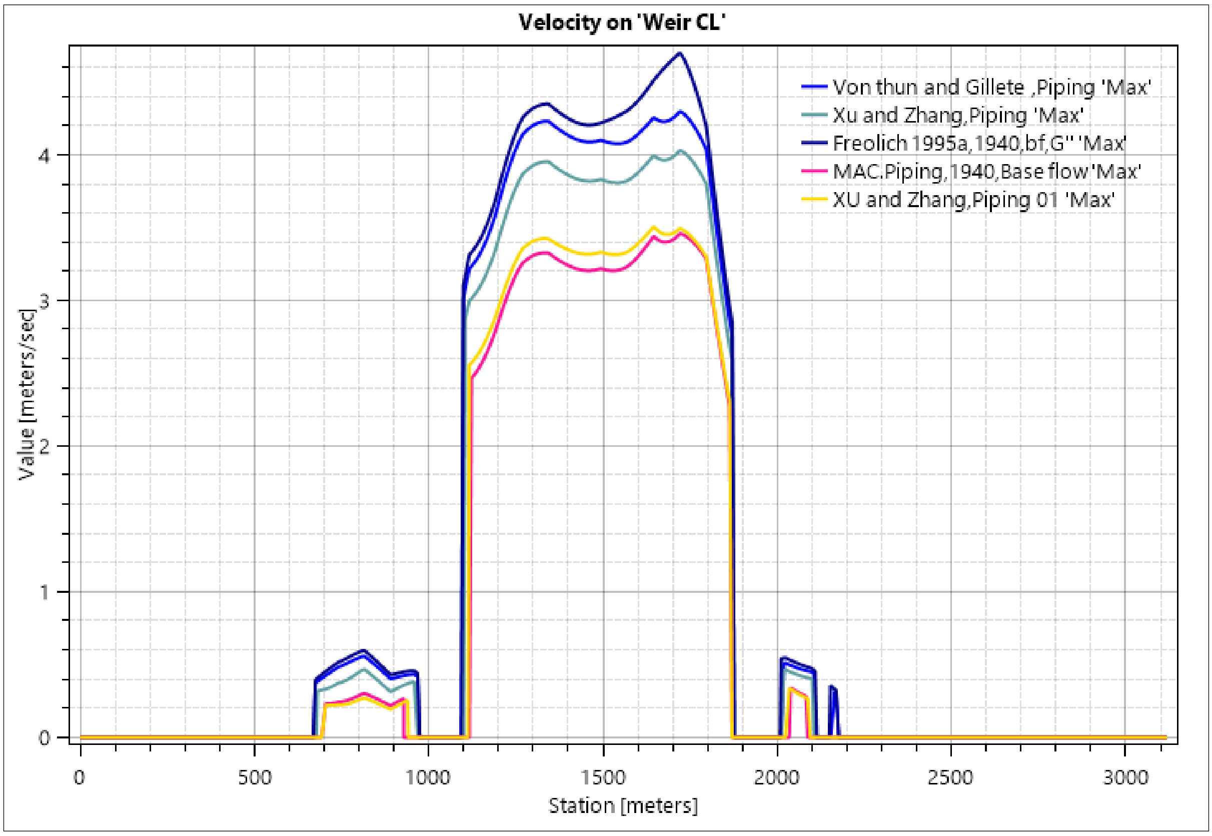


Figure 5-18: Pickup weir area velocity for different methods

The velocity drawing at the pickup weir area is shown in Figure 5.18. Like the overtopping failure case, the velocity pattern for different breach parameter equations has the same trend. The peak velocity of the flood is experienced at the dam area due to high hydrostatic pressure of the stored water behind the dam and then followed by confined and steeper section of the terrain geometry.

As shown below, each set of breach parameters and failure times produce a different outflow hydrograph. Most of the hydrographs are sharp hydrograph in which tends to attenuate much more quickly. As the hydrographs move downstream, it substantially converged and attenuates the peak flow starting at a river station 29.97km, which is the start of the floodplain.

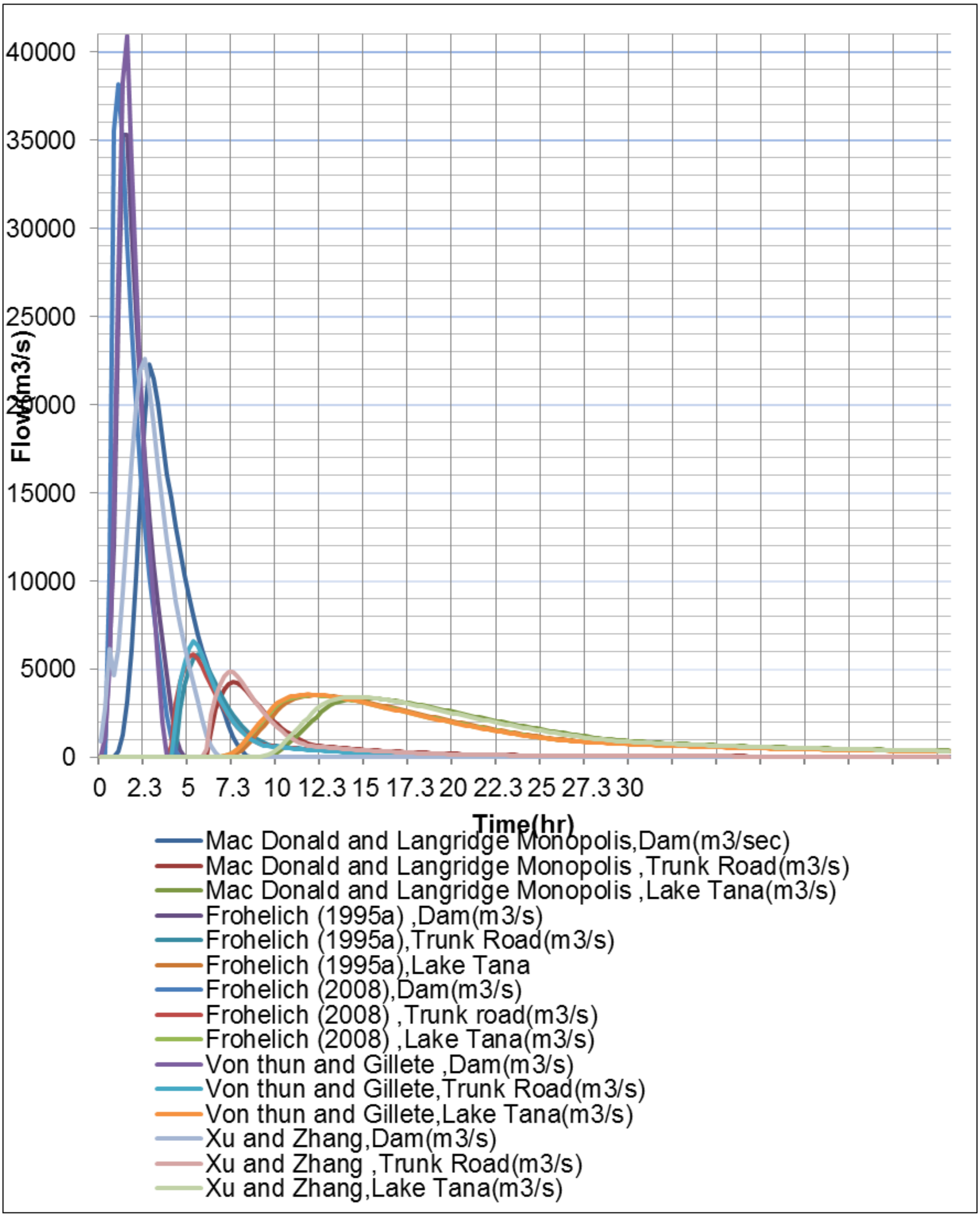


Figure 5-19: Flow hydrographs at selected locations for different breach parameters.

5.3.3 PIPING BREACH OUTFLOW VERIFICATION

The computed peak outflow from the model compared to the regression equations and the envelope curves of historic failures as test of reasonableness is shown below.

