ADDIS ABABA UNIVERSITY INSTITUTE OF TECHNOLOGY

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

DAM BREACH MODELING AND DOWNSTREAM RISK ANALYSIS

(FOR RIBB DAM)

A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OF ADDIS ABABA UNIVERSITY IN PARTIAL FULFILLMENT OF THE DEGREE OF MASTERS OF SCIENCE IN CIVIL ENGINEERING (HYDRAULIC ENGINEERING).

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ABSTRACT

Analysis and simulation of embankment dam breach events and the resulting floods are critical to differentiating and reducing threats due to potential dam failures. Development of effective emergency action plans requires accurate prediction of inundation levels and the time of flood wave arrival at downstream critical locations.

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) can be to develop a dam failure model. HEC-GeoRAS was used to extract geometric information from a Digital Elevation Model (DEM) and then imported into HEC-RAS 4.1 where one dimensional Unsteady-flow simulation of the dam breach performed. The simulation results were mapped using the GIS extension tool HEC-GeoRAS on Arc Map. Inundation mapping of water surface profile results from dam failure models provides a level of the flood hazard and insight for emergency action plan.

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

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1. INTRODUCTION AND BACKGROUND

1.1 INTRODUCTION

Dams are important part of this nation's infrastructure, providing flood control, water supply, irrigation, hydropower, navigation, and recreation benefits. Despite their many beneficial uses and value, dams also present risks to property and life due to their potential to fail and cause catastrophic flooding. Or dam is an artificial barrier and its appurtenant works constructed for the purpose of holding water or any other fluid or dam is a barrier made of earth, rock , concrete or a combination of earth and rock that is constructed across a river for impounding or diverting the flow of water for the purpose of power generation, irrigation, water supply, flood control or recreation. Dams can be classified depending on different ways, among those are size and their uses are discussed below.

Jurisdictional Size Dam: is a dam creating a reservoir with a capacity of more than 100 acre-feet, or creates a reservoir with a surface area in excess of 20 acres at the high-water line, or exceeds 10 feet in height measured vertically from the elevation of the lowest point of the natural surface of the ground where that point occurs along the longitudinal centerline of the dam up to the crest of the emergency spillway of the dam. For reservoirs created by excavation, or where the invert of the outlet conduit is placed below the surface of the natural ground at its lowest point beneath the dam, the jurisdictional height shall be measured from the invert of the outlet at the longitudinal center line of the embankment or from the bottom of the excavation at the longitudinal center line of the dam, which ever is greatest.

Non-jurisdictional Size Dam: is a dam creating a reservoir with a capacity of 100 acre-feet or less and a surface area of 20 acres or less and with a height measured as 10 feet or less.

Minor Dam: is a jurisdictional size dam that does not exceed 20 feet in jurisdictional height and/or 100 acre feet in capacity.

Small Dam: is a dam with a jurisdictional height greater than 20 feet but less than or equal to 50 feet and/or a reservoir capacity greater than 100 acre-feet, but less than 4,000 acre-feet.

Large Dam: is a dam greater than 50 feet in jurisdictional height, and/or greater than 4,000 acre-feet in capacity.

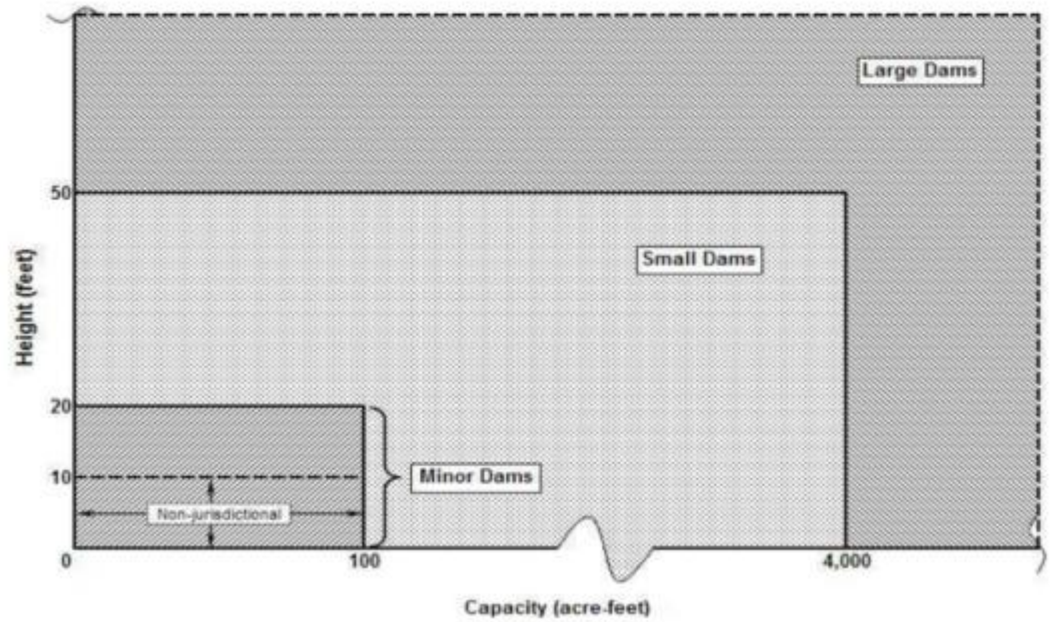


Figure 1.1 Dam Size Détermination

Depending on their uses dams can be classified as:

Diversion Dam: is a dam constructed for the purpose of diverting water from a natural watercourse into a canal, tunnel, ditch, or pipeline that typically impounds an insignificant volume of water, and for which the impacts of failure are not a significant public safety hazard.

Flood Control Dam: is a special purpose dam that is normally dry and has an un-gated outlet structure for the controlled release of water impounded during and subsequent to a flood event. The jurisdictional size and classification of the dam are determined using the height and capacity of the reservoir to the emergency spillway elevation, or using the elevation of the maximum routed water surface elevation if no emergency spillway is provided.

Depending on hazard during failure happens, dams can be classified as:

High Hazard Dam: is a dam for which loss of human life is expected to result from failure of the dam. Designated recreational sites located downstream within the bounds of possible inundation should also be evaluated for potential loss of human life.

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

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

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

Dam failures may occur due to a variety of causes such as a significant hydrologic event, seismic activity, operational error, and other deficiencies. If a dam breach occurs, an uncontrolled release of water impounded behind the structure will cause flooding in the downstream area.

Dam failure is the partial or complete collapse of the dam or its foundation, leading to uncontrolled release of water in the downstream areas by different causes (piping, overtopping, sliding or due to earth quake). Dam failure is followed by many catastrophic results or risks like, loss of life, property damage and so on.

To mitigate the risks due to dam failure, dam owners and regulators carefully analyze and inspect dams to identify potential failure modes and protect against them.

Dam break models were established to estimate the consequences of a hypothetical breach of a dam during flood flow conditions and normal operation under sunny day weather. Flood condition analyses are intended to estimate the required discharge capacity to pass the Inflow Design Flood (IDF) without overtopping the dam. Sunny day condition analyses are used to design a dam and its appurtenant to withstand against earth quakes or unforeseen events.

1.2. STATEMENT OF THE PROBLEM

A well-designed, constructed, and operated dam can reduce flood risk in areas downstream by temporarily impounding flood waters and attenuating the observed peak flood flows in exposed low lying areas, even if the dam is not specifically designed for flood mitigation. However, impounding water behind a dam also creates risk to downstream areas because of the potential for uncontrolled release of the reservoir pool caused by dam failure which could result in a peak flow discharge that greatly exceeds any possible natural flood event. There are several potential causes of dam failure including hydrologic, hydraulic, geologic, seismic, mechanical, and operational (FEMA, 2013).

One of the most common causes of dam failures is the inability to safely pass flood flows. Failures caused by hydrologic conditions can range from sudden failure, with complete breaching or collapse of the dam, to gradual failure, with progressive erosion and partial breaching (FEMA, 2013)

If a dam breach occurs, an uncontrolled release of water impounded behind the structure will cause flooding in the downstream area and affect the population and damages the economic resources. Nowadays, there are many embankment dams existing, under construction and planned to be constructed in Ethiopia. Therefore, in addition to proper design and construction method, the dam failure and risk analysis must be undertaken for the sake of safety.

Ribb Irrigation and Drainage Project Dam, an Earth rock-fill embankment dam with impervious clay core is now under construction. This dam is proposed to impound the water within the reservoir of capacity 234 million meter cube at its normal pool level. The main purpose of this dam is to retain, control and release this water for downstream irrigation fields without the disturbance of the downstream natural river flow system and for flood control. Unlike its benefits, its safety also should be considered for the consequence in case the failure may exist during and after the construction time. At the downstream of Ribb Dam, there is an irrigation field of 20,000 hectares 28 kilometers downstream of the dam on both sides of the river banks.

A town called Woretta and Kemkem woreda are also located near 40 km downstream. Even if the spillway of the proposed dam was designed to pass the inflow design flood, the dam breach modeling and downstream risk analysis for the worst hydrologic and non-hydrologic are important for the safety of the dam and to prevent the life loss and property damages that may happen in case the failure occurs.

Thus, for Ribb earth rock-fill dam, two dam breach scenarios were proposed to analyze the effect of the outflow breach hydrographs on the downstream floodplains. These scenarios are hydrologic (overtopping) and non-hydrologic (Sunny Day) or Piping failure.

The problem that needs to be solved in this paper is the proper modeling of Ribb Dam breach and the effect of the breach outflow flood on the downstream flood plain. Therefore, for the proposed dam breach modeling, prediction of breach outflow hydrograph and computer simulation to evaluate the dam failure and its impact to downstream area, which is the first phase of dam breach analysis, is discussed for dam failure scenarios.

1.3. OBJECTIVE

The main objective of this study is to model the Ribb Earth Rock-fill Embankment Dam breach and to analyze the consequences due to the dam breach/failure on the downstream floodplain.

The specific objectives are:

- To predict dam breach parameters using three breach parameter estimation methods
- To compare and discuss the results
- To select most appropriate method for Ribb earth rock-fill dam breach modeling case
- To calculate peak outflows for overtopping and piping failure approaches
- To route the model outflow hydrographs through the downstream floodplain.
- To prepare floodplain inundation maps
- To prepare an emergency action plan (EAP) to give a response in order to prevent or reduce the damages that may occur on downstream because dam failure.
- To make sensitivity analysis using Ribb dam and reservoir.

1.4 ORGANIZATION OF THE THESIS

This thesis is organized in Eight chapters.

Chapter one: Contains introduction and background, statement of the problem and objectives of the study.

Chapter two: Discusses the literature review on embankment dam breach modeling methods, breach characteristics and parameter estimation, and hydraulic modeling for the analysis of downstream routing the outflow hydrograph from dam breach, different researcher's literatures on dam breach modeling and outflow hydrograph routing, mapping flood inundation area, etc. .

Chapter three: Here in this chapter the methodology and steps to be taken to process the study well are discussed. And also the study area, data collection, hydraulic model development, etc. are discussed.

Chapter four: Dam breach parameter estimation and model setup are discussed. In this part, dam breach parameters, breach development time, peak breach outflow discharges are estimated and the river (Ribb) is represented in hydraulic model (HEC-RAS). Flood inundation area delineation mapping also presented in this chapter.

Chapter five: covers the results of the modeling and the discussions on the output results.

Chapter six : the sensitivity analysis of dam beach is discussed

Chapter seven:the emergency action plan preparation is presented.

Chapter eight: conclusions and recommendations are presented

2 LITERATURE REVIEW

2.1. INTRODUCTION

Dams are an important part of human beings now a days infrastructure by providing flood control, water supply, irrigation, hydropower, recreation benefits and many others. Despite their many beneficial uses and value, dams also present risks to property and life due to their potential to fail and cause catastrophic flooding. To mitigate these risks, dam owners and regulators carefully analyze and inspect dams to identify potential failure modes and protect against them.

Dam break models were established to estimate the consequences of a hypothetical breach of a dam during flood flow conditions and normal operation under sunny day weather. Flood condition analyses are intended to estimate the required discharge capacity to pass the Inflow Design Flood (IDF) without overtopping the dam. Sunny day condition analyses are used to design a dam and its appurtenant to withstand against earth quakes or unforeseen events. Many investigations have been conducted to develop methods used to predict the peak discharge from a breached embankment dam. Most of these investigations have used simple-regression analysis to relate the peak outflow through the breach to the depth of water behind the dam at failure, the volume of water behind the dam at failure, or the product of the depth and volume. As indicated in Table 1, the results of eleven (11) discreet investigations reported between 1977 and 1995 are presented to include the predictive expression, type of statistical curve fit, and number of case studies used in the analysis.

The variables in the relationships are: Q_p = peak outflow (cubic meters per second (m^3/s)), h_w = height of the water behind the dam at failure (m), h_d = height of the dam (m), S = reservoir storage at normal pool (m^3), and V_w = volume of the water behind the dam at failure (m^3).

It is apparent that each investigator used slightly different terms to describe the effective head and volume of water that created a breach through an embankment dam. Effective head has been represented as both the height of the water behind the dam (h_w) and the height of the dam (h_d). The volume of outflow through the breach has been represented as the volume of water behind the dam at failure (V_w) and the reservoir storage (S). Additionally, definitions of reservoir storage vary for each investigator. For example, Singh and Snorrason (1984) refer to the storage term as "reservoir storage at normal pool," and Costa (1985) describes volume as the reservoir volume at the time of failure. Costa's definition of volume does not include additional inflow during a flood and presumably could include "dead storage" beneath the breach invert. Arguably, the best term to represent storage would be a measurement of the volume of outflow through the breach during failure, but in many case studies this has not been reported. (FEMA P-946 /JULY 2013)

2.2 HISTORY OF DAM BREACH MODELING

Dam failure may occur due to many causes like significant hydrologic event, seismic activity, operational error and other deficiencies. If a dam breach occurs an uncontrolled release of water impounded behind the structure will cause flooding in the downstream area. In 1977, the NWS developed **DAMBRK**, a model to analyze the dam breach process and route peak breach outflows to determine inundation depths downstream of the dam. Between 1977 and the mid-1990s, a series of regression relations were developed to predict breach parameters and peak discharge from breached embankment dams. Statistical regression analyses such as MacDonald and Langridge-Monopolis (1984), USBR (1988), Von Thun and Gillette (1990), Dewey and Gillette (1993), and Froehlich (1995, 2008) were developed for use with empirical methods of evaluating dam failure and breach parameters. The breach modeling process was further advanced with the prediction of the reservoir outflow hydrograph and the routing of the hydrograph downstream through the use of two types of breach models: physical and empirical.

Physical models are based on physical laws and empirical relations governing flow and erosion. However, these types of models are not widely used in dam breach assessment because of lack of data to estimate breach erosion. The most notable model is the **NWS BREACH** (1985), a physically based mathematical model used to predict the breach characteristics and the discharge hydrograph emanating from a breached earthen dam. The model was developed by coupling the conservation of mass of the reservoir inflow, spillway outflow, and the breach outflow within the sediment transport capacity of the unsteady uniform flow along an erosion-formed breached channel (Fread, 2001; Wahl, 2004). (FEMA P-946 /JULY 2013)

Conversely, empirical models, also known as parametric models, are based on predetermined controlled input parameters for estimation of a resulting breach through regression equations. These models are based on vast study information for estimation of time-to-failure and ultimate breach geometry, which can then be used to simulate breach growth as a time-dependent linear process, computing breach outflow in a triangular or trapezoidal shape. Examples of one-dimensional empirical breach models include **NWS DAMBRK** (1988) and its successor NWS Flood Wave Dynamic Model (**FLDWAV**) (1998), HEC-1, HEC- HMS, and **HEC-RAS** (State of Colorado Department of Natural Resources, 2010; Wahl, 1997 and 2004).

HEC-1 and HEC-HMS are watershed modeling software capable of generating a breach hydrograph from predefined breach parameters (i.e., breach width, time of failure, etc.) input in the model. NWS DAMBRK/FLDWAV, an unsteady model, and HEC-RAS, capable of both steady-state and unsteady routing, are based on the St. Venant equations for flow computations, which generate a breach hydrograph and route the flow wave downstream. Both HEC-HMS and HEC-RAS are frequently updated by the USACE to include additional capabilities. HEC-RAS has the capability of interfacing with GIS for generation of inundation maps using predefined locations downstream of the breach event coupled with a flow quantity, water surface elevation, and travel time to the location.

Simplified methods for predicting peak breach discharge were developed using regression equations by Kirkpatrick (1977), Soil Conservation Service ([SCS]1981), Hagen (1982), USBR (1982), MacDonald and Langridge-Monopolis (1984), Singh and Snorrason (1984), Costa (1985), Evans (1986), and Froehlich (1995, 2008), each of whom relate the predicted peak discharge as a function of various dam and/or reservoir parameters developed from analyses of historical dam failures (Wahl, 2004).

Simplified methods used in breach hydrograph generation and downstream routing include the SCS Technical Release (TR) No. 66, Simplified Dam Breach Route Procedure (1985), and the NWS SMPDBK (1991). TR-66 presents a method for estimating the breach hydrograph and the peak flood flow at a predefined location, associated maximum depth of flow, and the time to peak flow using a simplified Attenuation-Kinematic (Att-Kin) flood routing method (SCS, 1985).

SMPDBK is a simplified model for predicting downstream flooding produced by dam failure. This program is still capable of producing the information necessary to estimate flooded areas resulting from dam-break floodwaters while substantially reducing the amount of time, data, and expertise required to run a simulation of the more sophisticated unsteady NWS DAMBRK, now called FLDWAV. SMPDBK is capable of predicting necessary information to estimate flooded areas resulting from a dam break, but does not account for backwater effects of additional downstream inflow.

In 2012, FEMA developed the Geo Dam BREACH toolset, which is based on the NWS SMPDBK model. Geo Dam BREACH includes an automated GIS-based mapping function for producing breach inundation mapping and FEMA non-regulatory products for dams. It also includes a semi-automated EAP function.

Recent developments in dam breach modeling have been concentrated in the area of two-dimensional hydraulic modeling for dam breach flood routing and on the erosion processes of dam failure in physically based modeling. Two-dimensional models with GIS integration are now common, allowing more sophisticated analyses. Two-dimensional models—such as the Decision Support System for Water Infrastructural Security (DSS-WISE) developed by the National Center for Computational Hydro science and Engineering (NCCHE) of the University of Mississippi,1 MIKE© software by DHI,2 and FLO-2D© by FLO-2D Software, Inc. 3—solve either full dynamic or simplified forms of conservative or non-conservative two-dimensional shallow water equations where as one-dimensional flow uses one-dimensional, cross-section-averaged shallow water equations. Two-dimensional model strengths are highlighted in unconfined alluvial fans where overland flow cannot be accurately modeled in a one-dimensional model *(*Altinakar, 2008; Wahl et al., 2008; Wahl, 2009 and 2010*).*

Researchers are focusing on developing a new generation of physically based hydraulic routing models to estimate the erosion processes associated with dam failure; understanding these processes better will facilitate more accurate determinations of downstream dam breach inundation mapping zones. Current models rely on the user to input parametric descriptions of the breach event and the model simulates flow through the breach as it enlarges at the specified rate (Wahl, et al., 2008). (FEMA P-946 /JULY 2013)

2.2.1 BREACHED DAMS IN HISTORY

Two dams were breached in British Columbia (BC). On June13, 2010, Testalinden Dam failed and caused a debris and mud torrent that demolished five downstream homes and farms, severed a main provincial highway and introduced significant quantities of sediment into fish bearing waters. Most recently, Mount Polley Mine tailing dam was breached and released five million cubic meters of water and tailing waste in local lakes and creeks area. On March 12, 2004 the Big Bay Dam embankment, of Lamar County, Miss. failed in the vicinity of the principal spillway 12 years after construction. The Big Bay embankment is approximately 576 m _1,890 ft_ long and 15.6 m _51.3 ft_ high. With the failure occurring at approximately normal pool, 17,500,000 m3 _14,200 acre-ft_ of water was released, inundating 23 km_14.3 mi_ of valley to depths of up to 10.0 m _33 ft_ from the dam to the Pearl River. In all, 104 structures were documented as being damaged or destroyed as a result of this failure. No human lives were lost.

By far the world's worst dam disaster occurred in Henan province in China, in August 1975, when the Banqiao Dam and the Shimantan Dam failed catastrophically due to the overtopping caused by torrential rains. Approximately 85,000 people died from flooding and many more died during subsequent epidemics and starvation; millions of residents lost their homes (Qing, 1997). This catastrophic event is comparable to what Chernobyl and Bhopal represent for the nuclear and chemical industries (McCully, 1996). In the Netherlands, in February 1953, a high–tide storm caused the highest water levels observed up to date and breached the dikes in more than 450 places, causing the death of nearly 1,900 people as well as enormous economic damage (Gerritsen, 2005) and many more. (FEMA P-946 /JULY 2013)

2.3 HAZARD CLASSIFICATION AND FAILURE SCENARIOS OF DAM BREACH

Downstream hazard is defined as: the potential loss of life or property damage downstream of a dam from flood waters released at the dam or waters released by partial or complete failure of the dam. It is descriptive of the setting in areas downstream of the dam and is an index of the relative magnitude of the potential consequences to human life and development should a particular dam fail.

The downstream hazard classification system adopted for use in Washington State is shown below in table. In determining the downstream hazard classification of a given project, hypothetical dam failures should be evaluated for two reservoir conditions, i.e. normal pool level(sunny day failure) and maximum storage elevation(flood conditions) **.**

The hazard potential classification of a dam, along with its size (height and capacity) classification, is used by United States, State agencies to regulate dam design and dam breach modeling. FEMA guidance recommends a three-step rating system that defines low-, significant-, and high-hazard potential classifications depending on the potential for loss of life, economic loss, and environmental damage resulting from a hypothetical dam failure. (FEMA, 2013)

Now a days the two primary dam breach study approaches used in dam breach analysis are an event-based approach and a risk-based approach. The event-based approach has been traditionally the most widely used for dam breach analysis. For the event-based approach, both a non-hydrologic “fair weather failure,” also referred to as a “sunny day failure,” and a specific hydrologic failure event, such as the Probable Maximum Flood (PMF), are usually established based on a dam’s hazard potential classification. . The failure scenarios generally show the worst scenarios when extremely catastrophic events occur.

Here in this study a risk-based approach is not used which is commonly used for dam design purposes to establish the Spillway Design Flood (SDF) or Inflow Design Flood (IDF) for a dam.

2.3.1 SUNNY DAY FAILURE (NON-HYDROLOGIC FAILURE)

This is a sudden dam failure that occurs during normal operations, with the water level at full supply and the water released causing the largest change in flows. It may be caused by foundation failure, earth quake or other such event. This scenario normally refers to internal erosion (piping) failure.

The sunny day failure examines a scenario when the reservoir is full up to the normal spillway level and there is no significant inflow in to the reservoir or in any parts of the adjoining catchments. This failure is important to consider, as the incremental damage caused by the dam failure is likely to be significant, since there were no other flood related processes occurred which have already been causing damage or forewarning to enable evacuation out of harm's away.

The sunny day failure is normally used to determine the potential impact category (PIC) rating for a dam, since the incremental losses from a sunny day failure are normally greater than the incremental losses associated with a rainy day failure.

The breach side slopes have been taken as 0.5h:1v based on case studies undertaken by Macdonald and Lagrange-Monopolis(1984) which is considered reasonable for a rock fill dam type. 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 as follows:

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

A breach at the normal pool elevation of the reservoir is used to estimate the volume and associated breach discharge that would result from a failure event during fair weather conditions. For an embankment dam, this type of event is modeled as piping/internal erosion failure, whereas for a concrete dam, this event is modeled as a monolith collapse resulting from sliding, foundation instabilities, or a seismic event.

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

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

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

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

Various Federal agency publications provide guidance for establishing the initial water surface elevation of a reservoir during a fair weather failure event. Each of these specified elevations is used to characterize different failure modes as well as the potential volume of the reservoir at the time of failure.

2.3.2 FLOOD INDUCED FAILURE (HYDROLOGIC FAILURE)

This is a dam failure resulting from a natural flood of a magnitude that is greater than what the dam can safely pass. This scenario normally refers to overtopping failure. 2. 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, according to State regulation could range from the 1-percent-annual-chance flood event (often called the 100-year flood) to a percentage of the PMF.

Many State and Federal agencies also allow the use of a risk-based approach to establish the SDF or IDF for dam design purposes.

2.4. MODES OF DAM FAILURE

Definition: The partial or complete collapse of the dam or its foundation, leading to uncontrolled release of water in the downstream areas.
Historically, all types of dams have experienced failures due to one or more type of event/loading. However by far the majority of dam failures that have occurred have been earthen dams caused by some level of flood. There are many mechanisms that can be the driving force of a dam failure. The following is a list of mechanisms that can cause dam failures: Flood event, piping/seepage (internal and underneath the dam), land slide, earthquake, foundation failure, equipment failure/malfunction(gates, etc), structural failure, upstream dam failure, etc.

Depending on the type of dam and site specific conditions, a dam may be susceptible to failure from multiple causes. Additionally, the breach shape and timing of a dam failure varies depending of the type of dam. For instance, concrete gravity dams tend to have a partial breach, as one or more monolith sections formed during dam construction fail, where as concrete arch dams tend to fail suddenly and completely(Canadian Dam Association, 2007b). In contrast, embankment dams do not usually have a complete or sudden failure, but rather tend to breach to the point where the reservoir is depleted or to where the breached materials resist erosion, such as at the dam foundation.
The most common causes of dam failure between January 1975 and January 2011 are summarized on the following table. Flood or Overtopping and piping or seepage was the most common causes of dam failures. .
Among these deriving forces resulting a dam to fail are piping and overtopping failures will be analyzed in this thesis.

Table 2-1: Causes of dam failures 1975-2011

Causes of Failure	Number of Dam Failures	Percentage of Dam Failures
Flood or Overtopping	465	70.9
Piping or Seepage	94	14.3
Structural	12	1.8
Human Related	4	0.6
Animal Activities	7	1.1
Spillway	11	1.7
Erosion/Side/Instability	13	2.0
Unknown	32	4.9
Other	18	2.7
Total number of dam failures	656	

The many causes of dam failures are commonly summarized as follows. Hydrologic, geologic, structural, seismic and human influencing.

1. Hydrologic Failure Modes

Hydrologic dam failures are induced by extreme rain fall or snow melt events that can lead to natural floods of variable magnitude. The main causes of hydrologic dam failure include overtopping, structural overstressing and surface erosion due to high velocity flow and wave action.

- Overtopping failure

This type of failure is among the most common failure modes for embankment dams. It may be caused by when inflows higher than the design inflow, mistake in operation of spillway or an outlet structure, inadequate carrying capacity of spillways, settlement of the dam or landslides in to the reservoir. According to the national performance of dams program in USA (NPDP ,2007)

,245 of 256 dam failure events recorded in the USA during the year 1994 happened due to high inflow discharges. Any embankment dam will fail if the spillway capacity is too small and flood waters rise high enough to flow over the top of the dam for a considerable amount of time. In August 1979, a flood three times larger than the design flood results failure of the Machhull dam in India causing more than 200 injuries (Hagen, 1982). Overtopping failure may also occur when a reservoir's out let system is not functioning properly, thereby raising the water surface elevation of the dam.

Overtopping of a dam as a result of flooding is the most common failure mode for embankment dams.

Dam failure begins when appreciable amounts of water begin flowing over or around the dam face and begin to erode the face of the dam. For embankment dams, the failure typically begins at a point on the top of the dam and expands in a generally trapezoidal shape.

FIGURE 2.1:WATER OVERTOPPING THE MORGAN FALLS DAM

2. Structural Overstressing Of Dam Components

Higher loading conditions are typically found in dams where the reservoir elevation is increased due to a hydrologic event. While the dam may not be overtopped, the surcharge may be increased overstressing the dam's structural components. This overstressing may then result in an overturning failure, sliding failure or failure of specific components of the dam. Embankment dams may be at risk when water surface increases resulting seepage rates that exceeds the design seepage control measures for the dam.

3. Surface Erosion from High Velocity and Wave Action

Surface erosion can occur along earthen spillways, the upstream or downstream embankment slopes or along other appurtenant structure inlet and outlet channels. Surface erosion is primarily caused by high velocity runoff, reservoir wave action and ice action. High flow velocities may cause head cutting along spillway sides that can progress towards the spillway crest, eventually leading to a full dam breach (FEMA, 2004a).

4. Geologic Failure Modes

These include piping and internal erosion as well as slope instability. For embankment dams, geologic failures are typically cause by long term seepage of water stored in the reservoir, the water seeps through the dam or the foundation and abutments weakening the embankment over time. If seepage is uncontrolled it may lead to internal erosion or piping of the embankment materials with in the dam.

A geologic failure may also result from inadequate geotechnical design of the embankment and foundation, inadequate seepage controls or increased load situations like rapid increase or drawdown of water level due to a hydrologic event, landslide, earthquake or wave action.

- Piping failure

Piping failure is a failure mode due to water penetrating through the dam body carrying with its small particles of dam material continuously widening the gap.Typically piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid. Once the erosion reaches the reservoir it can enlarge and cause catastrophic dam failure.

Piping failures typically occur only in earthen dams. The failure begins when water naturally seeping through the dam core, increases in velocity and quantity to begin eroding fine sediments out of the soil matrix. If enough material erodes, a direct piping connection may be established from the reservoir water to the dam face. Once such a piping connect is formed, it is almost impossible to stop a dam from failing. Piping failures begin at a point in the dam face and expand as a circular opening. When the circular opening reaches the top of the dam, it continuous expanding as a trapezoidal shape. The water flow through the circular opening is modeled as orifice flow, but in the second stage (as trapezoidal in shape) is modeled as weir flow. Piping can also occur along conduits, outlet works and abutments. An example of piping failure is shown below.

Internal Erosion: Similar to piping, internal erosion is the occurrence of erosion where two adjacent zones interface within the embankment or at the contact between the embankment and foundation. Internal erosion is differentiated from piping is, internal erosion originates internally, whereas piping originates externally. When voids of the material in to which seepage is flowing are larger than a critical size required to retain the particles, the particles of the up gradient material can be transported in to or through the adjacent material, there by resulting in internal erosion.

5. Structural Failure Modes

Structural failure modes can occur when there is a failure of a critical dam component, poor construction materials, inadequate maintenance and repair, or gradual degradation and weakening over time. (FEMA, 2010). Additionally, structural failure may be inter related with other modes of failure. For example, structural failure of the main embankment may be related to internal piping, or a critical dam component could fail due to over-stressing during a flood event.

FIGURE 2.2 STRUCTURAL FAILURE:

6. Seismic Failure Modes

Earth quakes are another important cause of dam failures, especially in seismic zones of the United States. Seismic failures are generally related to either ground movement or liquefaction. Ground movements may cause a dam to shift, settle, or crack in ti an undesirable configuration that prevents the dam from performing as designed.