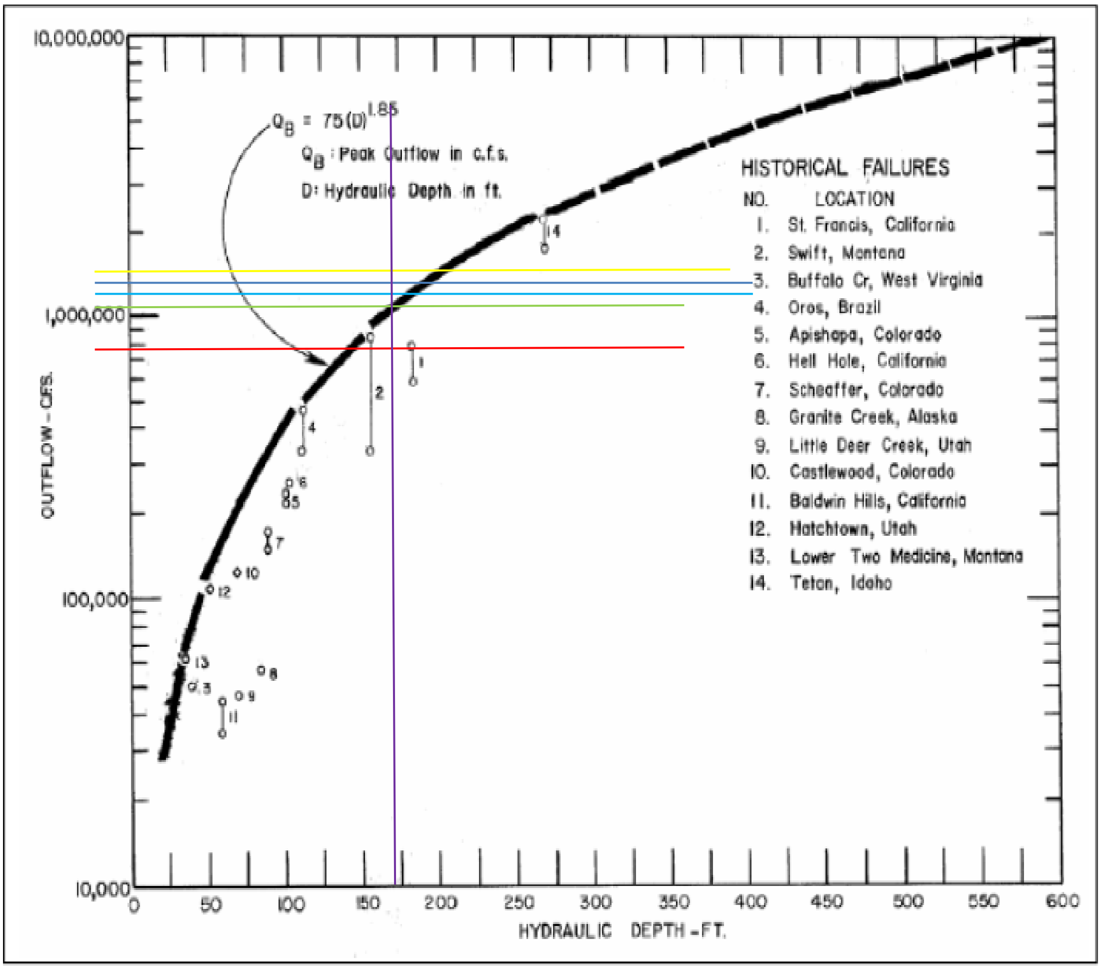


Figure 5-20: Verification of piping outflows using historic outflow rates envelope

Mac Donald and Langridge Monopolis method 22,269m3/s or 786422ft3/s

Frohelich (1995a) method 35,297m3/s or 1246502ft3/s

Frohelich (2008) method 31,057m3/s or 1096768ft3/s

Von thun and Gillete method 40,926m3/s or 1445288ft3/s

Xu and Zhang method 22,583 m3/s or 797511ft3/s

Hydraulic depth 132ft

A detail assessment of the graph reveals that Mac Donald and Langridge Monopolis and Xu and Zhang's methods have resulted a better realistic result with a difference of 20.31% and 21.42%, whereas Frohelich’s (1995a) ,Frohelich’s (2008) and Von thun and Gillete's methods are found to have 49.72%,42.86% and 56.64% difference respectively. In fact, the equation and the envelope are developed from limited data sets in which most of the dams considered were small dams.

In addition, the peak discharge from the HEC RAS run results and peak discharge equation are summarized as below.

Table 5-16: 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 (%)
Mac Donald and Langridge Monopolis	22,269	67,930	-45,661	205.04
Frohelich (2008)	31,057	38,837	-7,780	25.05
Xu and Zhang	22,583	105,184	-82,601	365.77

Moreover, another check for the reasonableness is done by reviewing the flow rate and velocities going through the breach in comparison with breach development time and size.

Table 5-17: HEC-RAS results and Equation breach development time comparison.

Methods	HEC RAS Dam breach time (hr)	HEC RAS Minimum Tail water time (hr)	Time Difference (hr)	Breach development time from the equation (hr)	Difference (hr)	Percentage
MacDonald and langridge-Monopolis	24:00	08:15	8:15	3.31	4.94	59.88%
Frohelich (1995)	24:00	04:30	04:30	1.47	3.03	67.33%
Frohelich (2008)	24:00	05:30	05:30	1.19	4.31	78.36%
Von thun	24:00	04:00	04:00	1.59	2.41	60.25%

and Gillete						
Xu and Zhang	24:00	07:30	07:30	3.62	3.88	51.73%

As stated in the above findings, the difference is huge for all breach equations in which it will not be realistic to accept the result.

Thus, In accordance with all validation and reasonableness work, no method is consistent and the Author decided to consider the worst scenario which is Von thun and Gillete method. Moreover, it is observed that all the breach model results are very much exaggerated when compared with the other ones. In fact, all the piping mode of failure has obviously resulted in lesser magnitudes than overtopping does.

5.4 UNCERTAINTY ANALYSIS

Through Monte Carlo simulation by McBreach, the exceedance probability peak discharge and their respective sampled breach parameters for both overtopping and piping modes of failure are produced. Table 5.18 shows the specified exceedance probability peak discharge and their respective sampled breach parameters based on the breach parameter data, distribution and deterministic value as per table 4.6 after 10000 realization in which the mean and standard deviation converges.

Table 5-18: Specified exceedance probability peak discharge and their respective sampled breach parameters for overtopping modes of failure.

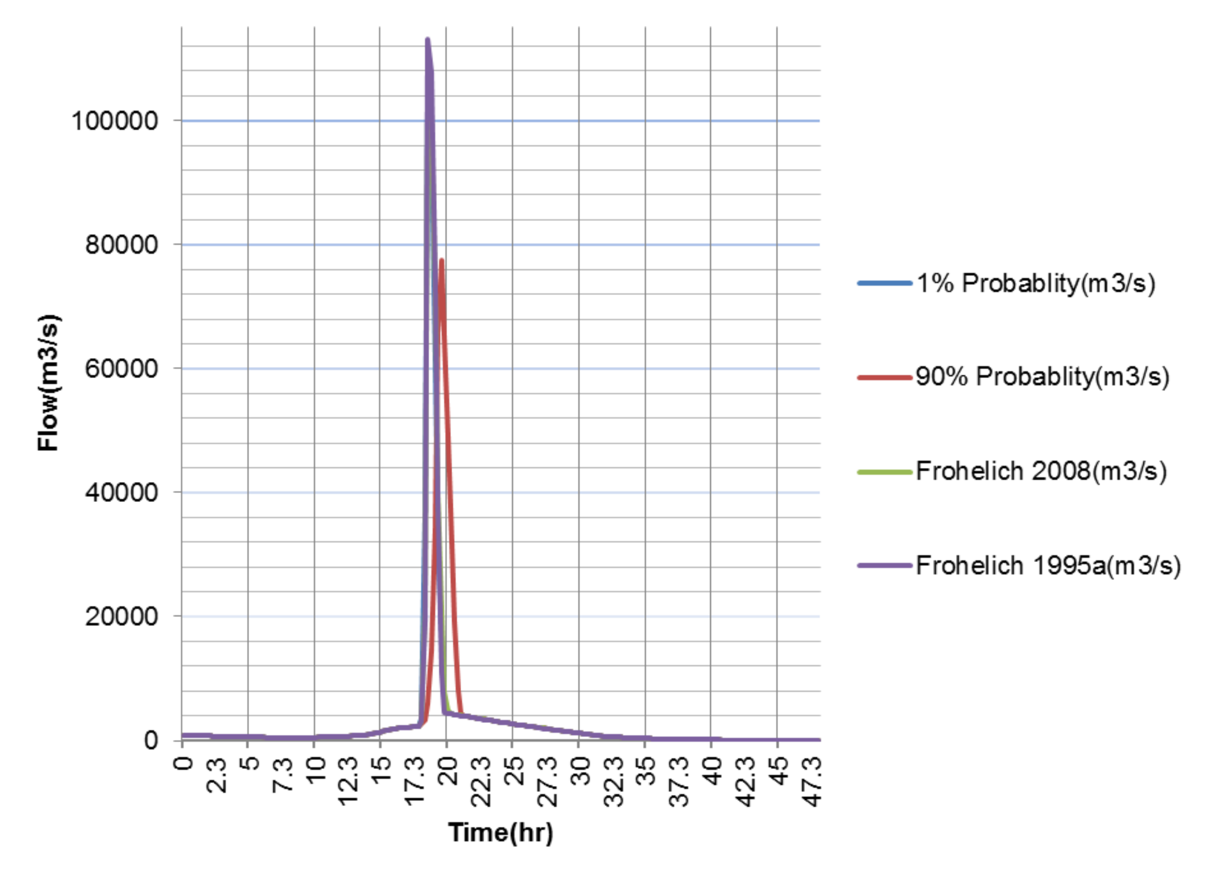
Exceedance Probability	0.2%	1%	5%	10%	50%	90%	95%	99%
Bottom width, m	172.19	144.27	147.87	159.20	173.37	137.22	131.62	155.12
Discharge Coefficient	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44
Formation time, hr	1.20	1.15	1.59	1.97	2.87	2.99	2.82	3.73

Initiation Elevation, m	1945.66	1945.66	1945.66	1945.66	1945.66	1945.66	1945.66	1945.66
Invert Elevation, m	1873.00	1873.00	1873.00	1873.00	1873.00	1873.00	1873.00	1873.00
Left side slope, m/m	0.84	0.59	0.86	0.70	0.86	0.77	0.38	0.60
Right side slope, m/m	0.83	1.02	0.77	0.91	0.73	0.67	0.89	0.38
Progression	Sine	Sine	Sine	Sine	Sine	Sine	Sine	Sine
Peak discharge, m3/s	110463	104379	973879	938179	84581	77521	75312	70607

The exceedance probability peak discharges and their respective sampled breach parameters for piping is smaller in which more emphasize is given only for the overtopping failure case.