For embankment dams, two failure scenarios are typically considered in dam breach analyses: liquefaction and seismic-induced piping. Earthquakes can cause extreme stress on a dam, and liquefaction can occur when soils are loaded causing the soil to transfer from a solid to liquefied state. Soil liquefaction can cause a dam to fail almost instantaneously or it can cause slumping that exposes the dam crest to overtopping and subsequent erosion. Seismic-induced piping can occur due to internal cracking caused by ground motions of an earthquake (United States Department of the Interior [USDOI], 2011).

Failure mechanisms due to seismic activities include:

- Slope instability
- Permanent deformations
- Fissures or cracking
- Differential settling
- Rupture of principal spillway outlet pipeline
- Liquefaction (Australian National Committee on Large Dams Incorporated, 2000; Canadian Dam Association, 2007c).
- Seismic failures are occurred in seismic zones

7. Human Influenced Failure Modes

Human-influenced dam failure incidents can be related to improper design or maintenance, miss operation including scheduled volume releases, or terrorist acts.

Maintenance: Specialists should prepare maintenance checklists indicating the maintenance procedures and protective measures for each structure and for each piece of operating, communications, and power equipment, including existing monitoring systems. Special attention should be given to known problem areas.

Special instructions should be provided for checking operating facilities following floods, earthquakes, tornados, and other natural phenomena.

Maintenance procedures include preventive measures such as painting and lubrication as well as repairs to keep equipment in intended operating condition, and minor structural repairs such as maintaining drainage systems and correcting minor deterioration of concrete and embankment surfaces. The design staff should be apprised of any significant maintenance work. (FEMA 93, *Federal Guidelines for Dam Safety* (FEMA, 1979; reprinted in 2004).

Miss operation: Miss operation, as defined in FEMA 541, *Embankment Dam Failure Analysis* (FEMA, 2005), is “the sudden or accidental and/or non-scheduled operation of a water retaining element of a dam that releases stored water to the downstream channel in an uncontrolled manner. Miss operation also includes the deliberate release of floodwater because of an emergency (USDOI, 2011).Situation, but without the issuance of a timely evacuation warning to the downstream interests… [It] also includes the inability to operate a gate in an emergency, a condition that could lead to over topping of the dam and potential breach."

Scheduled volume releases: The release of reservoir volume is a common practice for maintenance purposes, and to provide additional flood storage volume in a reservoir in anticipation of an extreme flooding event. The rapid release of reservoir volume in an upstream dam may result in dam over-topping at a downstream dam, resulting in dam failure. A rapid release of storage volume in a reservoir may also result in a rapid draw-down and a geologic failure. Improper releases of storage volume may result in a dam failure.

Terrorist incidents: Terrorist activities can range from purposeful miss operation of the dam to physical attacks on the structure itself. Two common scenarios are typically considered when analyzing human-influenced dam failure: rapid failure of spillway gates, and a lowering of the dam crest. For an embankment dam, the rapid lowering of the dam crest could subjugate the dam to overtopping and subsequent erosion.

2.5 DAM BREACH MECHANISMS AND ESTIMATING DAM BREACH PARAMETERS

A key element for calculating a dam breach hydrograph for a specific dam involves estimating the dam breach parameters for dam breach modeling related to the geometry and timing (e.g., width, depth, shape, and time of failure) of the breach formation.

It has been noted by several sources that the selection of breach parameters for modeling dam breaches contain the greatest uncertainty of all aspects of dam failure analysis and therefore a careful evaluation and understanding of the associated breach parameters is necessary (Wurbs, 1987; USBR, 1998; Wahl, 2004; Gee, 2008, etc.).

A number of methods are available for estimating breach parameters for use in dam breach studies. Since the selection of the breach parameters is specific to each dam, guidance is provided describing methods currently applied by dam safety professionals without recommending a standardized method.

2.5.1. BREACH PARAMETERS DEFINITIONS

Breach formation time (also time-to-failure) – The duration of time between the first breaching of the

upstream face of the dam (breach initiation) and when the breach has reached it full geometry.

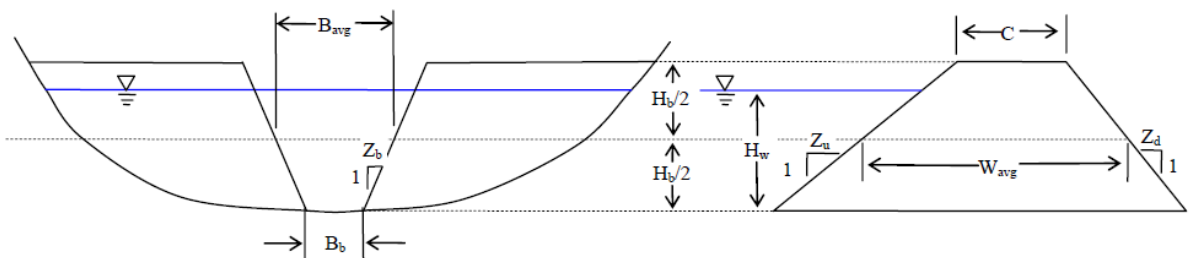


FIGURE 2.3:BREACH PARAMETERS DEFINITION SKETCH

- **Breach depth (also breach height)** – The breach depth is the vertical extent of the breach measured from a specific elevation to the invert of the dam breach.
- **Breach width** – The breach width is the average of the final breach width, typically measured at the vertical center of the breach.
- **Breach side slope factor** – The breach side slope is a measure of the angle of the breach sides represented as X horizontal to 1 vertical (XH: 1V).

A dam breach usually occurs in two distinct phases starting with the breach initiation followed by the breach formation.

Breach initiation: During the breach initiation phase, flow through the dam is minor and the dam is not considered to have failed. It may be possible to prevent a dam breach during this phase if flow is controlled.

Breach formation: Breach formation (defined above) begins when the flow through the dam has increased and progressed from the upstream face to the downstream face of the dam, is uncontrolled, and will result in the failure of the dam.

Overtopping Failures

Overtopping failures can occur very differently depending on the composition of the dam. Perhaps the simplest overtopping failure to discuss is ***failure of a cohesive soil embankment***. According to a study by Ralston (1987), a small head cut typically forms on the downstream face of a cohesive soil embankment and progresses upstream as shown in Figure below.

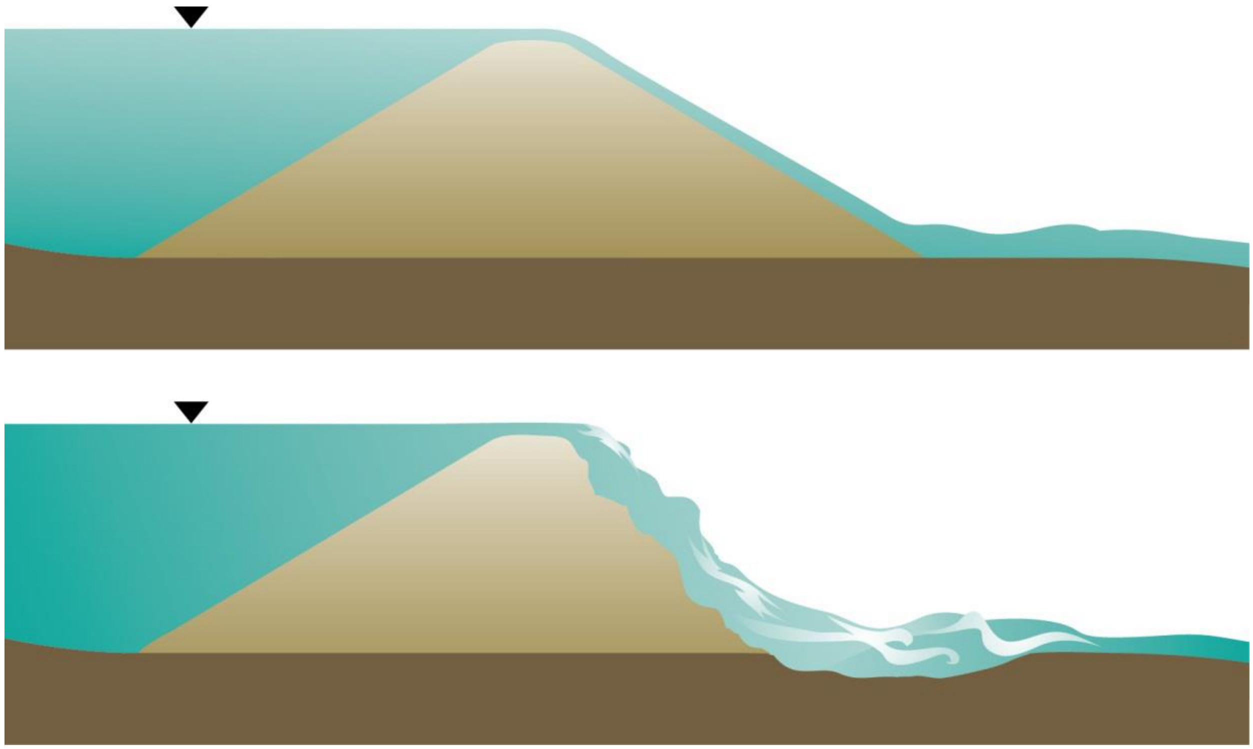


FIGURE 2.4:EROSION ON THE DOWNSTREAM FACE OF A COHESIVE SOIL EMBANKMENT DAM

The breach is considered to begin when erosion occurs across the width of the dam crest. After the breach initiates at the top of the dam crest, it enlarges to its ultimate extent. If there is no physical reason to believe the embankment would fail at a certain location, the breach should be modeled as initiating at the maximum section typically located at the centerline of the downstream main channel.

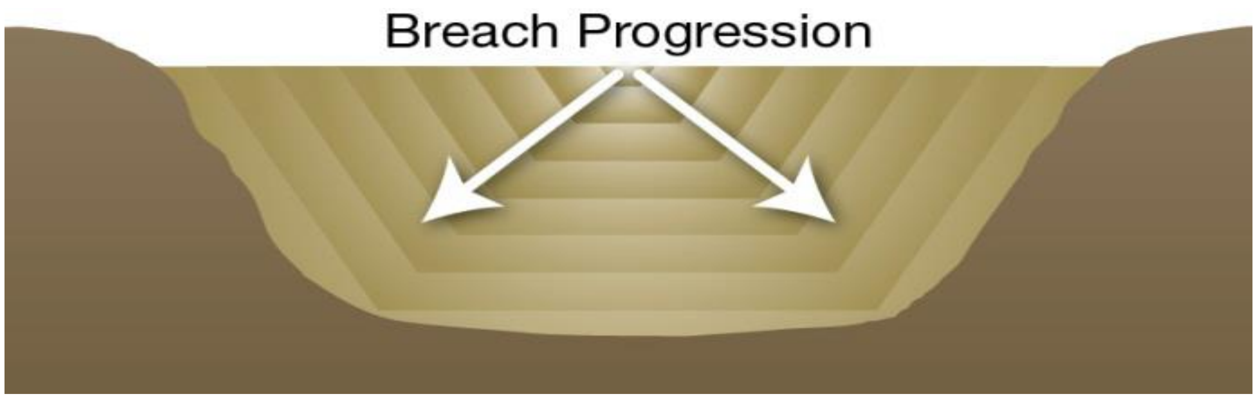
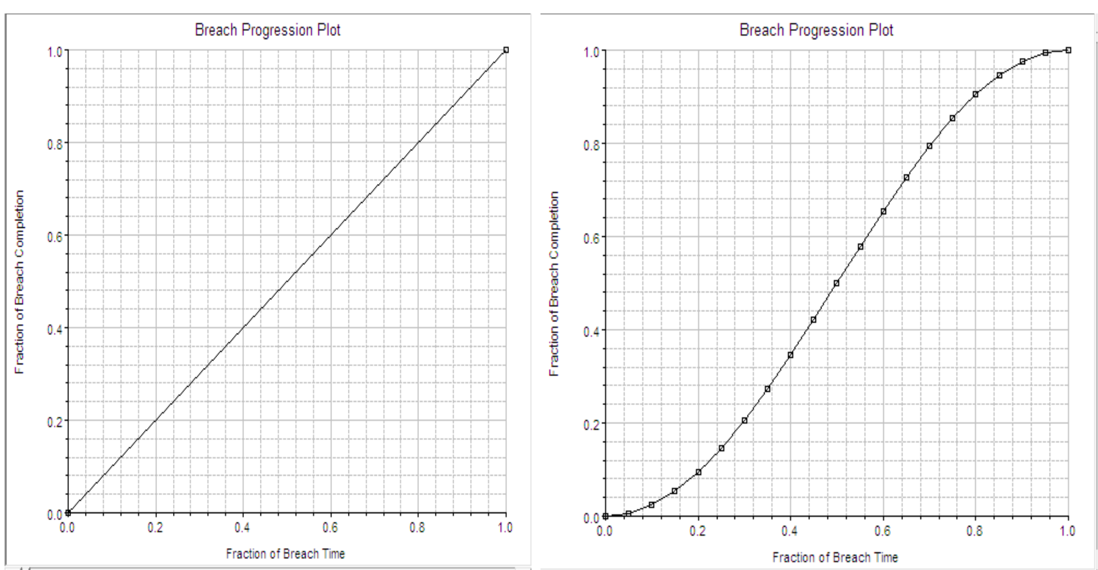


FIGURE 2.5:OVERTOPPING TRAPEZOIDAL BREACH PROGRESSION

The breach may stop growing when the reservoir has emptied and there is no more water to erode the dam or the dam has completely eroded to the bottom of the reservoir or has reached bedrock (Gee, 2009). The breach progression may be modeled as either a linear progression or a sine wave progression: (FEMA P-946 /JULY 2013)

• **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



1) Linear

2) Sine wave

FIGURE 2.6 BREACH PROGRESSION

Piping/ Internal Erosion

Piping and internal erosion occurs when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam, leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid (refer also to Section 4). Water flow through the embankment will appear muddy as erosion increases. Once the erosion reaches the reservoir, the piping hole can enlarge and cause the dam crest to collapse. The figure below shows a schematic of a fully formed piping hole.

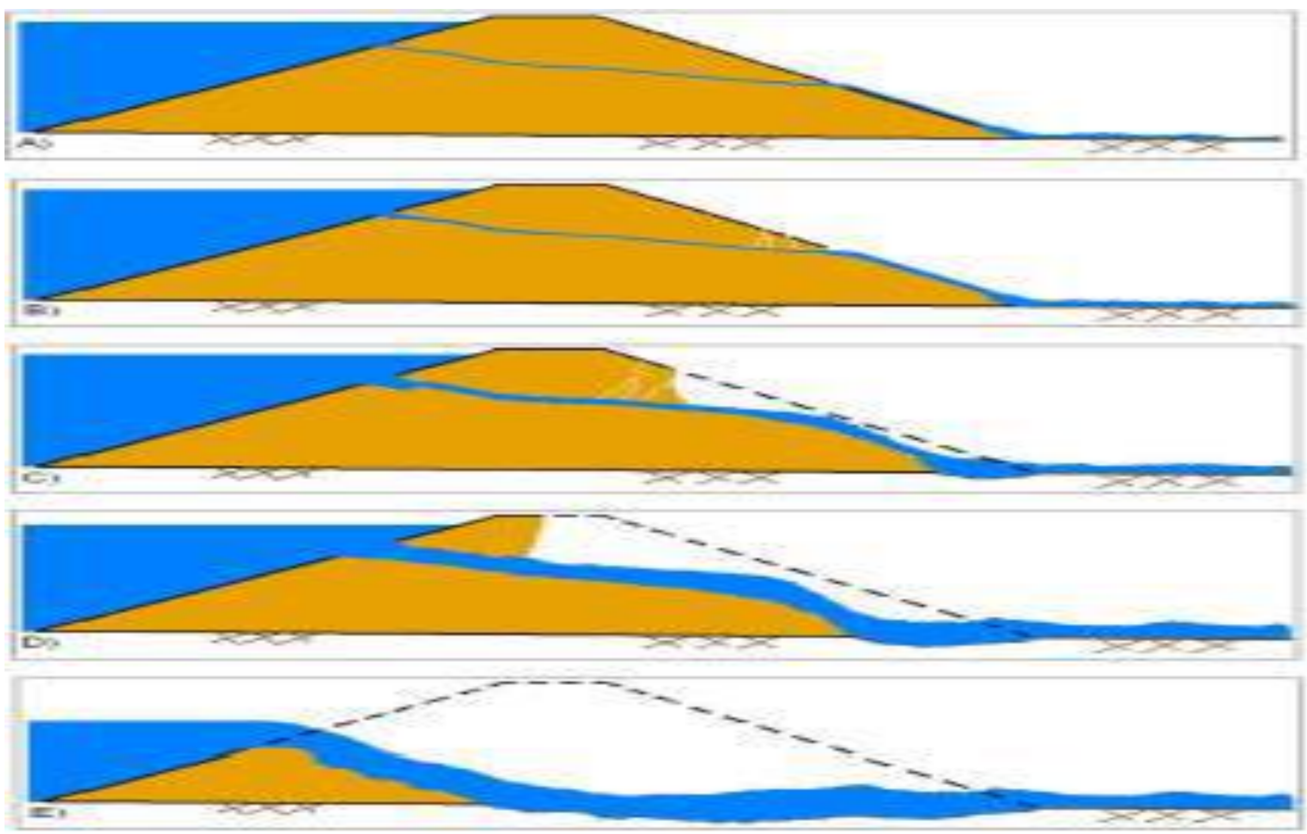


FIGURE 2.7:BREACH PROCESS FOR PIPING MODE OF FAILURE

Piping failures are typically modeled in two phases, before and after the dam crest collapses. Water flow through the piping hole is modeled as orifice flow before the dam crest collapses and as weir flow after the dam crest collapses. For small dams constructed from cohesive soils, it is possible for the reservoir to completely empty before the dam crest collapses (State of Colorado Department of Natural Resources, 2010).

There are several possible options to identify the breach initiation time. For breaches associated with a hydrologic event, the initiation can be considered to begin when the reservoir water level reaches a certain elevation or after the water level has exceeded a certain elevation for a specified duration. Here in this study, piping and overtopping failures will be analyzed.

TABLE 2.2:DAM BREACH WEIR AND PIPING COEFFICIENTS

Dam type	overflow/weir Coefficients	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

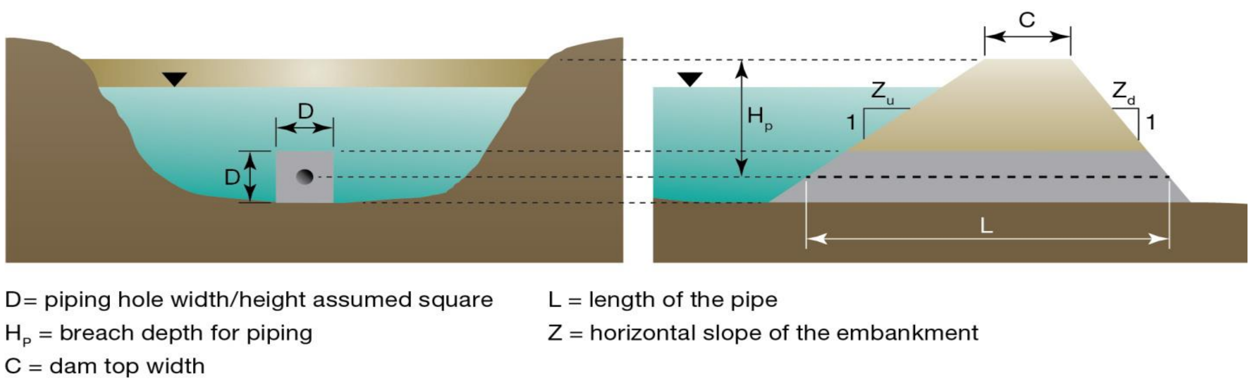


FIGURE 2.8:SCHEMATIC OF PIPING HOLE

2.5.2 BREACH PARAMETER ESTIMATION METHODS

According to dam breach modeling analysis methods overview made by Tony L. Wahl (Wahl T. L., 2010), three principal strategies for dam-breach flood modeling have emerged since the 1970s. The first strategy was to predict the breach outflow hydrograph directly and then use one of the available routing models to route that flood downstream (Wahl T. L., 2010) so that flooding consequences could be determined. The second approach was to parameterize the breach so that its evolution through time could be described in relative simple mathematical terms, allowing the breach outflow hydrograph to be determined by combining the description of the breach development with a weir equationor other appropriate model for simulating the hydraulic performance of the breach opening. Typical breach parameters determined were the maximum breach size, rate of breach development (or total time needed for full breach development). In this second approach, breach parameters could be determined by several different means externally to the flood routing model, but determination of the breach outflow hydrograph took place in the routing model. The third approach is to use a combined model that simulates specific erosion processes and the associated hydraulics of flow through the developing breach to yield a breach outflow hydrograph. Early models that took this approach were run separately from flood routing models, with the breach outflow hydrograph provided as input to the routing model. There is work being done now to integrate breach modeling and flood routing capabilities into a single model.

According to the State of Colorado Guidelines on Dam Breach Analysis, the four critical elements of any breach analysis are: breach parameter estimation (i.e. breach size/shape and time of failure), breach peak discharge and breach hydrograph estimation, breach flood routing, and estimation of the hydraulic conditions at critical locations. The most commonly used approaches for these required elements of the analysis are Comparative Analysis, Physically Based Erosion Models, Parametric Regression (Empirical) Equations and Predictor Regression Equations.

2.6.1. COMPARATIVE ANALYSIS

This method compares a given dam of interest with those in a database of well documented volumes are compared with a list of similar sized dams that have failed. Dam breach parameters and peak discharges reported from the failure case histories of similarly configured dams are then directly applied to the dam being analyzed. If the dam under consideration is very similar in size and construction to a dam that failed, and the failure is well documented, appropriate breach parameters or peak outflows may be determined by comparison.

2.6.2. PHYSICALLY BASED EROSION MODELS

Since the 1960s there have been numerous developments of physically-based, numerical dam breach models. In 1965, the first breach model was proposed (Cristofano, 1965), pioneering the development by others of physically based models BRDAM (1977), Dam Break Forecasting Since the 1960s there have been numerous developments of physically-based, numerical dam breach models. In 1965, the first breach model was proposed (Cristofano, 1965), pioneering the development by others of physically based models Model (DAMBRK) (1977), Breach Erosion of Earth-Fill Dams and Flood Routing (BEED) (1985), and BREACH (NWS, 1988).

Currently, the NWS BREACH model is a well-known and commonly applied physically based model developed by a Federal agency. The NWS BREACH model was developed to more realistically simulate breaches initiated by overtopping or piping in an embankment dam.

These methods predict the development of an embankment breach and the resulting breach outflows using an erosion model based on principles of hydraulics, sediment transport, and soil mechanics.

2.6.3. PARAMETRIC REGRESSION (EMPIRICAL) EQUATIONS

Parametric regression equations are empirically derived using case study information to estimate the time-to-failure and ultimate breach geometry, then simulate breach growth as a time-dependent linear process to compute breach outflows using principles of hydraulics. Numerous equations to predict breach parameters have been developed based on analyses of case studies. Table below, adapted from DSO-98-004 (USBR,1998), provides the most common parametric regression equations developed based on information from case studies of historic dam failures available.

These equations, developed from case study information, are used to estimate the time-to-failure and ultimate breach geometry. The breach can then be simulated to proceed as a time-dependent linear process with the computation breach outflows using principles of hydraulics.

These equations have been used on several dam break studies and have been found to give a reasonable range of values for earthen, earthen with a core wall (i.e. clay), and rock fill dams. The following is a brief discussion of each equation (USACE, 2014). The relationships between dam breach parameters and the regression equations were developed from analyses of historical dam failures. Wahl (2004) conducted a literature review of breach parameter equations, including 16 peak breach outflow equations, which are regression relations that predict peak outflow as a function of various dam and/or reservoir parameters. Table below presents the level of error for each of the 16 empirical prediction equations evaluated in his study. These empirical equations are typically used for Tier 1 reconnaissance level studies and are rarely used in detailed breach assessments.

In 2010, Wahl discussed the application of empirical methods to calculate breach flow and concluded that the MacDonald and Langridge-Monopolis (1984), USBR (1988), Von Thun and Gillette (1990), and Froehlich (1995, 2008) methods are the most commonly used, empirically derived equations for predicting peak breach flow and three of these methods are also used in this study.

Froehlich (1995a): Froehlich utilized 63 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rock fill data sets to develop as set of equations to predict average breach width, side slopes and failure time. The data that Froehlich used for his regression analysis had the following ranges.

- Height of the dams: 3.66-92.96 meters
- Volume of water at breach time: 0.0130 – 660.0 m3 x 106

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

$B_{avg} = 0.1803\ K_o\ V_w^{0.32}\ h_b^{0.19}$

$Tf = 0.00254\ V_w^{0.53}\ h_b^{-0.9}$

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

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

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

- Height of the dams: 3.05 - 92.96 meters
- Volume of water at breach time: 0.0139 – 660.0 $m^3 \times 10^6$

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

$$Bave = 0.27\, K_o Vw^{0.32} hb^{0.04}$$

$$T_f = 63.2(V_w/gh_b^2)^{0.5}$$

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 should be:

1H: 1V for overtopping failures

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

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

MacDonald and Langridge-Monopolis (1984): MacDonald and Langridge-Monopolis utilized 42 data sets (Predominantly Earth-fill dams, Earth-fill dams with a clay core, rock- fill dams) to develop a relationship for what they call the "Breach Formation Factor". The Breach Formation Factor is a product of the volume of water coming out of the dam and the height of water above the dam. MacDonald and Langridge-Monopolis then related the breach formation factor to the volume of material eroded from the dam's embankment. The data that MacDonald and Langridge-Monopolis used for their regression analysis had the following ranges.

- Height of the dams: 4.27 - 92.96 meters
- Breach outflow volume: 0.0037 – 660.0 m3 x 106

The following is the MacDonald and Langridge-Monopolis equation for the volume of material eroded and breach formation time, as reported by Wahl (1998):

<u>For earth-fill dams:</u>

Veroded = 0.0261 (Vout * hw) $^{0.769}$

Tf = 0.0179 (Veroded) $^{0.364}$

<u>For earth-fill with clay core or rock-fill dams:</u>

Veroded = 0.00348 (Vout * hw) $^{0.852}$

Where: Veroded = volume of material eroded from the dam embankment (cubic meters)

Vout = volume of water that passes through the breach (cubic meters)

hw = depth of water above the bottom of the breach (meters)

Tf = breach formation time (hours)

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

The base width of the breach can be computed from the dam geometry with the following equations:

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

Veroded = 0.00348 (Vout *hw) $^{0.852}$

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

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 (0.5H : 1V)

Von Thun and Gillette (1990): Von Thun and Gillette used 57 dams from both the Froehlich (1987) paper and the MacDonald and Langridge-Monopolis (1984) paper to develop their methodology. The method proposes to use breach side slopes of 1.0H: 1.0V, 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
- Volume of water at breach time: 0.0037 – 660.0 m3 x 106

The Von Thun and Gillette equation for average breach width is:

Bave = 2.5hw + Cb

Where: Bave = average breach width (meters)

hw = depth of water above the bottom of the breach

Cb = coefficient, which is a function of reservoir size

Von Thun and Gillette developed two different sets of equations for the breach development time. The first set of equations shows breach development time as a function of water depth above the breach bottom:

Tf = 0.02 hw + 0.25 (erosion resistant)

Tf = 0.015 hw (easily erodible)

Where: Tf = breach formation time (hours)

hw = depth of water above the bottom of the breach (meters)

The second set of equations shows breach development time as a function of water depth above the bottom of the breach and average breach width:

Tf = Bavg/4hw (erosion resistant)

Tf = Bavg/4hw+61 (easily erodible)

Where :hw = height of water

Bavg = average breach width

TABLE 2.3:VALUES OF COEFFICIENT CB

Reservoir Size (m^3)	C_b (m)
<1.23 x10^6	6.1
1.23 x 10^6 - 6.17 x 10^6	18.3
6.17 x 10^6 - 1.23 x 10^7	42.7
>1.23 x 10^7	54.9

TABLE 2.4:PUBLISHED PARAMETRIC REGRESSION EQUATIONS FOR PREDICTING BREACH PARAMETERS

Reference	Number of Studies	Relations Proposed(S,I units, meters, M^3/s, hours)
Froehlich (1995)	63	Bavg = $0.1803KoV_w^{0.32}h_b^{0.19}$ Tf = $0.00254V_w^{0.53}h_b^{(-0.9)}$ K_o = 1.4 for overtopping , 1.0 for piping
Froehlich (2008)	74	Bavg = $8.239\ K_oV_W^{0.32}h_b^{0.04}$ Tf = $63.2\ (V_w/gh_b^2)^{0.5}$ K_o= 1.3 for overtopping, 1.0 for piping
Von Thun and Gillette (1990)	57	B, Z, Tf guidance

Macdonald and Langridge - Monopolis (1984)	42	Earth- Fill dams: $V_{er} = 0.0261(V_{out} x h_w)^{0.769}$ (best-fit) $Tf = 0.0179 (V_{er})^{0.364}$ (upper envelope) Non-Earth-fill dams: $V_{er} = 0.00348(V_{out} x h_w)^{0.852}$ (best fit)

TABLE 2.5:EMPIRICAL EQUATIONS FOR ESTIMATING PEAK DISCHARGE AND ASSOCIATED ERROR

Reference	Equation	Number of case Studies		Mean prediction error (local cycles)	Width of uncertainty band, ±2S_e	Prediction interval around hypothetical predicted value of 1.0
		Before outlier exclusion	After outlier exclusion			
Peak flow equations						
Kirkpatrick (1977)	$Q_p = 12.68(h_w + 0.3)^{25}$	38	34	−0.14	±0.69	0.28–6.8
SCS (1981)	$Q_p = 16.6(h_w)^{1.85}$	38	32	+0.13	±0.50	0.23–2.4
Hagen (1982)	$Q_p = 0.54(S - h_d)^{0.5}$	31	30	+0.43	±0.75	0.07–2.1
Bureau of Reclamation (1982)	$Q_p = 19.1(h_w)^{1.85}$ envelope eq.	38	32	+0.19	±0.50	0.20–2.1
Singh and Snorrason (1984)	$Q_p = 13.4(h_d)^{1.89}$	38	28	+0.19	±0.46	0.23–1.9
Singh and Snorrason (1984)	$Q_p = 1.776(S)^{0.47}$	35	34	+0.17	±0.90	0.08–5.4
MacDonald and Langridge-Monopolis (1984)	$Q_p = 1.154(V_w h_w)^{0.412}$	37	36	+0.13	±0.70	0.15–3.7
MacDonald and Langridge-Monopolis (1984)	$Q_p = 3.85(V_w h_w)^{0.411}$ envelope eq.	37	36	+0.64	±0.70	0.05–1.1
Costa (1985)	$Q_p = 1.122(S)^{0.57}$	35	35	+0.69	±1.02	0.02–2.1
Costa (1985)	$Q_p = 0.981(S - h_d)^{0.42}$	31	30	+0.05	±0.72	0.17–4.7
Costa (1985)	$Q_p = 2.634(S - h_d)^{0.44}$	31	30	+0.64	±0.72	0.04–1.22
Evans (1986)	$Q_p = 0.72(V_w)^{0.53}$	39	39	+0.29	±0.93	0.06–4.4
Froechlich (1995)	$Q_p = 0.607(V_w^{0.295} h_w^{1.24})$	32	31	−0.04	±032	0.53–2.3

2.6.4. PREDICTOR REGRESSION EQUATIONS

Predictor regression equations are empirically developed equations used to estimate peak discharge based on actual case study data. These equations are used as a prediction method to determine a reasonable outflow hydrograph shape.