Figure 5.21 shows the peak flow and time to peak for different probability values, Frohelich 2008 and Frohelich 1995a.

Figure 5-21: Outflow breach flow hydrographs for different probability values, Frohelich 2008 and Frohelich 1995a.



5.5 SENSITIVITY ANALYSIS

The sensitivity analysis of manning's roughness coeffcient (n) on the breach out flow hydrograph and on the downstream flow condition is done by reduction and increment of manning's n by a value of 30%. It is performed on MacDonald and langridge-Monopolis method simulation for overtopping mode of failure.

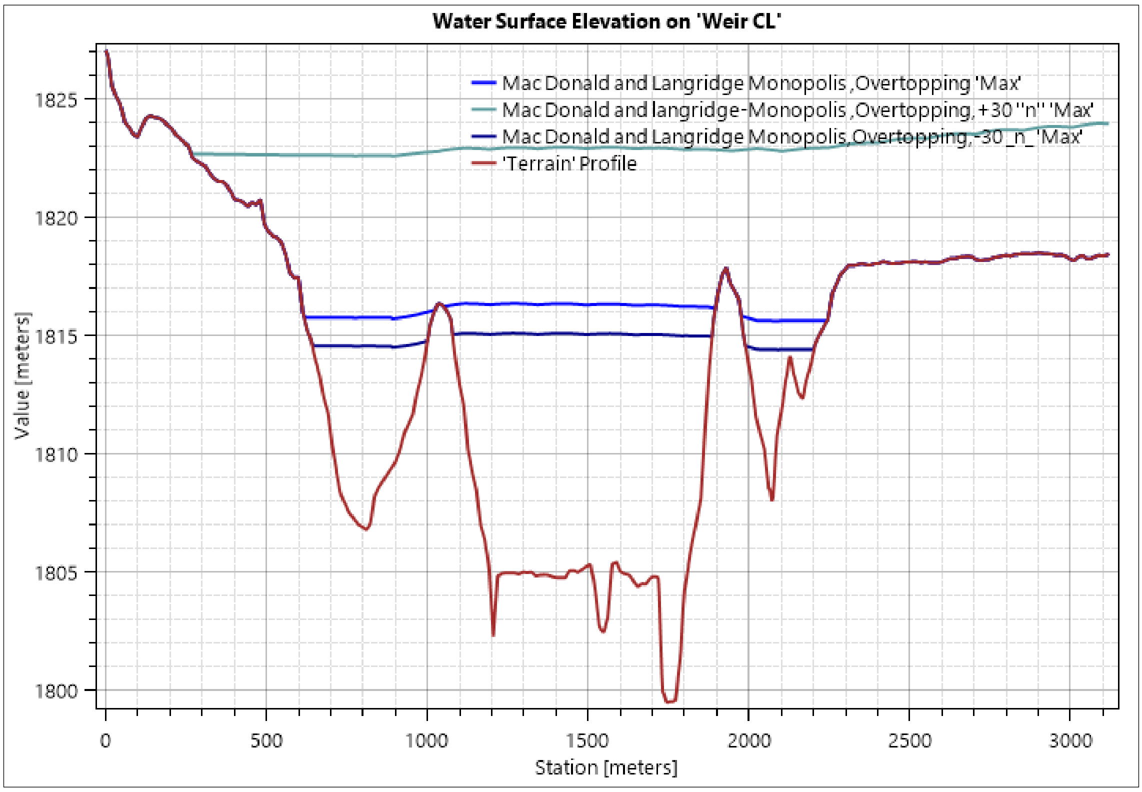


Figure 5-22: Pickup weir area cross section showing water surface elevation for different Manning roughness coeffcient “n”

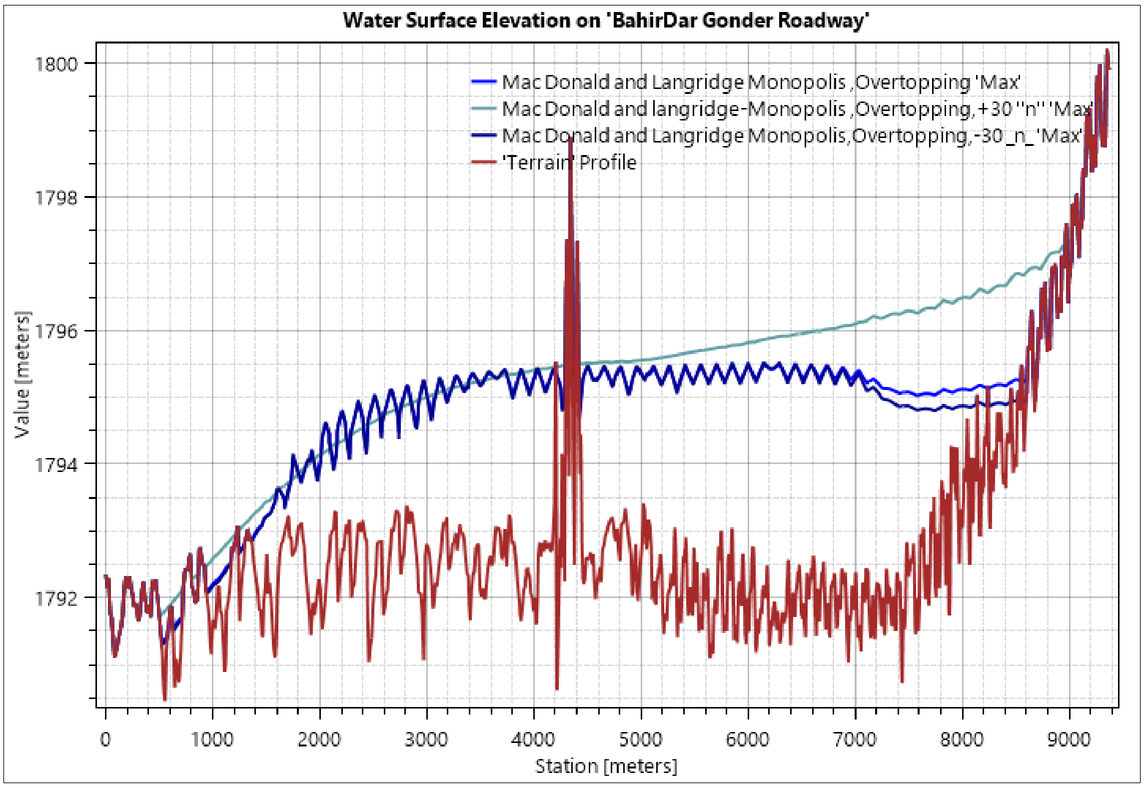


Figure 5-23: Trunk road area cross section showing water surface elevation for different Manning roughness coeffcient “n”.

Table 5-19: Maximum flow depth, Maximum velocity, time to peak and peak discharge for different manning's roughness coeffcient values.

Flow parameter	Location Distance from the Dam CL(Km)	Sensitivity scenarios				
		Increment on "n" +30%	Original value of "n" +0%	Reduction on "n" -30%	%Increment on original value	%Reduction on original value
Flow depth	0.26	30.41	27.62	23.95	10.10	-15.32
	29.97	23.42	16.73	15.54	39.99	-7.66
	50.98	5.54	4.67	4.69	18.63	0.43
	75.38	9.03	9.02	9.02	0.11	0.00
Velocity	0.26	10.64	10.89	12.21	-2.30	10.81
	29.97	4.50	5.40	6.86	-16.67	21.28
	50.98	3.43	3.50	4.69	-2.00	25.37
	75.38	1.00	1.28	1.78	-21.88	28.09
Time to peak	0.26	20Sep 2020,20:00	20Sep 2020,20:00	20Sep 2020,20:00	0hr	0 hr
	29.97	20Sep 2020,21:50	20Sep 2020,21:30	20Sep 2020,21:10	+0:20hr	-0:20hr
	50.98	20Sep 2020,23:00	20Sep 2020,22:45	20Sep 2020,22:15	+0:15hr	-0:30hr
	75.38	21Sep 2020,05:00	21Sep 2020,03:45	21Sep 2020,01:15	+1:15hr	-2:30hr
Peak	0.26	67570	67570	67570	0	0
	29.97	44526	45526	46568	-2.20	4.29

discharge	50.98	16572	17854	18093	-7.8	1.32
	75.38	7103	8954	12185	-20.67	26.52

As per Table 5-19, it was found that a 30% increment in manning roughness values have significant effect on the computed maximum water surface elevation for the reach between the dam and trunk road way (39.99%). However, the 30% decrease in manning roughness values doesn't cause a significant effect on flow depth in the above specified reach but cause a significant effect on the velocity, travel time and discharge for the reach between the pickup weir to Lake Tana (28.09%).

Therefore, it can be deduced that judicial selection of manning roughness values would have a significant effect while a preparing inundation maps but not significantly on the emergency action plans and on evacuation plans for the reach between the dams to trunk road way. The reverse holds true especially for the reach from trunk roadway to Lake Tana in which a certain safety time margin in terms of hours might be provided for the computed flood wave arrival times.