These equations estimate the dam breach peak discharge empirically based on case study data of peak discharge and hydrograph shape

Table below presents the empirical relationships developed by various authors for predicting peak breach discharge. The equations presented in Table below were adapted from DSO-98-004 (USBR, 1998). These equations are based on case study data used to develop empirical equations relating peak breach outflow to dam height and/or reservoir storage volume. The predictor regression equations provide an alternative method of computing the dam breach discharge; they can be used instead of determining breach parameters and then using a hydrologic-hydraulic model to compute the breach hydrograph.

TABLE 2.6:PREDICTOR REGRESSION EQUATIONS FOR PREDICTION OF PEAK BREACH FLOW (FEMA)

Reference	Case Studies	Relations Proposed
MacDonald and Langridge-Monopolis (1984)	42	$V_{er} = f(V_{out} * h_w)$ $T_f = f(V_{er})$ $Q_p = f(V_{out} * h_w)$
Von Thun and Gillette (1990)	57	*Z*-guidance $B_{avg} = f(h_w * S)$ $T_f = f(h_w, errosion\ resistance)$
Froehlich (2008)	74	$Q_{max} = 3.1 B_{avg} H_w^{1.5} \left(\frac{\gamma}{\gamma + T_f \sqrt{H_w}} \right)^3$

2.7.DAM BREACH ANALYSIS STUDY APPROACHES

The two primary dam breach study approaches used by State governments and Federal agencies are an event-based approach and a risk-based approach.

2.7.1. EVENT BASED APPROACH

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

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

The greatest advantage to using an event-based approach is that it is a direct approach, 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.event for low-hazard dams up to the PMF for high-hazard dams is selected based on the potential for loss of life due to a dam failure or for significant economic and environmental losses.

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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

2.7.2 RISK-BASED (CONSEQUENCES-BASED) APPROACH

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

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

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

Inflow Design Flood and the Incremental Hazard Evaluation

IDF is “the flood flow above which the incremental increase in water surface elevation downstream due to failure of a dam or other water retaining structure is no longer considered to present an unacceptable additional downstream threat” (In FEMA 94).Therefore, incremental hazard evaluation and the establishment of the IDF is, in essence, a risk-based approach.

The selection of the IDF is based on the evaluation of the magnitude of several flood events The incremental hazard evaluation begins with simulation of a dam failure during a hydrologic flooding condition, typically beginning with the PMF or percentage of the PMF as specified by the State hazard potential classification requirements. The same hydrologic event is then run for non-failure conditions. The water surface elevations for both the breach and non-breach events are compared to determine the increase in the water surface elevation resulting from the dam breach. If the incremental increase in downstream water surface elevation between the failure and non-failure scenarios results in an acceptable increase in consequences, (as defined by State requirements) a smaller percentage of the PMF flood inflow or other magnitude flood is then used to repeat the process. The process is repeated until the incremental increase in consequences due to failure falls within acceptable requirements specified by the State.

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

Loss of Life/Population at Risk

Probable loss of life is an important factor used in hazard potential classification systems and emergency action planning. DSO-99-06 (USBR, 1999) presents a risk-based method to estimate the number of fatalities that would result from dam failure. This method was developed using data from about 40 floods, many of which were caused by dam failure. These publications outline the following seven steps to complete an analysis for loss of life:

- Step 1: Determine dam failure scenarios to evaluate
- Step 2: Determine time categories for which loss of life estimates are needed

- Step 3: Determine when dam failure warnings would be initiated
- Step 4: Determine area flooded for each dam failure scenario
- Step 5: Estimate the number of people at risk for each failure scenario and time category
- Step 6: Apply empirically based equations or methods for estimating fatalities
- Step 7: Evaluate uncertainty

The number of fatalities resulting from dam failure is most influenced by three factors:

1) the number of people occupying the dam failure floodplain,

2) the amount of warning provided to the people exposed to dangerous flooding, and

3) the severity of the flooding. Without exception, dam failures that have caused high fatality rates were those in which residences were destroyed and timely dam failure warnings were not issued.

Estimating when dam failure warnings would be initiated is probably the most important part of estimating the loss of life that would result from dam failure.

For each failure scenario and time category, the population at risk must be calculated. Population at risk is defined as the number of people occupying the dam failure floodplain prior to the issuance of any warning. The method developed for estimating loss of life provides recommended fatality rates based on the flood severity, amount of warning time, and a measure of whether people understand the severity of the flooding. Recommended fatality rates for estimating loss of life may be determined based on a set of criteria that includes 15 different combinations of flood severity, warning times, and flood severity understandings.

2.8DAM BREACH MODELING ANALYSISTOOLS

Models for prediction of a dam breach have existed since the mid-1960s. However, the need for further development of dam breach models was realized in the 1970s as a result of several fatal dam failures, The sections that follow document the history of dam breach modeling and outline the current state of dam breach modeling tools. Recommendations on the selection of modeling software are provided.

2.8.1 DAM BREACH HYDROGRAPH AND PEAK OUT FLOW GENERATION TOOLS

The most common methods for either breach hydrograph generation or dam breach peak outflow computation are discussed below. These models/methods do not include the capability of a hydraulic routing of the breach hydrograph downstream. The NWS BREACH model is no longer supported by the NWS. The applicability, strengths, limitations, and governing equations applicable to available methods for breach hydrograph generation are summarized in the following table.

Table2.7:Methods for Breach Hydrograph generation only

Method	Applicability	Strengths	Limitations	Governing Equations
Empirical Equations	Earthen dam failure	Fast and simple to use	Large potential level of error; suitable for Tier 1 studies	Empirical relationships derived based on analysis of historical dam failures
		Minimal input data		
WinDAM B	Earthen dam failure overtopping breach	Models headcutting and downstream tailwater effects, and evaluates erosion in spillways using SITES technology	Limited to homogeneous embankments with simple embankment geometry. Level pool routing may not be applicable to some reservoirs.	Stress-based, energy-based headcutting equations available as analysis options. Integrity analysis for vegetation equations for riprap surface protection are based on threshold concepts for rock on steep slopes. The outflow hydrograph is obtained through a time-stepped solution.
		Physically based model using erosion and sediment transport principals		
NWS-BREACH	Since 2005, the model source code has not been supported by the NWS.			
USACE HEC-HMS	Concrete and earthen dam failure	Ease of program use	Level pool routing is not applicable to some reservoirs.	Continuity equation and an analytical or empirical relationship between reservoir/reach storage and discharge
		Inherently stable		Level pool routing

2.8.2 DOWNSTREAM FLOOD ROUTING

HEC-HMS Program

HEC-HMS is a hydrologic modeling program typically used to conduct hydrologic simulations of the precipitation-runoff process of dendritic drainage basins. The program can also be used to perform dam failure analysis. HEC-HMS was developed by the USACE in 1992 to replace the HEC-1 program.

In HEC-HMS, the user identifies ultimate breach parameters (i.e., breadth width, side slopes, time-of-failure) for dam breach simulations. Because the user defines the ultimate breach parameters, both earthen and concrete dam breaches may be simulated.

A dam breach simulation in HEC-HMS may be computed through two breach methods: overtopping or piping. For overtopping, the failure is simulated at a point on the top of the dam and expands in a trapezoidal shape until it reaches the maximum size input into the program. The piping dam breach function of HEC-HMS is used to simulate failures caused by piping inside an earthen dam. The failure begins with the water naturally seeping through the dam core until it increases in velocity and quantity enough to begin eroding fine sediments out of the soil matrix. The piping failure uses many of the same user-input parameters as the dam overtopping breach; however, it also requires the initial piping elevation and piping coefficient. The time growth curve may be specified in HEC-HMS as either linear, non-linear (sine wave), or user specified.

Similar to the precursor program HEC-1, HEC-HMS uses a level pool routing procedure for the upstream reservoir to estimate the breach hydrograph. The reservoir is represented as either a controlled or uncontrolled water body with the assumption of level pool and a monotonically increasing storage-outflow function. Hydrologic routing employs the continuity equation and an analytical or empirical relationship between reservoir/reach storage and the discharge. Output results from HEC-HMS include a resulting breach hydrograph that must be used in conjunction with other software, such as HEC-RAS, for downstream routing of the generated flood wave.

The main advantage of using HEC-HMS to simulate a dam failure is the ease of program use. The program does not suffer from the instability issues of its counterpart HEC-RAS. A major difference between HEC-HMS and HEC-RAS, , is that HEC-HMS uses level pool routing whereas HEC-RAS uses dynamic pool routing (full St. Vein ant equations of conservation of mass and conservation of momentum) for reservoir drawdown. However, dynamic routing requires detailed bathymetric data for the reservoir, which are frequently difficult and expensive to obtain. Level pool routing, on the other hand, only requires a simple stage-storage curve for estimating reservoir drawdown. Goodell et al. (2009) argued that dynamic routing is generally a more accurate method for estimating reservoir drawdown. However, level-pool routing is often an adequate method for drawdown computation. This is especially true for small reservoirs that are roughly equal in length and width and do not have a considerably long fetch length.

USACE HEC-RAS Program

The USACE HEC-RAS program released in 1995 is a one-dimensional steady- and unsteady-flow modeling program. The current version of the program can perform four functions: (1) steady-flow routing, (2) unsteady-flow routing, (3) movable-boundary flow for sediment transport analysis, and (4) water quality analysis have a considerably long fetch length.

The steady-flow component of the modeling system uses a standard step method intended for the solution of water surface profiles for steady, gradually varied flow. The basic computations are based on the one-dimensional energy equation in which energy losses are evaluated by friction and contraction/expansion of the channel. The momentum equation may be used when the water surface profile is rapidly varied in conditions such as a mixed flow regime. The system can handle a full network of channels, a dendritic system, or a single river reach. The steady-flow component is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. To perform a steady-state analysis for routing a resulting breach flow downstream in HEC-RAS, an upstream boundary condition must be provided in the model. This boundary condition is the peak outflow generated from the breach hydrograph that has been determined externally, in such forms as HEC-HMS, NWS BREACH, or an empirical equation.

The unsteady component of the HEC-RAS modeling simulates one-dimensional unsteady flow and can perform subcritical, supercritical or mixed flow regime computations. The governing equations for unsteady flow are the conservation of mass (continuity) and momentum equations derived from the full equations of motion (St. Venant equations). Upstream boundary conditions typically consist of an inflow hydrograph from the upstream watershed into a defined reservoir. For a dam breach analysis, the reservoir outflow is dynamically routed

Failure modes integrated into the HEC-RAS model include overtopping and piping. Additional failure modes may be approximated with variations to one of those two methods. Overtopping failures start at the top of the dam while a piping failure can start at a specified elevation/location and grow to the maximum specified extents. Breach parameters, such as breach width, depth, side slopes, and development time are estimated external to the model. Values for the breach size and development time are needed to produce a reliable estimate of the outflow hydrographs and resulting downstream inundation areas.

In HEC-RAS, both steady-state and unsteady-flow analysis use the same set of geometric data. This geometric data includes the reservoir storage volume, dam and downstream channel characteristics, cross-sectional data, etc. Differences in results between these two routing methods are a result of the computation procedures and inclusion of flow attenuation in unsteady-flow routing. The ASPFM has noted a generally small computational difference of 0.1 to 1 foot between steady and unsteady-flow analysis based on hypothetical event analysis (Altinakar, 2008). Further suggesting that while the difference between the two methods can be outside of this specified range, these differences do not necessarily mean that unsteady flow is more accurate than steady flow. The ASPFM has identified three key features between the steady-state and unsteady flow that provide computation differences:

1. **Losses**: Steady-flow losses computations use absolute differences in velocity head at adjacent cross-sections multiplied by an expansion or contraction coefficient, whereas unsteady-flow loss computations are computed by the momentum equation.

2. **Friction Slope**: Average friction slope between cross-sections is determined by averaging the conveyance method for steady flow. For unsteady flow, the average friction slope between cross-sections is computed directly from a simple average of the computed friction slopes.

3. **Discharge**: Steady-flow computations compute losses through downstream obstructions, such as culverts and bridges, directly from the obstruction geometry and the type of flow conditions through the structure. In unsteady flow, a family of curves is developed for defining the headwater-tailwater-discharge relationships through each obstruction for a full range of flow.

HEC-RAS can perform inundation mapping of water surface profile results directly using the RAS Mapper or the external HEC-GeoRAS tool. Using the HEC-RAS geometry and computed water surface profiles, RAS Mapper creates an inundation depth and floodplain boundary dataset. Additional geospatial data can be generated for analysis of velocity, shear stress, stream power, ice thickness, and floodway encroachment data. HEC-GeoRAS is a set of GIS tools that prepare the geometric date for import into HEC-RAS and generate the flood inundation data from the HEC-RAS output.

2.9 DAM BREACH FLOW ROUTING

Routing of the dam breach discharge hydrograph is a required step in a hazard evaluation or development of flood inundation mapping for Emergency Action Plans for all but the screening approach. Routing of the breach hydrograph is performed to evaluate the attenuated or reduced peak discharge at critical locations downstream of the dam. In addition to calculating the attenuation, determining the flood wave arrival time and the depth/velocity of flow at those critical locations are also very important parts of the analysis. If required, inundation mapping can also be generated from a hydraulic model of the downstream failure path.

Hydrologic routing employs the continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the end. In the absence of significant backwater effects, the hydrologic routing models offer the advantages of simplicity, ease of use and computational efficiency. Hydrologic routing models provide attenuated flow hydrographs at locations of interest, but cannot provide information on water surface elevations or flow velocities. Hydraulic routing employs the continuity equation and both energy and momentum balances to calculate open channel flow profiles. These equations are often referred to as the St. Venant equations or the dynamic wave equation. The full unsteady flow equations have the capability to simulate the widest range of flow situations and channel characteristics. Basic data requirements for hydraulic routing techniques include: Flow data (hydrographs), channel cross-sections and reach lengths, roughness coefficients, and internal boundary conditions. Hydraulic modeling is further subdivided into steady flow analysis and unsteady flow analysis. The difference between steady and unsteady models is the treatment of time. In unsteady flow, time dependent changes in velocity are analyzed explicitly as a variable, while steady flow analysis models neglect time all together. HEC-RAS can be utilized for both steady and unsteady flow analyses.

The HEC-RAS dynamic unsteady flow model is recommended for all situations if the accuracy and detail is needed for determining a hazard rating (e.g. borderline cases). However, if accuracy is not critical and the channel slope is greater than two feet per mile, a hydrologic routing technique may be more appropriate.The one-dimensional models discussed in this study also have downstream routing capabilities. Two-dimensional models which were not discussed in this paper use full dynamic or simplified forms of one- and two-dimensional shallow water equations to solve both one-dimensional channel flow and two-dimensional overland flow. The fully dynamic one dimensional flow routing through the reservoir and downstream river reach which is applied for this study is shown below.