5.6 FLOOD INUNDATION MAPPING

Flooding from the Ribb and Fogera rivers is not uncommon in the Fogera plain during the rainy season. Ribb Irrigation and Drainage Project is situated in Ribb River System Delta area near Lake Tana. Ribb River typically exhibits lower river reach characteristics as it flows dissecting Ribb Irrigation and Drainage Scheme Command area. Meandering, minor bed shrinking, excessive sedimentation and sub division in to smaller channels and continuous decreasing of bed slope and thus flow concentration (BRL Ingenerie, 2015). The reduction of the river slope result in a less and less in channel flow concentration and dominance of flood plain flow. It is separated floodplains for both rivers extend further up both rivers for many kilometers upstream of the highway. Insufficient cross-drainage through the highway embankment has exacerbated upstream flooding in the past, but this has been partially mitigated by the recent placement of culverts. (Flood Risk Mapping Consultancy for Pilot Areas in Ethiopia, 2010).

In fact, all observed flooding's in the previous years are mainly caused by the heavy rainfall intensity in which the flooding will be worse if this hypothetical dam breach happened. The consequences of flooding are enormous, complex and far-reaching. These implications include direct damage to buildings and properties, economic activity disruption and the displacement of people, the resulting evacuation costs and temporary accommodations and environmental consequences. Although damage is often associated with mainly with the depth of flooding, extent of flooding including the duration of flooding, velocities associated with peak flows and the sediment content of flood waters in which the flood inundation map work will be a prerequisite. Flood hazard map will aid to analyse and precisely define the magnitude and extent of flood on a set area in turn to demarcate the areas at risk and help to review the area's planned settlement and infrastructure.

The flood inundation map for the entire study area for Ribb dam failure is presented for overtopping type failure and for piping type failure in which the likely scenario and the worst case scenarios are selected. However, the flood inundation area for the overtopping failure modes is significantly higher than the piping failure mode due to the breach cross sectional area of flow. Indeed, the breach parameters have significant meaning on the outflow discharge and preparing flood inundation mapping for a dam breach. Economic cuts and the accomplishment of optimal safety in selecting an appropriate method should be taken into account and, in turn, have a direct influence on the economy of flood hazard minimization plan and emergency action plan (EAP).

Once the model is run, the in-built RAS Mapper tool in HEC-RAS was used to predict inundation extents and water surface elevations, including information on depths and velocities in turn also the flood arrival time that all the results enable the identification of flood hazard areas. Based on these maps, hazard classification is made in order to implement effective flood forecasting and warning system.

Flood hazard maps (extent, depth, velocity, and duration) are fairly straightforward to interpret and can be used for flood preparedness and response as well as for development planning. Eventhough the maximum discharge and water storage at the inundated area shall be considered, Frohelich (2008) method of parameter equation is selected for this flood mapping study as Frohelich (1995a) is not accepted as per the recommended government agency ranges.

The different flood area extent, depth and velocity maps produced for different time of occurrences are shown on Figure 5.24 to Figure 5.31 and also the profile and cross section drawing shown on Figure 5.32 to Figure 5.34. The maximum water surface area map given indicates that all infrastructures including the trunk road, the irrigation farm and Woreta town including the settlement around the town are inundated. Depth of flow at every location of the flood map can be extracted from the produced depth maps. Flood damage analysis can be made by investigating the depth of flow at every infrastructure. Flood depth with in the river is high ranging up to 33m as the flood travel to the Lake Tana.

Moreover, The 1% and 90% exceedance probability peak discharge and their respective sampled breach parameters are used to produce probability inundation (EPI) map. The 1% and 90% exceedance probability inundation map is shown below in on Figure 5.27 and Figure 5.28.

In addition, different studies and guidelines have different flood hazard classification limits and justifications. The FEMA (2020) guide line is used for this study. Flood severity grid prepared by FEMA is shown in the figure below with the depth*velocity product for each category of hazard level.

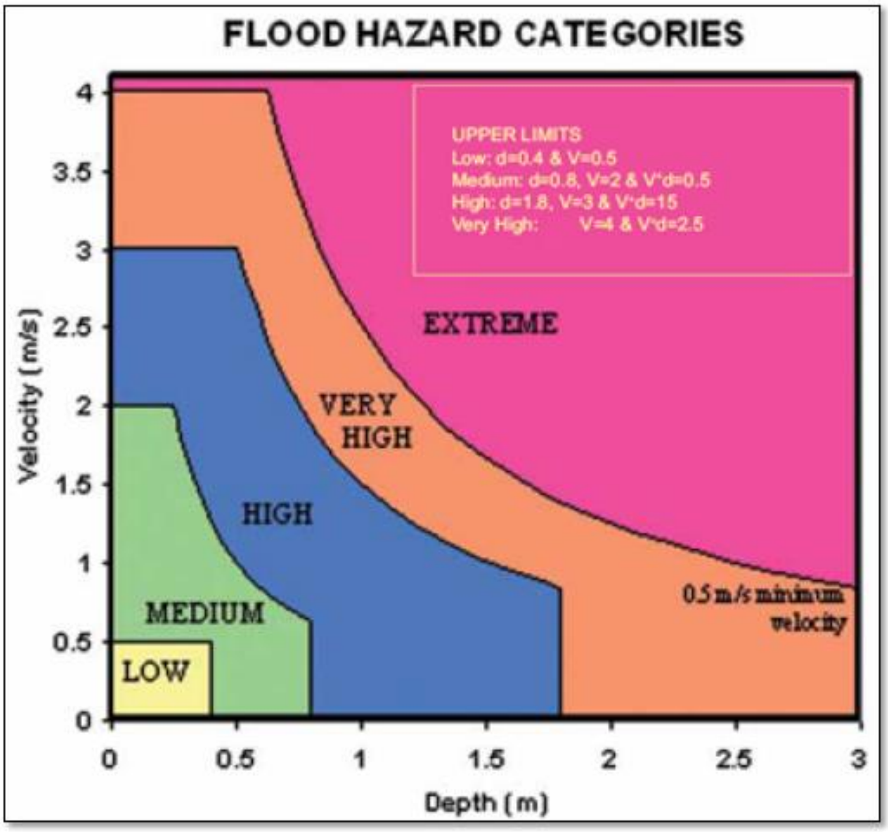


Figure 5-23: Flood hazard categories

According to FEMA (2020), to generate a flood severity grid that precisely matches the categorization shown in the figure above, additional rules would need to be applied when calculating the depth * velocity product that consider the depth and velocity upper limits of each category. Furthermore, flood severity thresholds differ reliant on whether they are considered in relation to the negative impact on humans, vehicles, or buildings (FEMA, 2020). When illustrating the dataset's results, the following depth * velocity categories might be used as a simplified way. Representing the information in the figure above in a simplified table, the FEMA document presented it as below.

Table 5-20: Simplified Flood Depth and Velocity Severity Grid Symbolization Categories (FEMA, 2020)

Flood Severity Category	Depth * Velocity Range (m2/sec)
Low hazard	< 0.2
Medium hazard	0.2 – 0.5
High hazard	0.5 – 1.5
Very High hazard	1.5 – 2.5
Extreme hazard	> 2.5

Thus, the flood Hazard map according to FEMA (2020) classification is prepared for overtopping failure (Frohelich (2008)) mode as shown in Figure 5.29. Thus, most areas between the dam and the trunk road are categorized high hazard and above high hazard. However, most part of the areas between the trunk road and Lake Tana are below high hazard level. The perimeter and area of inundation is 381Km and 299Km2 respectively. The flood affected with different magnitude all the twenty five kebeles from both woredas namely Agid Qirgna, Bira Abo,Agita, Estifanos,Shamo Godguadit, Birkutie, Bura Egaziabher Ab,Angot, Kidist Hana, Wagetera, Nabega, Bambik tsiyion,Genda wuha,Teza Amba, Kab, Tibaga, Shina Tsion, Wetneb, Addis Bete Christian, Rib Gabreal, Diba na Sifatira, Worta town zura, Abu Kokit, Shaga and Shina. Moreover, the entire irrigation scheme including the pickup weir, trunk road from Bahir Dar to Gonder and partial Woreta town will be inundated.

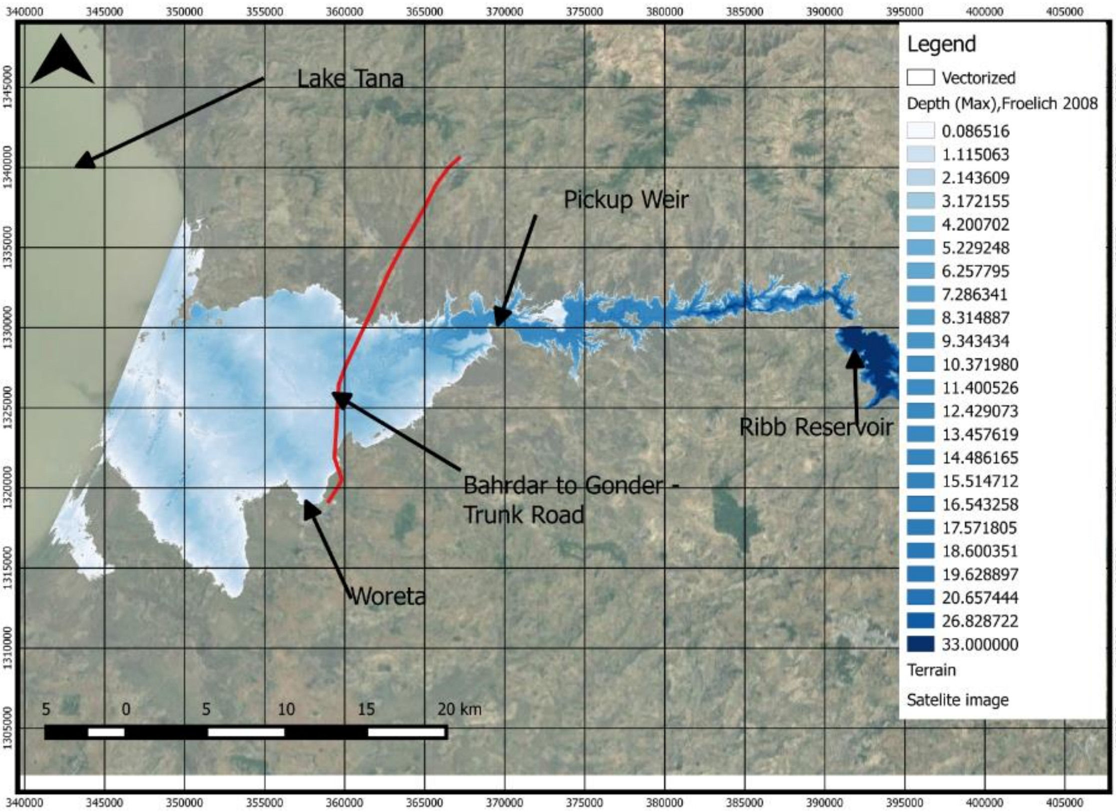


Figure 5-24: Maximum flood depth (Frohelich, 2008)