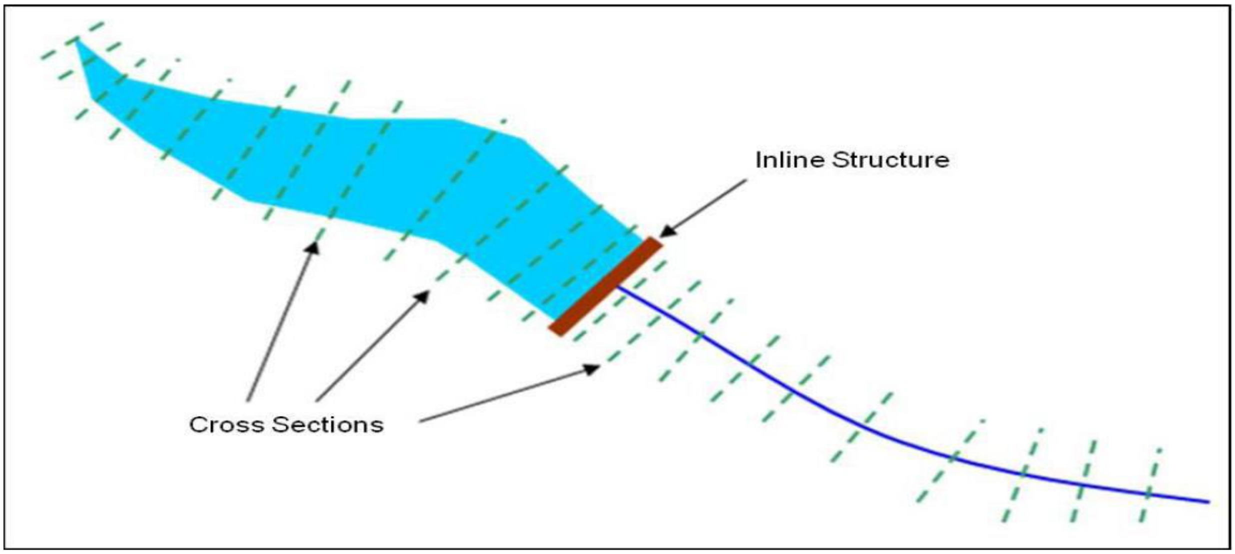


FIGURE 2.9 : CROSS-SECTION LAYOUT FOR ONE DIMENSIONAL FULL DYNAMIC ROUTING THROUGH RESERVOIR (HEC-2014):

2.9.1 BASICS OF ONE-DIMENSIONAL FLOW ROUTING

In inundation analysis, flow modeling is used to simulate the flow of a flood wave through a river reach and its floodplains. For modeling and design purposes, continuity, momentum and energy equations have been developed in the past to represent open channel flow in a mathematical way. These equations are based on three basic laws of physics, which are the continuity of mass, the continuity of momentum and the continuity of energy. Flow models simulate the flow through an open channel in a way that satisfies these basic equations for open channel flow or simplified versions of them.

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

Governing Equations

The law which govern the flow of water in a stream are the principle of conservation of mass (continuity equation) and the principle of conservation of momentum. These laws are expressed mathematically in the form of partial differential equations i.e. continuity equation and momentum equation.

Continuity Equation

$$\frac{\partial A_T}{\partial t} + \frac{\partial Q}{\partial x} - q = 0 \qquad \textit{Continuity Equation}$$

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

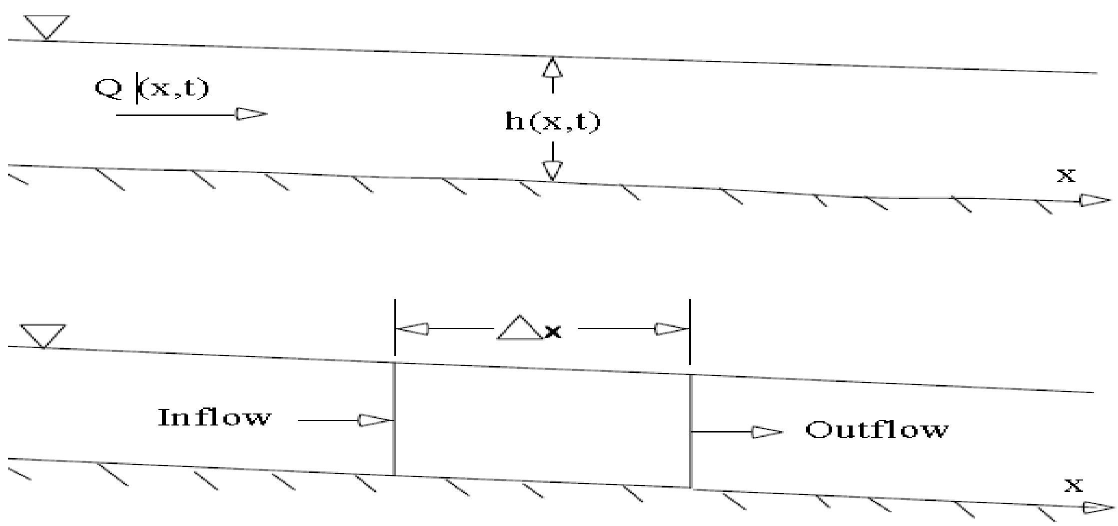


FIGURE2.9.1: ELEMENTARY CONTROL VOLUME FOR DERIVATION OF CONTINUITY AND MOMENTUM EQUATIONS (HEC-2010)

Momentum Equation

The momentum equation state that rate of change in momentum is equal to the external forces acting on the system.

$$\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA(\frac{\partial z}{\partial x} + S_f) = 0 \qquad \textit{Momentum Equation}$$

This equation is a vector equation applied in the x- direction. The momentum flux (MV) is the fluid mass times the velocity vector in the direction of flow (HEC-2010).

INUNDATION MAPPING

Dam-breach flood-inundation maps indicate areas that may be flooded as a result of a dam failure. The maps are used by wide range of end-users for planning and as a response tool to determine the effects of dam failure in downstream areas. In addition, the incremental areas flooded as a consequence of dam failure were considered for a dam classification exercise. For this study, flood inundation maps were generated using HEC-GeoRAS and ArcGIS.

Inundation maps can incorporate elements beneficial to dam safety officials, emergency responders, and mitigation planners. The maps can be used to facilitate communication during an event while at the same time convey relevant information regarding at-risk areas useful for effective long-term mitigation planning. For example, an inundation map may highlight the most vulnerable population areas. Such information is useful for mitigation planners, who may be able to minimize future flood damage via infrastructure projects and rezoning/relocation efforts. Dam breach inundation studies should continue to the point where adequate floodwater disposal is reached and the breach flood no longer poses a risk to life and property damage. The downstream extent of study should be established using the following criteria:

• There are no habitable structures in the dam breach inundation zone, and anticipated future development in the floodplain is limited;

• Dam breach flood flows are contained within a large downstream reservoir;

• Dam breach flood flows are confined within the downstream channel; or

• Dam breach flood flows enter a bay or ocean.

When determining the downstream limit of study based on the absence of at-risk habitable structures, the vertical accuracy of the dam breach modeling should consider whether structures located immediately adjacent to but outside the breach zone are appropriately classified as not at risk.

DEVELOPING AN EMERGENCY ACTION PLANS (EAP)

An Emergency Action Plan (EAP) is a formal but simple plan that identifies potential emergency conditions that could occur at a dam, and prescribes procedures to follow to minimize loss of life and the potential for property damage (Washington State, 2013). Ideally, the design, construction, operation, maintenance, and inspection of dams are all intended to minimize the risk of dam failures.

According to the State of Washington Department of Ecology (Washington State Department of Ecology, 2013), the primary function of an EAP is to provide a means of notifying downstream residents of failure or impending failure of a dam, so that the area can be evacuated in a timely manner. To accomplish this, the EAP must provide procedures to evaluate those conditions at the dam that could lead to failure, and clearly identify the circumstances under which the EAP is to be implemented. Secondly, it is used to identify strategies that can be taken following discovery of an emergency situation to prevent failure, or alternatively to delay failure until after downstream residents have been warned.

3 METHODS AND MATERIALS

3.1.DESCRIPTION OF THE STUDY AREA

3.1.1LOCATION AND HYDROLOGY

Ribb irrigation and drainage project dam is located in south Gonder of the Amhara National Regional state and at a distance of 625 km north of Addis Ababa and 130 km from Bahir Dar town. Immediate influence areas of the dam are found in two woredas of south Gonder zone named Ebinat and Farta.

The dam axis is located between the geographic grid of UTM E392174.64, N1330225.76 and E390813.45, N1330018.02, at an altitude of 1880 m to 1970 m. The left abutment is situated at an altitude of 1943 m, the center of the dam axis is situated at an altitude of 1873 m and the right abutment is situated at an altitude of 1966 m. The elevation in the watershed ranges from 1900 m asl around dam to almost in the upper ridge 4135 m asl. The Ribb river which is some 130 km long, has a drainage area of about 1790 sq km and an average annual average discharge of 11.6 m^3/s. The catchment area at the dam site is 685 sq km. the river which flows generally in a westerly direction and empties in to lake Tana, is one of the main streams flowing in to Lake Tana from the east. The Ribbriver with its tributaries, drains the western slope of the high mountainous area east of the town of Debre Tabor, with a peak elevation of approximately 3050 m.

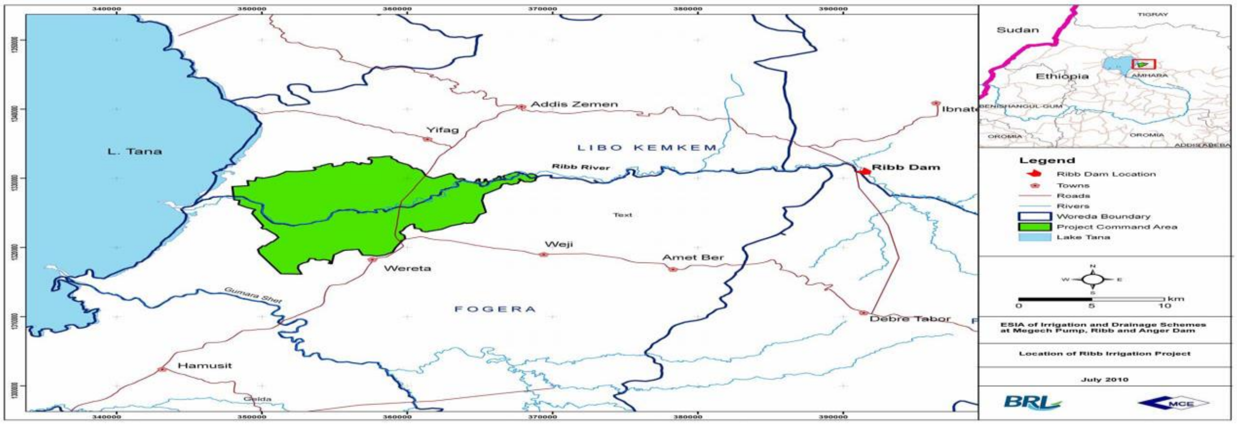


FIGURE 3.1:RIBB DAM SITE AND IRRIGATION SITE LOCATION

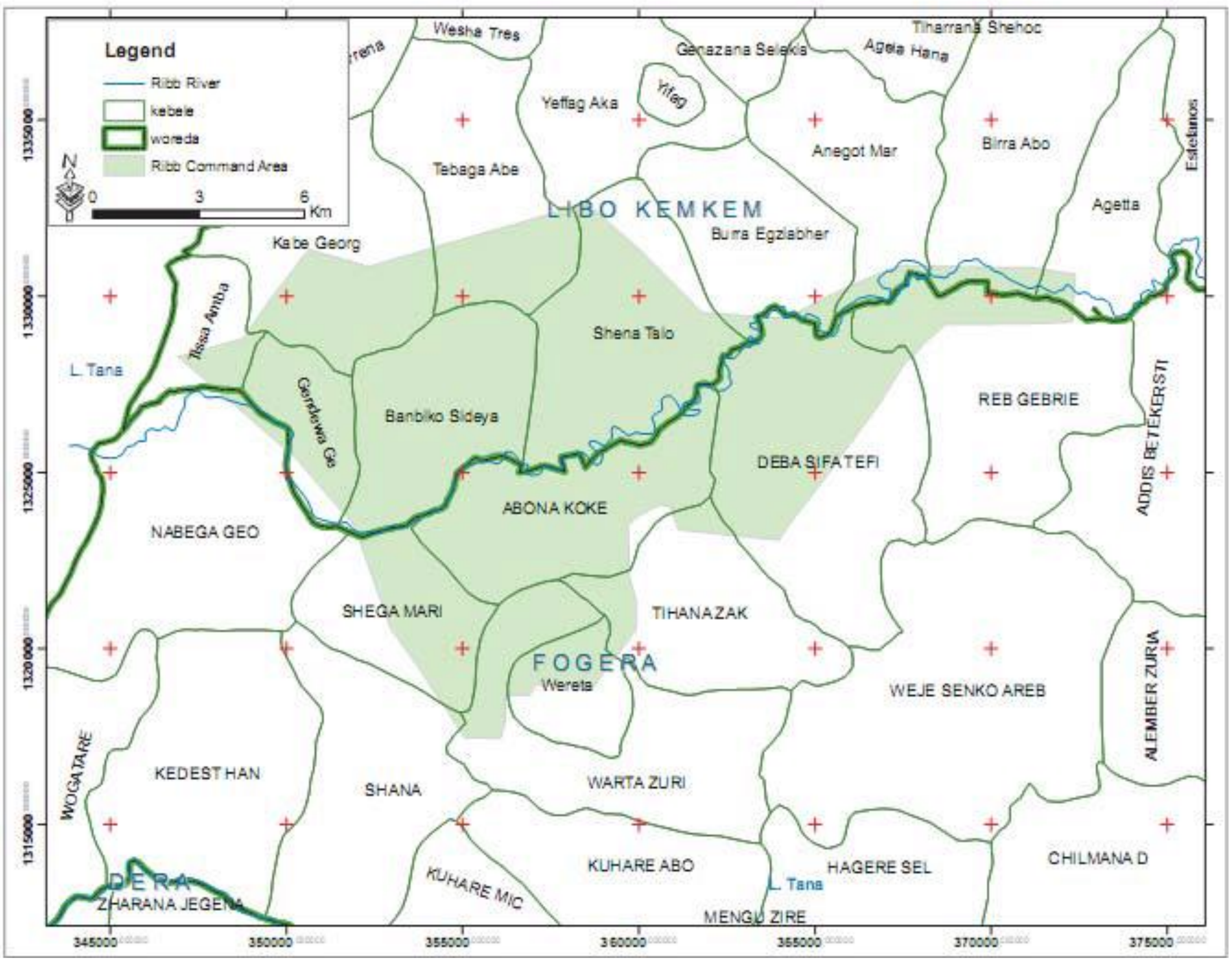


FIGURE 3.2 :RIBB IRRIGATION AND DRAINAGE PROJECT - COMMAND AREA AND KEBELE BOUNDARIES

3.1.2TOPOGRAPHY

The dam site is characterized by broad and flat flood plains, old bench forming terrace and low to high relief basaltic hills with steep to moderately steep slopes. The right and left abutments of the dam are characterized by steeper slopes with slope angle of 35° to 46° .There are developments of relatively few shallow seated gullies at the reservoir catchments attributed to rill and gully erosion. The peak topography in the area is marked by Shikra hill, which is at an altitude of 1973 m. The upper Ribb water shed is characterized as mountainous, wedge shaped and steep sloped (3.6%) water shed. The highest elevation of the water shed is about 4,100 m in its south eastern part. The lowest topography land is at the dam site, which is at an altitudeof 1873 m.