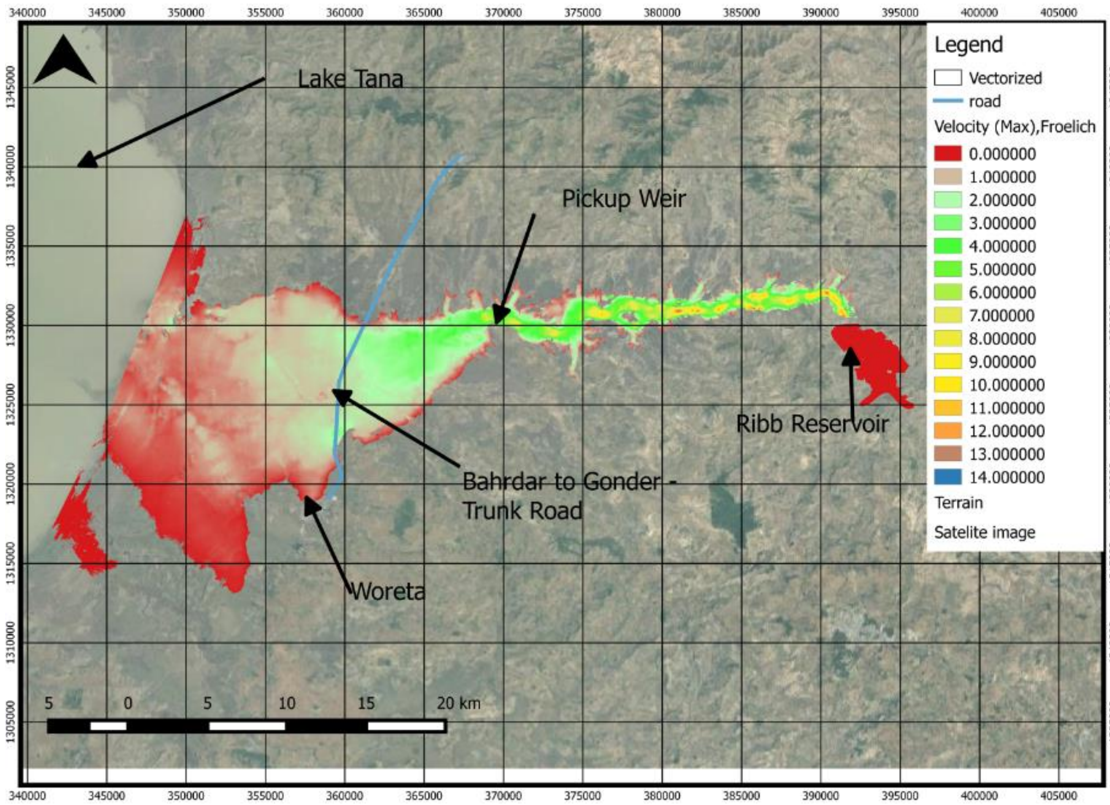


Figure 5-25: Maximum velocity (Frohelich, 2008)

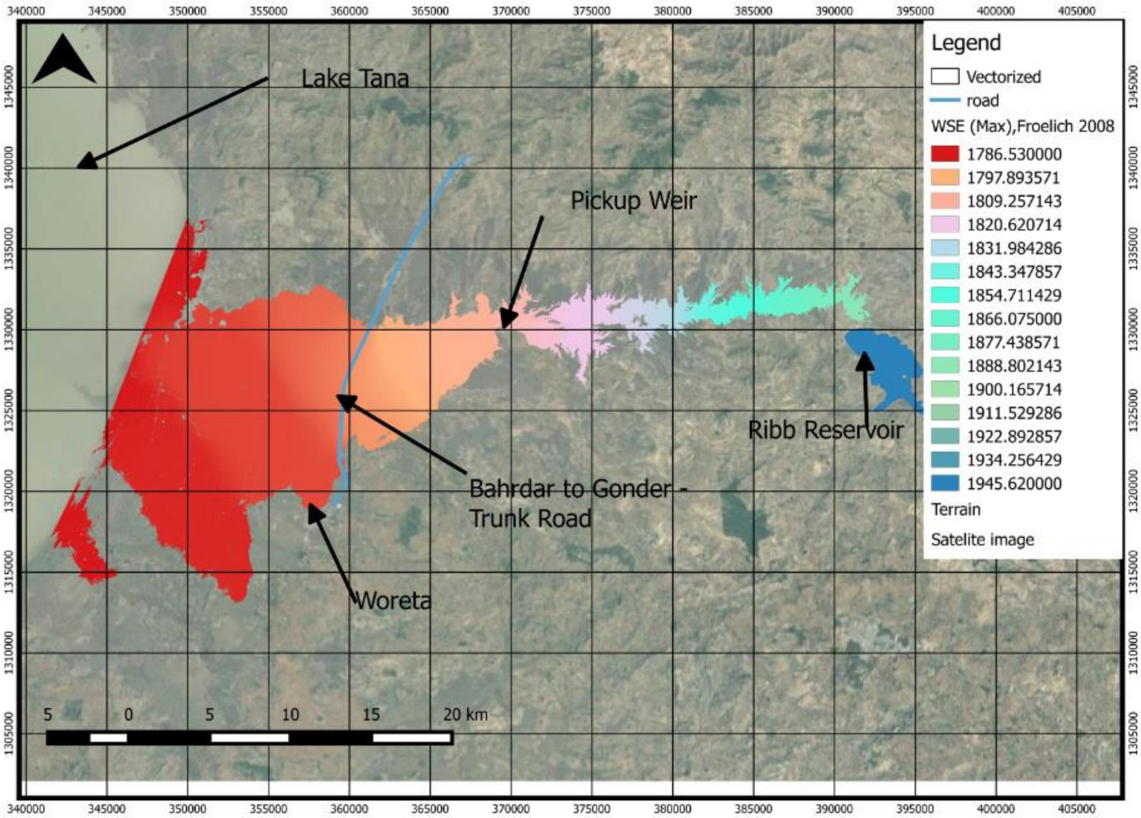


Figure 5-26: Maximum water surface elevation extent (Frohelich, 2008)