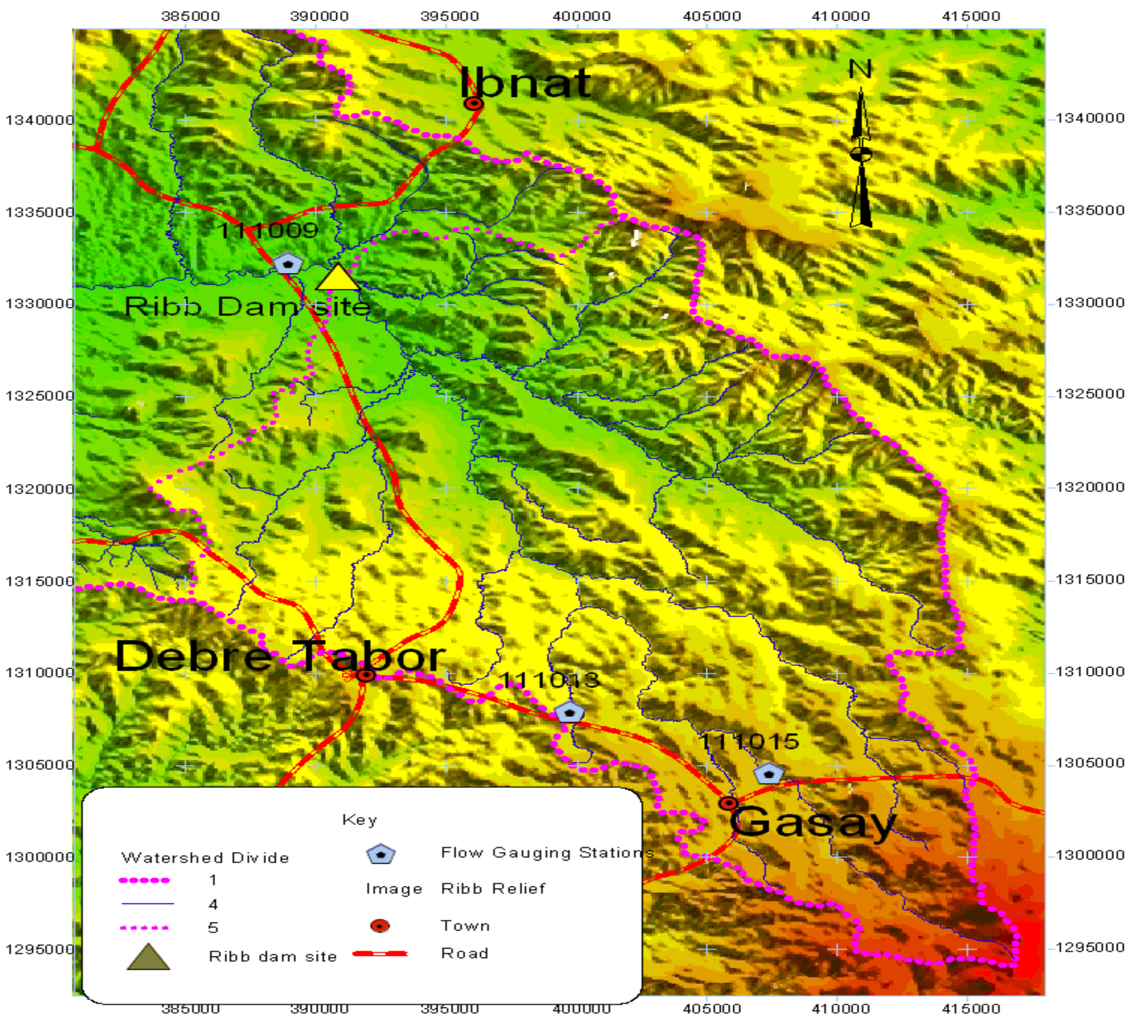


FIGURE 3.3: RELIEF MAP OF UPPER RIBB WATER SHADE

3.1.3 CLIMATE

The climate of the Ribb basin is marked by a rainy season from May to September, with monthly rain fall varying from 65 mm in May to 411 mm in July. Mean annual precipitation is about 1,400 mm in the upper part and about, 1,200 mm in the lower part. The dry season from October to April, has a total rain fall of about 8% of the mean annual rain fall. Dependable rain fall varies from less than 13 mm during the dry season to 80-275 mm/month during the period of June to August, equivalent to 40-80% of the average values. Temperature variations throughout the year are minor (19°C in December to 23°C 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 Dec and 140 mm/month in April.

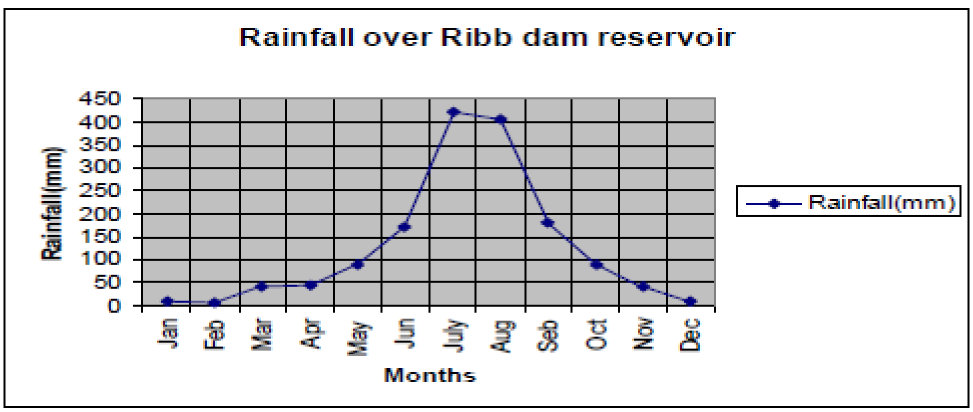


Figure 3.4 Rain fall over Ribb Dam Reservoier

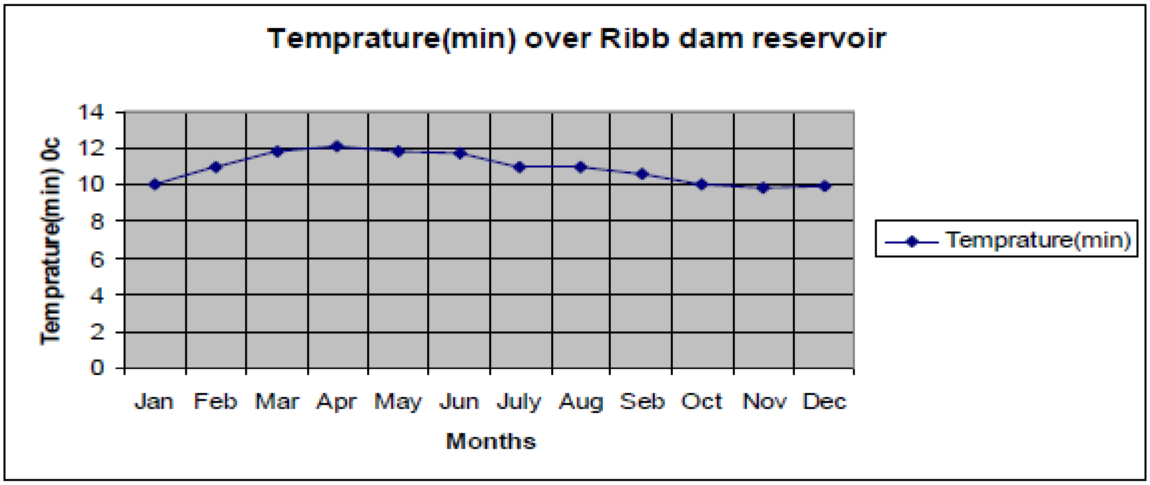


FIGURE 3.5:MINIMUM TEMPERATURE OVER RIBB RESERVOIR

3.1.4. GEOLOGY AND LAND USE

The geology of the Ribb Basin is dominated by a huge volcano named GunaTerara. It corresponds to the eruptive events that have occurred during the early Miocene to Pliocene periods, and is classified in the shield group basalt. The common lithotype for this material refers to lenticular basalt with large amount of interbeded scoriaceous lava and basalt agglomerates. Some paleosoils may be interbeded. The other smaller volcanoes located at the north are also considered being active during the same geological period. The lower part of the valley before Lake Tana is completely overlain by recent fluvial depositions, which are mainly formed by silt to clay deposits. Recent volcanic flows have also been noted but they appear to be localized in the lower section of the Ribb plain. No evidence of such flows has been mapped in the upper parts of the Ribb basin. The orientation of the major system is N–S and NNW–SSE for the second system. Minor fractures may come out locally. There is no evidence of slope instability in the Ribb Reservoir. This is mainly due to the smooth landscape of the reservoir. The reservoir might be fairly watertight due to the clayey blanket covering the slopes and to the basaltic agglomerates and the tuffs formations series forming the reservoir.

The dam site and large parts of the flooded area are extensively used for farming, settlements being denser at the upper reservoir slopes and top of the hills.

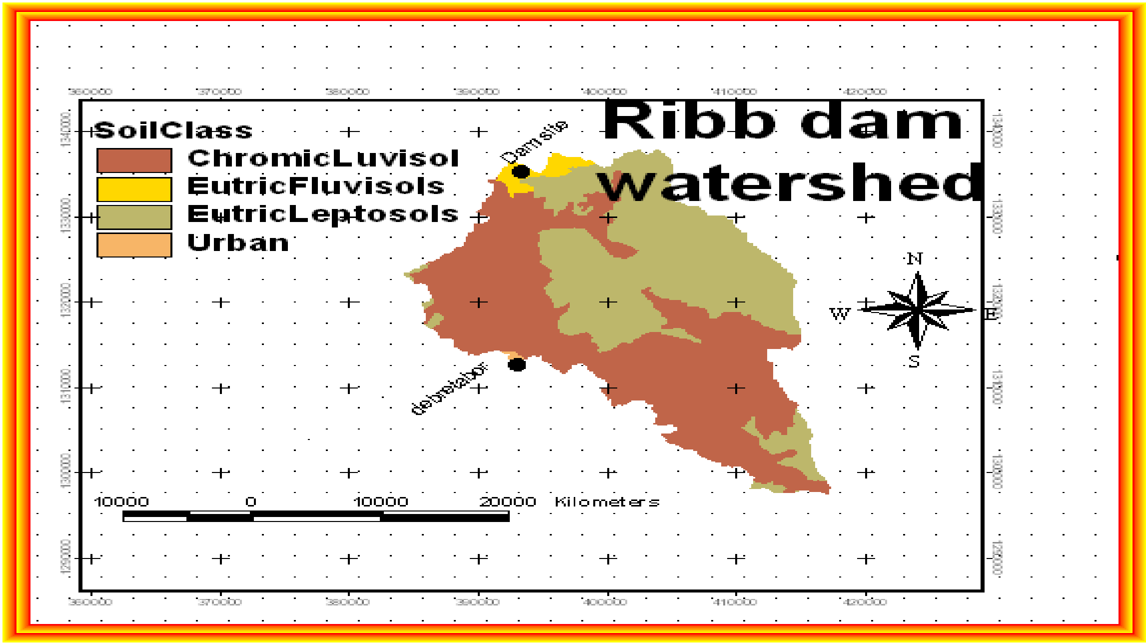


FIGURE 3.6:GEOLOGY OF RIBB DAM WATERSHED

3.1.6. DAM TYPE

An 800 m long straight dam axis has been selected, between the two prominent hills forming the left (E 391355.36 N 1330299.37) and right (E392174.64 N1330225.76) abutments. Consideration was given to the most economical alignment fitting the topography; and avoiding thin abutment ridges. Photo 6.2-1 below shows a picture taken at the dam site, indicating the dam axis. The main dam axis was selected to correspond to the BCEOM Pre-Feasibility Study boreholes line, between boreholes BH-R1-old and BH-CG3-old. Based on the preliminary data of these two boreholes, and the site visits, a complementary geotechnical field survey program was established. The location of the existing and planned boreholes is shown below.

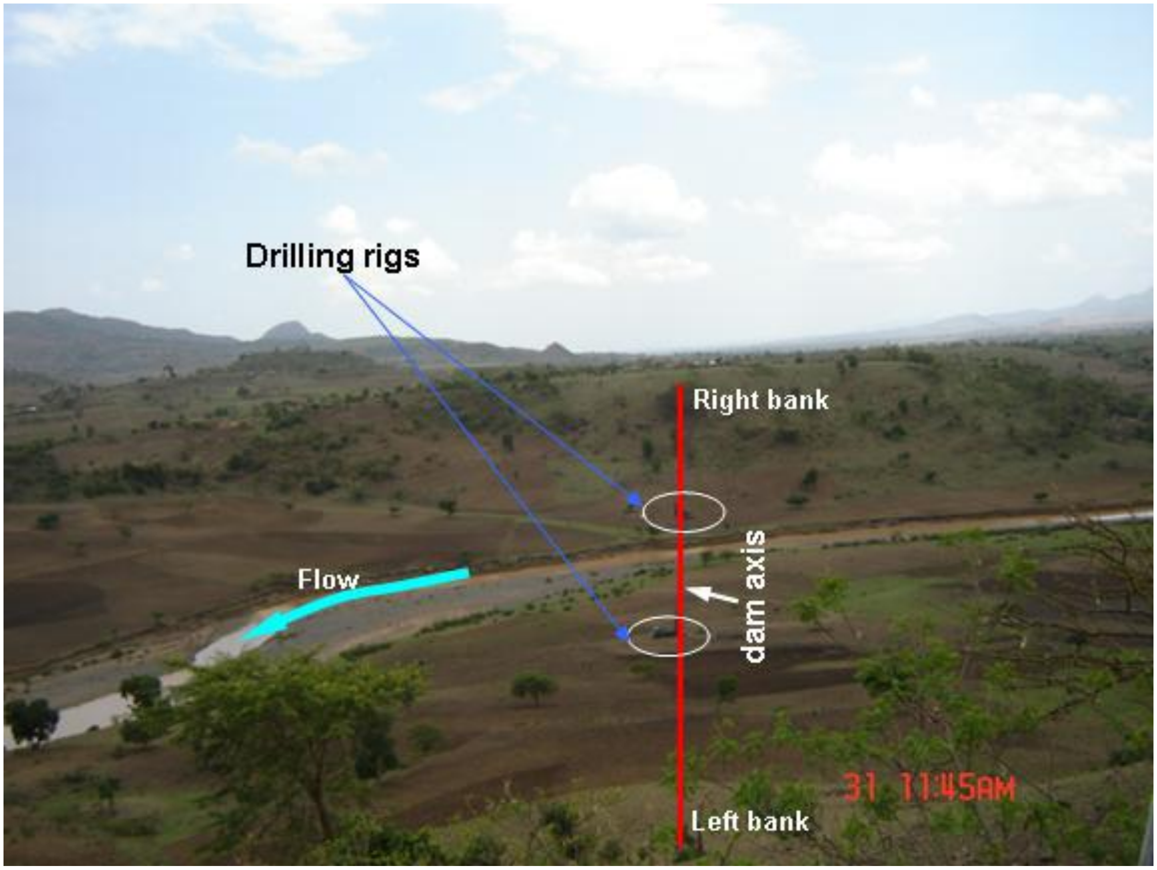


FIG 3.7:RIBB DAM SITE

The basic consideration in the design of Ribb Dam has been to achieve safety consistent with economy. In the BCEOM Pre-Feasibility Study, an earthfill dam was considered. Following this proposal and the present geotechnical investigation, primarily with respect to the availability of construction materials, different alternative dam types including an earthfill with central core, a rockfill with central core, and an earth-rockfill with central core were considered in the feasibility design stage of this study.

Based on the results of safety and economic analyses of several embankment sections that have been studied during the feasibility design stage, the selected dam type was an earth-rock fill dam with central core. This was selected because it has lower cost and allows the use of the expected large foundation alluvium excavations in the embankment fill.

The dam is a zoned embankment dam with an impervious central "Core" composed of clayey/silty material from the reservoir and downstream nearby areas (See Fig below). A central core has the advantage of providing higher pressure at the contact between the core and the foundation, thus, reducing the possibility of leakage and piping. The top level of the core has been fixed to be the same as the maximum flood level for 1day PMF, which is 1945.0 m.