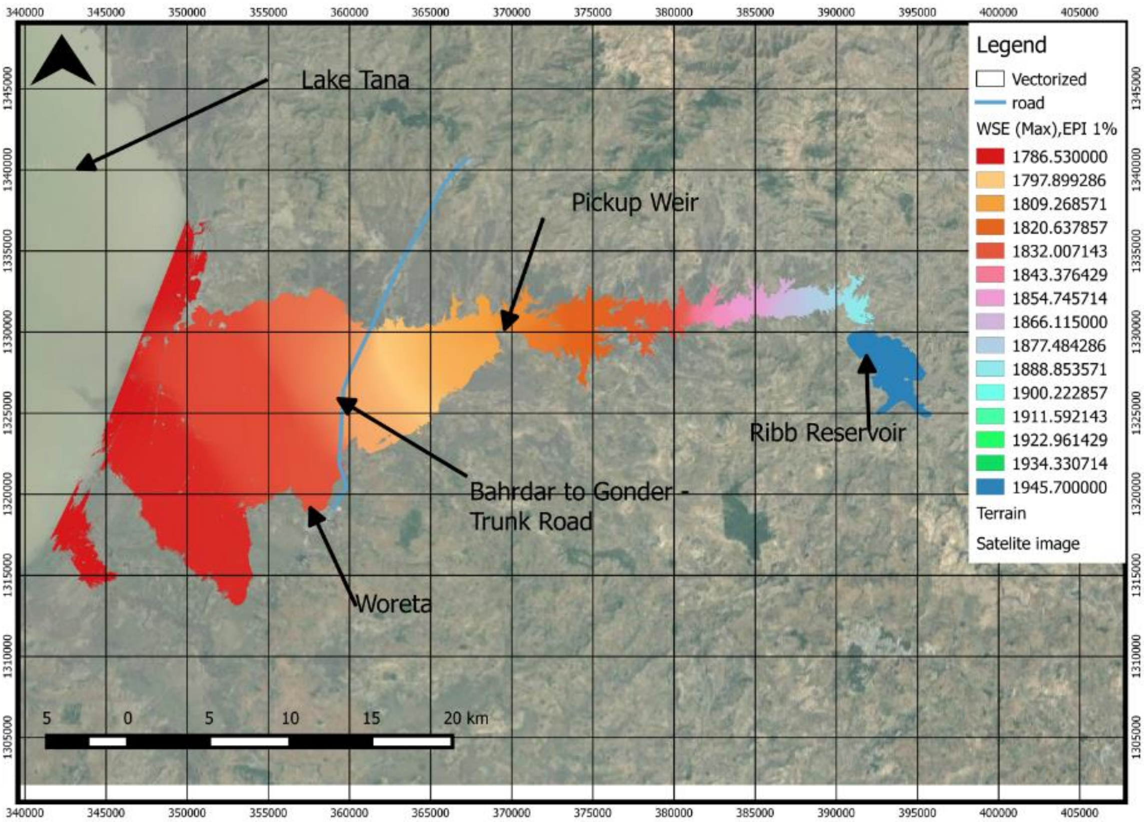


Figure 5-27: Maximum water surface EPI 1%

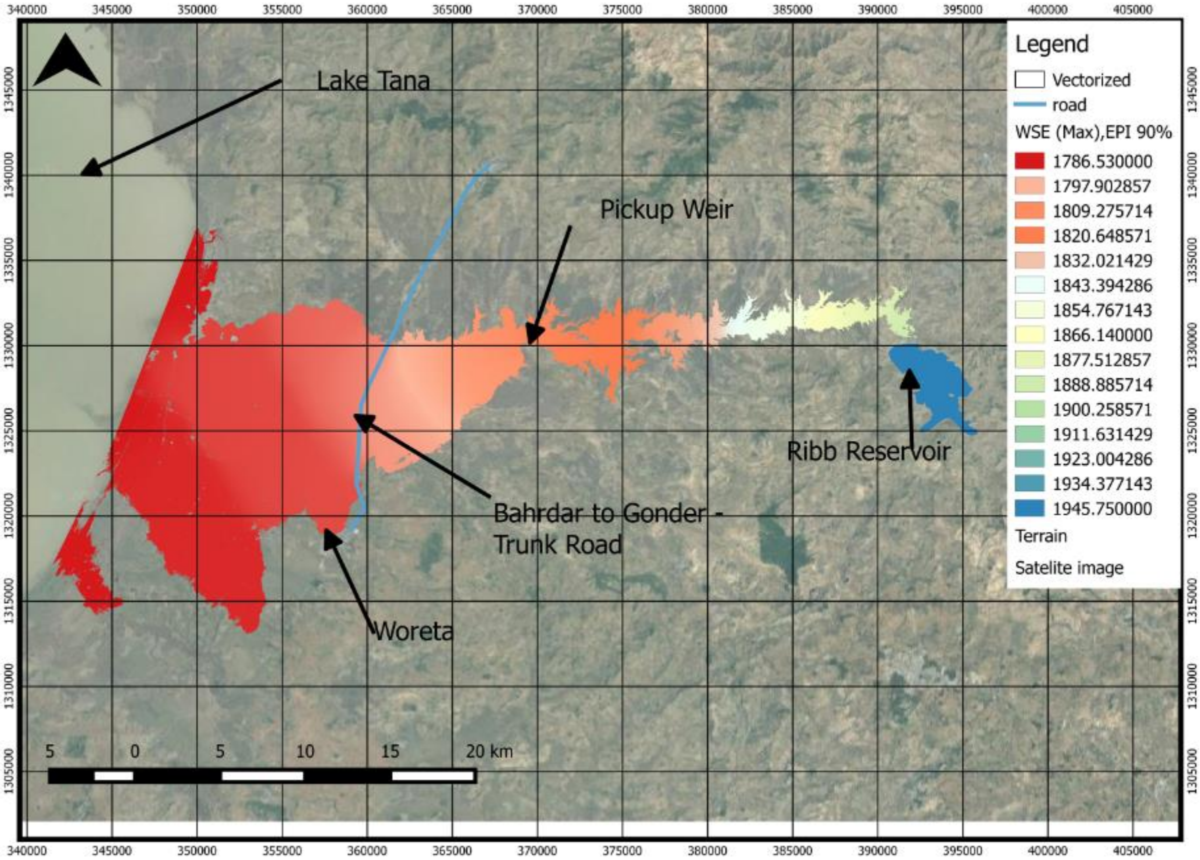


Figure 5-28: Maximum water surface EPI 90%

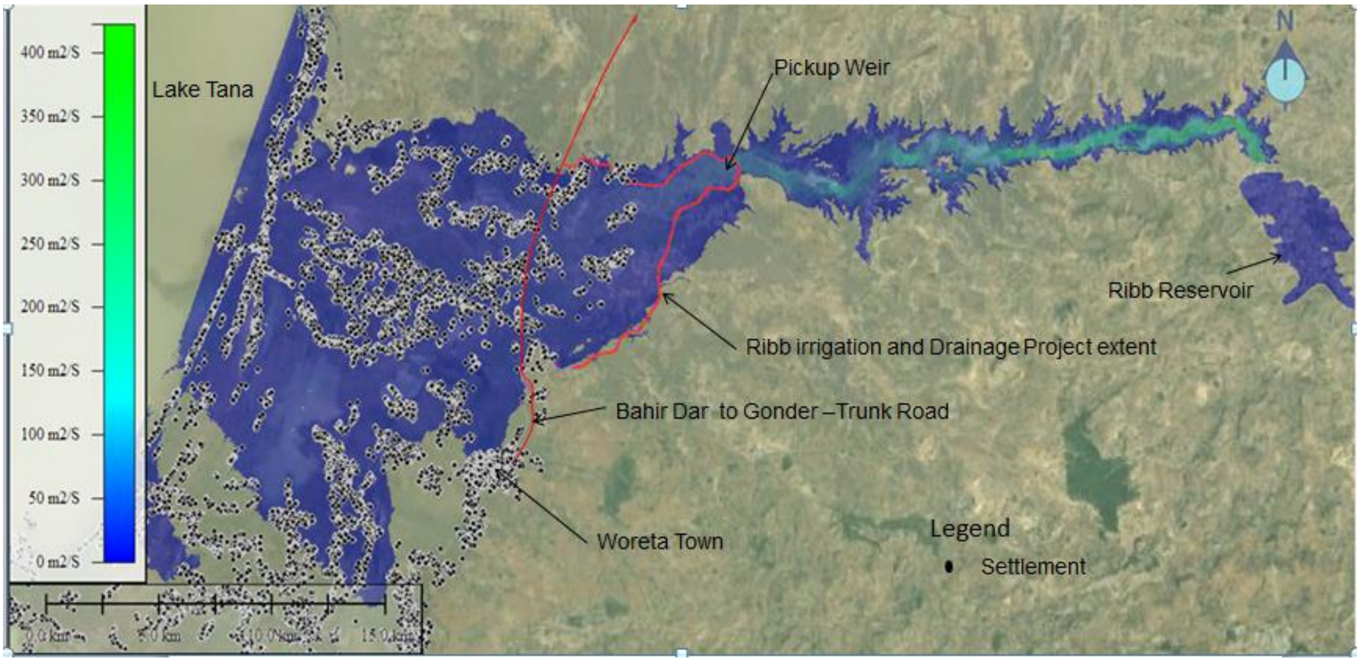
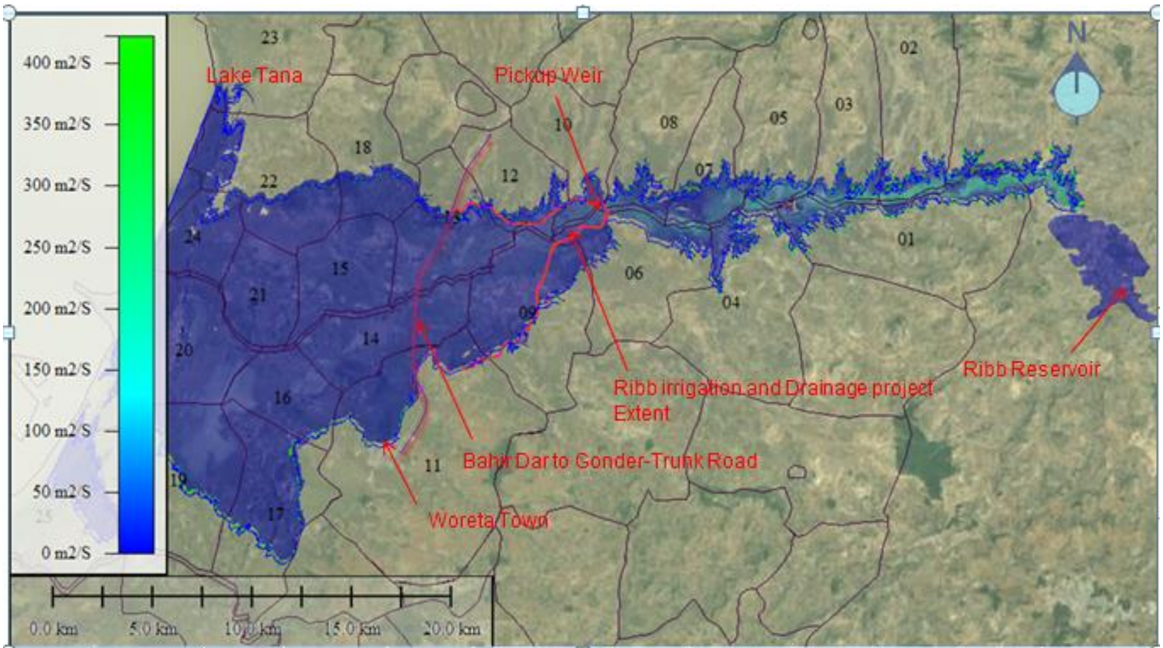


Figure 5-29: Hazard Map with the flood area settlement and the irrigation project extent (D*V) (Frohelich, 2008)



Code	Kebele Name	Code	Kebele Name	Code	Kebele Name	Code	Kebele Name	Code	Kebele Name	Code	Kebele Name
1	Wetneb	5	Estifanos	9	Diba na Sifatira	13	Shina Tsion	17	Shina	21	Genda wuha
2	Birkutie	6	Rib Gabreal	10	Angot	14	Abu Kokit	18	Tibaga	22	Kab
3	Shamo Godguadit	7	Agita	11	Worta town zura	15	Bambik tsiyion	19	Kidist Hana	23	Agid Qirgna
4	Addis Bete Christian	8	Bira Abo	12	Bura Egaziabher Ab	16	Shaga	20	Nabega	24	Teza Amba
										25	Wagetera

Figure 5-30: Flood extent and Hazard map with inundated kebele and Ribb irrigation and drainage project boundary (Frohelich, 2008)

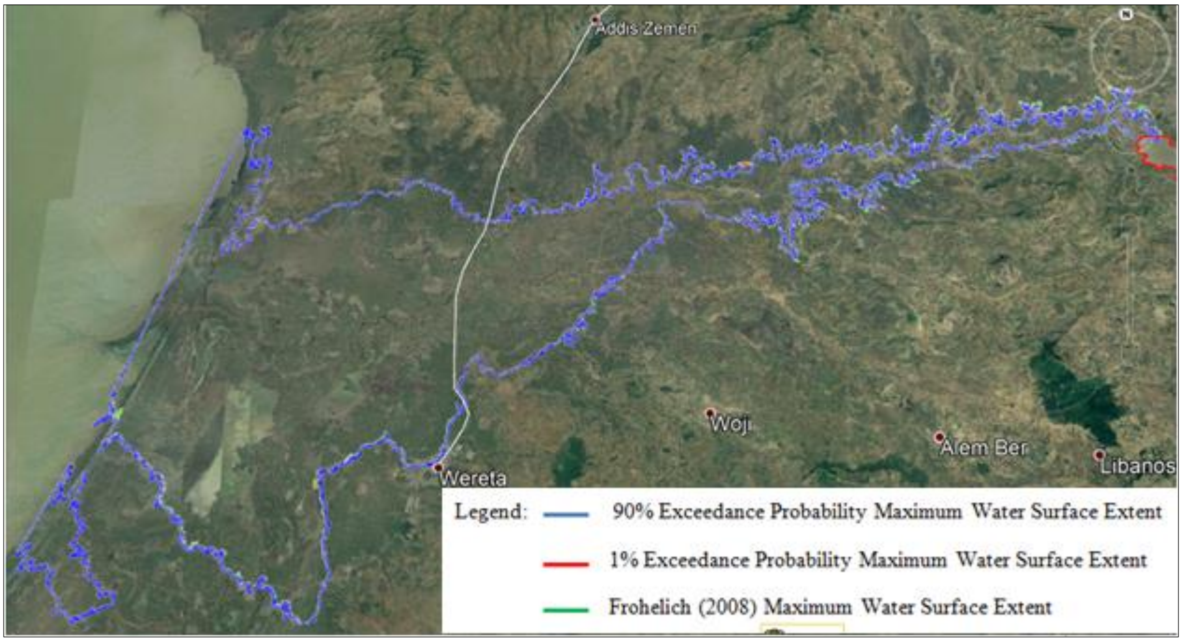


Figure 5-31: Maximum Water Surface Extent Boundary

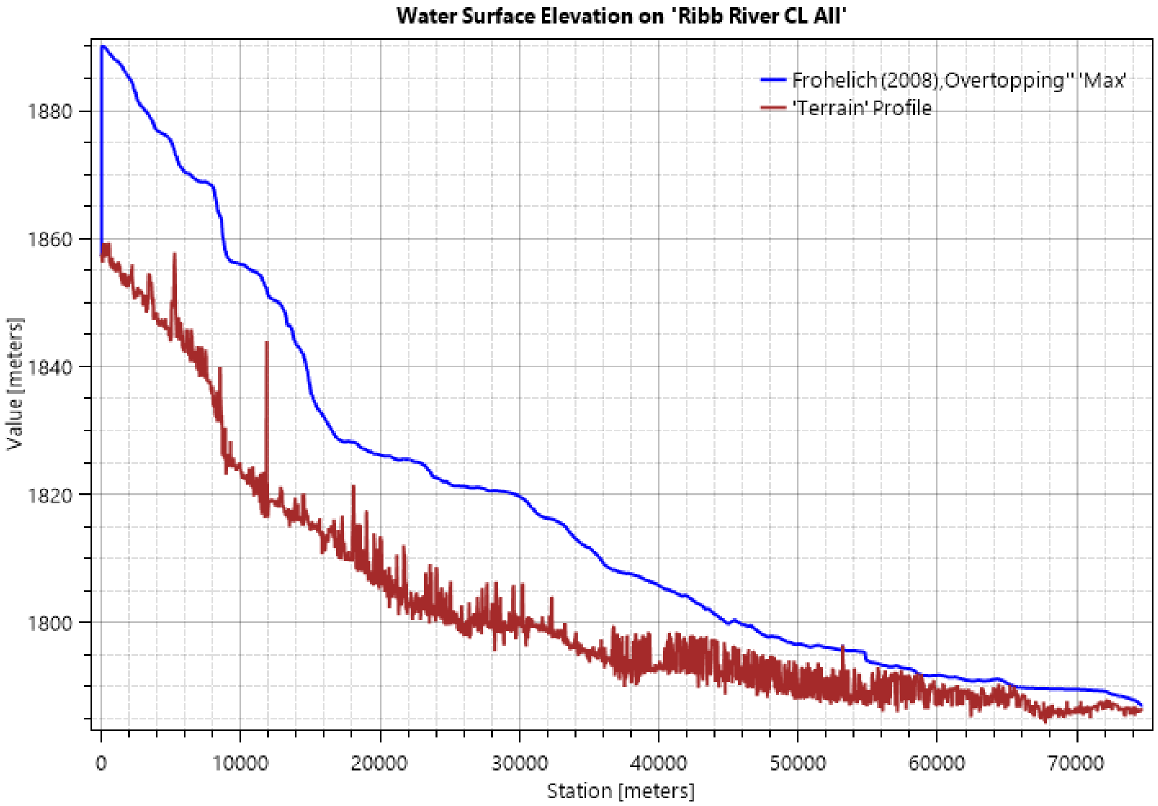


Figure 5-32: Maximum Water Surface Longitudinal Profile (Frohelich, 2008)

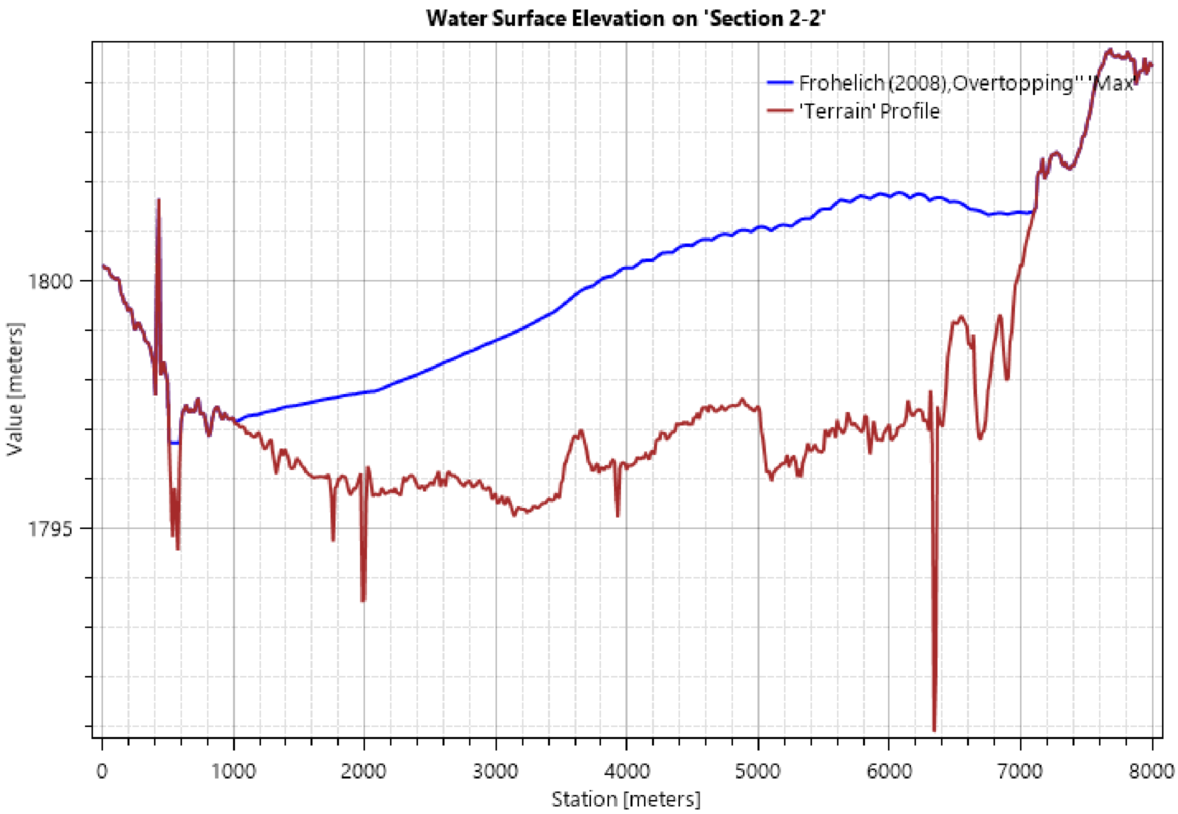


Figure 5-33: Maximum Water Surface Extent Boundary at Chainage 43+831(Frohelich, 2008)

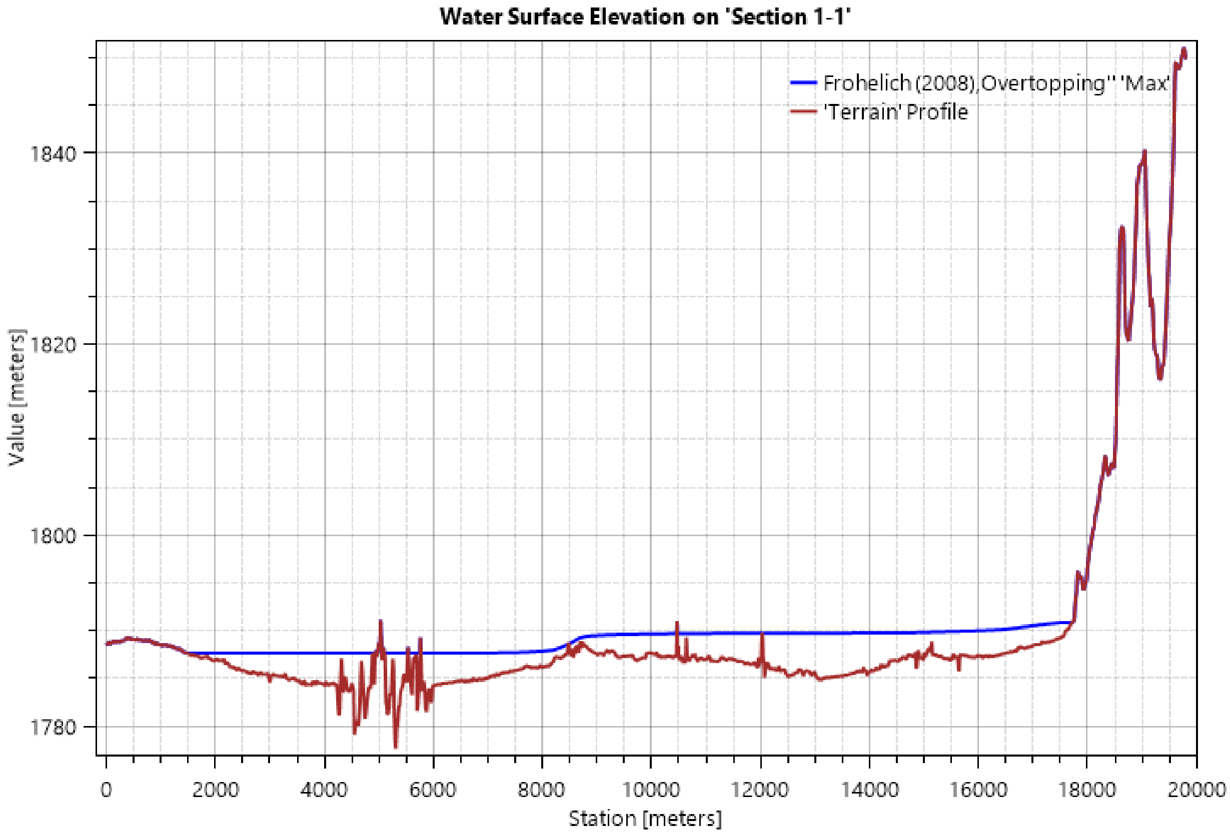


Figure 5-34: Maximum Water Surface Extent Boundary at Chainage 66+490(Frohelich, 2008)

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

The dam breach analysis, a requirement to the establishment of a dam safety plans and hazard management strategies, is a vital non-structural analysis which focuses on the post breach condition of the resulting flood. Hence through this thesis, the potential flow conditions and its potential consequence in the event of probable breach of the Ribb dam, located in the Amhara Regional state of Ethiopia, were analyzed under fourteen different scenarios. The Analysis was carried out using the Hydrologic Engineering Center River Analysis System (HEC-RAS V5.0.7) software developed by the US Army Corps of Engineers (HEC, 2016).

The thesis used both the deterministic and probabilistic approach for the modelling of the dam breach. The dam breach event has been modelled for both overtopping and piping failure modes and then modelling assessed the flood inundation characteristics. A 1.5 PMF inflow hydrograph and a base inflow hydrograph used as upstream boundary condition for overtopping failure mode and piping failure mode respectively and stage hydrograph of Lake Tana as downstream boundary condition for both modes of failure.

Briefly, the deterministic approach involves in the sensitivity analysis of the breach parameters and times (time to breach), performed by running all the five different non-physical breach parameters methods for each modes of failure as recommended by HEC RAS manual. The five analyzed methods are MacDonald and langridge-Monopolis, Frohelich (1995a), Frohelich (2008), Von thun and Gillette and Xu and Zhang.

A probabilistic approach is also used in which the uncertainty analysis is conducted and the exceedance probability peak discharge and their respective sampled breach parameters for a probability of 0.2%,1%, 5%, 10%, 50%, 90%, 95% and 99% are produced. In addition, the flood inundation mapping for the exceedance probability of 1% and 90% has been generated.

On the other hand, sensitivity analysis on manning's roughness coefficient 'n' is conducted. Such that to determine how the flood depth, velocity and inundation extent are affected based on changes in manning roughness coefficient 'n'. In this regard, the Manning's roughness coefficient 'n' value is increased and decreased by 30%.

In general, overtopping failure scenarios yielded significantly severe flow conditions at Ribb dam downstream as the cross-sectional area of the breach in overtopping failure scenarios was greater than that of in piping failure scenarios.

In summary, the following conclusion are drawn from the overall study,

- It revealed that the design inflow hydrograph, the PMF, can be safely evacuated through spillway without harming the dam. In fact, the dam is safe from breaching due to overtopping for an inflow hydrograph up to 1.5 PMF.
- Side slopes are found to be in a range of 0.5H: V both sides to 1.4H: V both sides, all are trapezoidal.
- Time of failure in hours is found to be in a range of 1.19hrs to 3.72hrs.
- Overtopping results of failures except breach time are obviously found to result in higher values of breach parameters when compared with piping modes of failure.
- The five non-physically empirical methods have resulted in peak flow values within a range of 67,570m3/s and 113,153m3/s during overtopping mode of failure; and between 22,269m3/s and 40,926m3/s for piping mode of failure.
- Frohelich (1995a) method of breach estimation has resulted in a peak flow of 113,153 m3/s and Frohelich 2008 has resulted second in a flow of 104,617 m3/s among the non-physically based methods during overtopping. However, due to the huge difference in reasonableness check, the author took Frohelich 2008. Moreover, Von thun and Gillette method has resulted in a peak flow of 40,926 m3/s and MacDonald and langridge-Monopolis has resulted in a minimum flow of 22,269 m3/s during piping failure mode simulation.
- The exceedance probability peak discharge for a probability of 0.2%, 1%, 5%, 10%, 50%, 90%, 95% and 99% are within the range of 70,607 m3/s and 110,463 m3/s. Similarly, their respective breach parameters also have the same trend.
- All cases of inundation were mapped and following the Frohelich 2008 method inundation map for overtopping modes of failure, the irrigation and drainage project area including the pickup weir, the Trunk roadway, all specified 25 kebeles form both woredas and portion of Wereta town will be severely inundated.

6.2 RECOMMENDATIONS

After analyzing the results obtained in this research and upon conclusions driven, the following recommendations are forwarded.

- As shown above in the result and discussion, the PMF inflow does not trigger overtopping failure, given it is adequately estimated overtopping failure event is likely very minimal. Therefore dam safety measures should focus on monitoring and surveillance of seepage and settlement to prevent potential failure event.
- The research work has shown that the time between the initiation of failure and area wide inundation is less than ½ day, thus, the time is very limited to initiate and implement emergency action plan for evacuation. Therefore, properly formed and staffed agency should be put in place for awareness, sensitization and execution. Furthermore, Emergency Action Plans (EPA) should be put in place in order to evacuate people potentially at risk in case of dam failure expediently.
- Based on results of the study, a research focusing on flood damage analysis can be done. Even if damage analysis has not been carried out given the intensity and extent of flooding and existence huge infrastructure investment in the area, for the future, flood insurance policy should be advocated for.
- Different researches on dam safety analysis should be conducted to monitor the wellbeing of the dam at different times and take remedial actions on identified problems.
- Flood protection dykes should be designed and implemented to protect dwelling areas and investment places currently located in a relatively high ground by assessing the flood map. Otherwise, wide scale adoption of dykes for flood protection will not provide safety in the flood plains, except for recurring events, given the computed maximum water surface elevation and associated depth.
- Overtopping failure is least likely for Ribb Dam; however, watershed management plan should not be overlooked as there is a large uncertainty in such analysis.
- Future research shall be done considering the geotechnical property of the dam to better understand and prevent potential risk of dam failure.

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