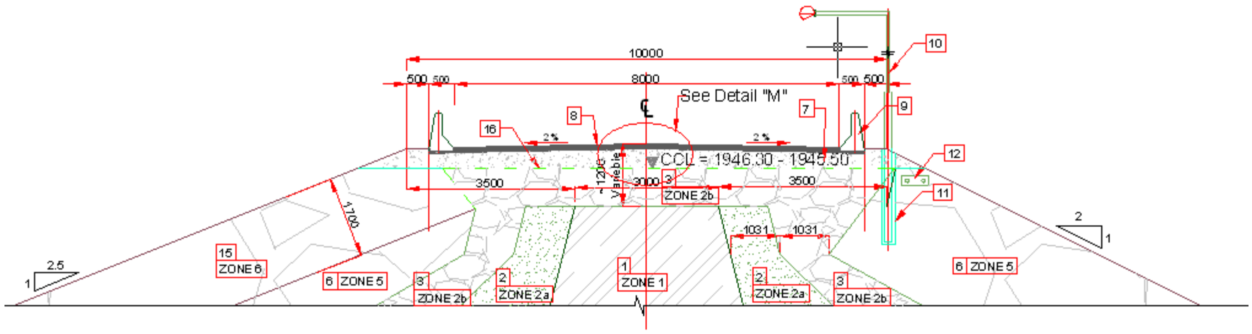


FIGURE 3.8:DAM CREST DETAIL

Legend

1) -zone1- Compacted core material fill

2)- Zone 2 – Filter materials

3) - Zone 3 – Compacted shell material

4) - Zone 5 – Transition section

5) - Zone 6 – Rock fill

6) - Two compacted layers of grade “A” base coarse material, each 250 mm thick

7) - Asphaltic concrete

8) - Guard rail

9) - Lamp post

10) - Concrete

11) - Electric cables

According to common practice, a core width at the base, or cutoff, should be at least 25 to 50% of the difference between the maximum water level in the reservoir and the minimum tail water elevation. For Ribb Dam, it has been decided to design the core width base to be 50% of the MWL water head. Given the core crest width and the core height, results in a 0.25H to 1V impervious core slopes. A core top width of 3 m has been provided as the minimum for construction purposes.

The width of the base of the core, at river bottom level (1873 m), will be approximately 39 m. At bedrock elevation at the lowest point (~1848 m.), the core width will be some 51.5 m. The permeability of the compacted core material should not exceed 10-5 cm/s. The impervious core of Ribb Dam is proposed to be flanked by a 1.75H: 1V slope shell composed of compacted sandy/gravel alluvium to be obtained from dam foundation excavations and additional sandy gravel shell materials from borrow areas.

This arrangement was cost effective in view of using the expected large alluvium excavations from the dam foundation at the valley floor in parts of the embankment. The shell is in turn flanked by a 1V:2.5H upstream slope and 1V:2H downstream slope (with 5 downstream berms) free draining rockfill composed of basalt to be obtained from quarry sites and some from spillway excavations. One of the basic requirements for design of earth-rockfill dams is to ensure safety against internal erosion, piping and development of excessive pore pressures in the dam. For Ribb Dam, a chimney drain adopting the core side slopes and composed of fine and coarse filters between the core and the shell have been proposed. To safely discharge the seepage water from the chimney filter/drain and to protect erosion of fines from the foundation alluvium, a horizontal drain composed of fine and coarse filters is also provided.

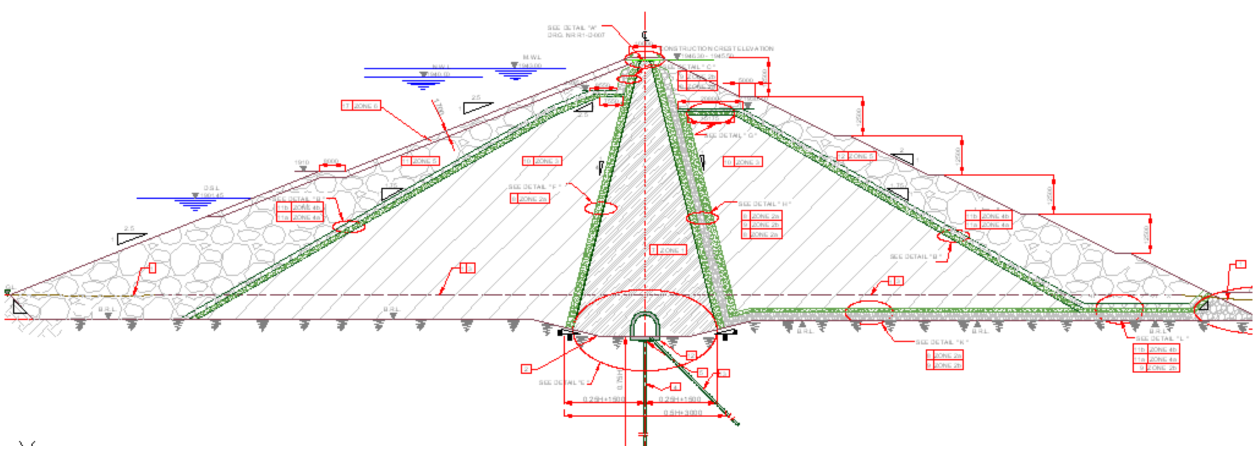


FIGURE 3.9:DAM CROSS-SECTION

Legend

1) **Zone 1** – Compacted core material fill

2) **Zone 2** – Filter material

3) **Zone 3** – Compacted shell material
4) **Zone 4** – Compacted alluvium material
5) **Zone 5** – Transition section
6) **Zone 6** - compacted rock fill

3.1.7. RIBB DAM RESERVOIR

The Ribb Reservoir flooded area can be characterized as mountainous river valley quite narrow at the dam site. The valley contains cultivated plots being located wherever the topography slopes are not too steep or rocky. The rest of the watershed appears to be grazing area and rocky shrubs. As this area is going to be flooded, the available construction materials, such as rockfill, shell, core material and filter material in the flooded area, should be exploited, leaving enough material cover to prevent increased losses/seepage. The volume of reservoir is 294Mm^3 at an elevation of 1943 m, which is the maximum water level of the reservoir.

FIGURE3.10:RIBB RESERVOIR AREA FROM FARTA WOREDA SIDE OF RIVER

3.2 DATA COLLECTION

To predict the failure impact analysis downstream the correctness and accuracy of data is necessary in dam breach modeling. Relevant information needs to be collected to model dam breach and downstream effects due to failure.

3.2. 1 GENERAL INFORMATION

This section gives an overview of the study area as well as the data used for hydraulic modeling on dam breach analysis. Floods due to dam failure are generally significantly larger than natural floods. In Ribb dam breach modeling and downstream analysis, hydrologic data ,reservoir capacity are all used here from Ministry of Water & Energy and W.W.D.S.E.

3.2.2. TOPOGRAPHIC DATA

A study area description of the land surface is an important part in the process of river hydraulics model. Elevation (topography) data is used to establish the area available to transport flow downstream. In order to map the flood inundation area due to the water surface profile outflow from the dam breach, topography data or elevation data must be known. The primary topographic (elevation) data for river channel, left and right banks and each cross-section used for the analysis of Ribb Dam breach is extracted using a Digital Elevation Model (DEM) of resolution 30m by 30m. This is done by the incorporation of ArcGIS 10.1 and its extension tool HEC-GeoRAS 10.1 along the river up to 40kilometers downstream of the dam.

A Digital Elevation Model (DEM) is a digital file consisting of terrain elevations for ground positions at regularly spaced horizontal intervals. Its uses range from scientific, commercial, industrial, and operational to military applications.

The study area ground surface is represented in Arc GIS 10.1 as a vector format. The vector format model stores data as a series of triangulated points forming a continuous network of triangles. This vector format is referred to as triangulated irregular network (TIN). A TIN format is the data storage as a basis for river hydraulics and the ground surface is more accurately described by a minimum of data.

3.5.3 HYDROLOGICAL DATA

The hydrological data such as inflow hydrograph, maximum probable flood (PMF) and reservoir capacity used for hydraulic modeling here in the study were collected from Water Works Design and Supervision Enterprise and from Ministry of Water and Energy office.

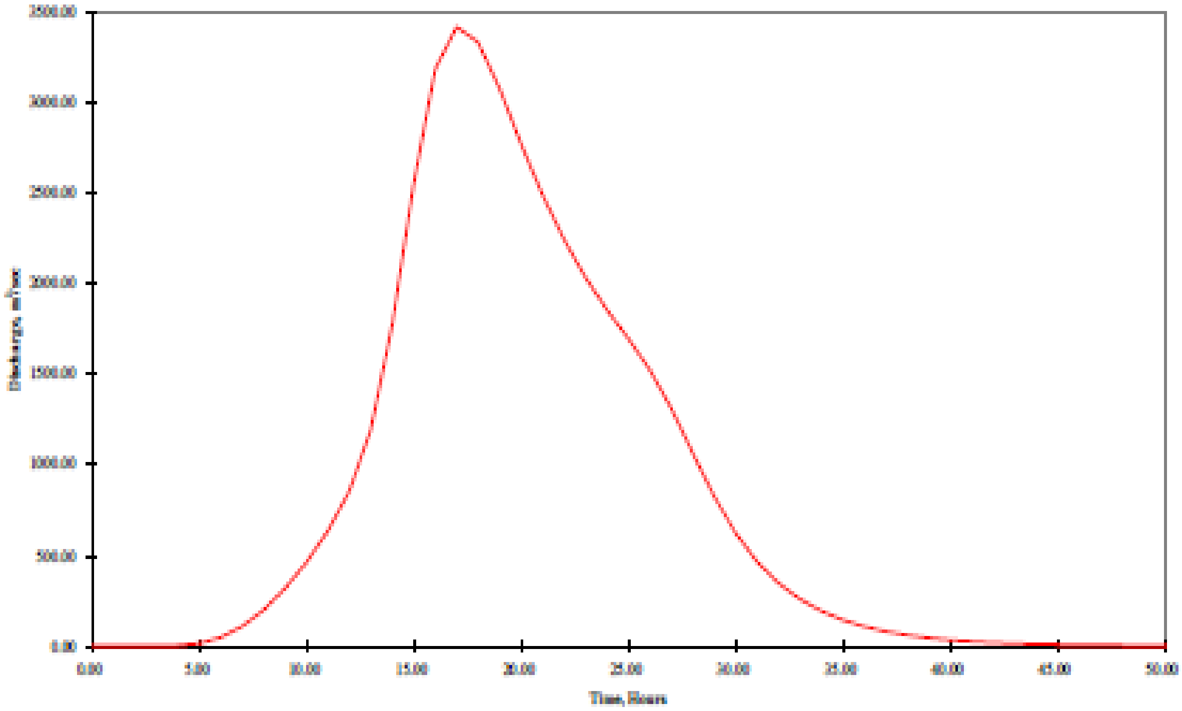


Figure 3.10.1   24- hour PMF hydrograph at Ribb dam Site

Table 3.1 Land use data for Ribb Catchment

Land use/Land cover type	**Size (ha)**	**Coverage (%)**
Cultivated land	40798.7	59.56
Grazing land	11241.3	16.41
Plantation	1307.53	1.91
Shrub Land	13200	19.27
Wood Land	68	0.01
Afro-Alpine	56.97	0.08
Bare land	1819.5	2.66
Urban	8	0.01
Total	68500	100

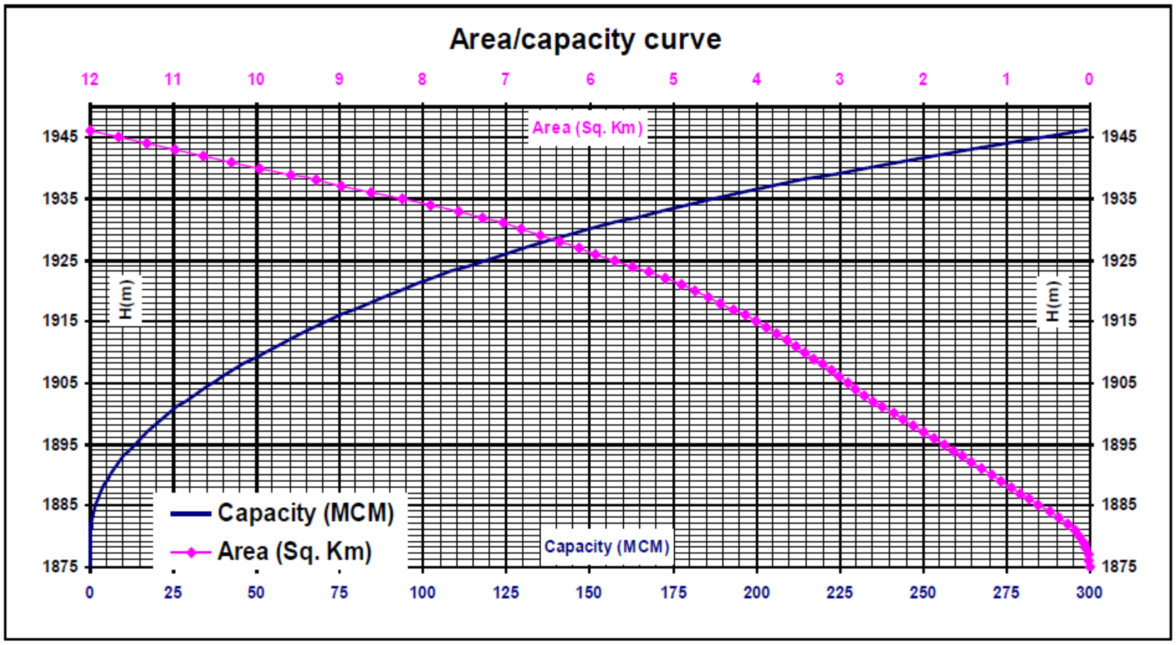


Figure 3.10.3.1 Ribb dam reservoir area - capacity curve

3.6 DOWNSTREAM COMMAND INFORMATION

The stored water behind Ribb Dam will be diverted into canal sys*t*em on both the right and left bank for providing water through a network of canal system for irrigation for command area half of 20,000 hectare irrigable land. This irrigable command area is located at a distance of about 28 kilometers downstream of Ribb Dam.

3.7 POPULATION AT RISK

Environmentally sensitive areas include the Ribb River, Lake Tana, the Fogera Plain, and wetlands along the lake shore and within the plain, especially in and around Welela and Shesher Ponds. Wetlands on the plain may be dependent on seasonal flooding of the Ribb Riverrange and type of use (e.g. spawning, rearing). Lake Tana, Fogera Plain and the associated wetlands are wintering habitat for vulnerable and threatened bird species20 such as Lesser Kestrel, Wattled Crane, Greater Spotted Eagle, Lesser Flamingo, Pallid Harrier and Great Snipe, so mitigation measures to protect these wetlands may be important. Water-borne diseases are the major health threats in the area with malaria being the most important.

The population size downstream and around command area, as projected from the 1994 census, was 39,958 in 1997. The majority of the population belongs to the Amhara ethnic group.

3.8 GENERAL PROCESS OF DAM BREACH MODELING

In dam breach modeling and downstream risk analysis process, some procedures must be followed. First, breach parameters were estimated using three empirical equations. Then, downstream channel geometry (elevation data) is extracted from Digital Elevation Model (DEM) using HEC-GeoRAS tool in Arc Map. The breach parameter and channel geometry are entered as input to HEC-RAS to model dam breach. The breach outflow is then routed through downstream channel to calculate hydraulic properties at critical locations. Finally, exporting the model result to Arc Map, the flood inundation will be mapped to analyze the affected critical locations. The general dam breach modeling and downstream analysis procedures are well described below.

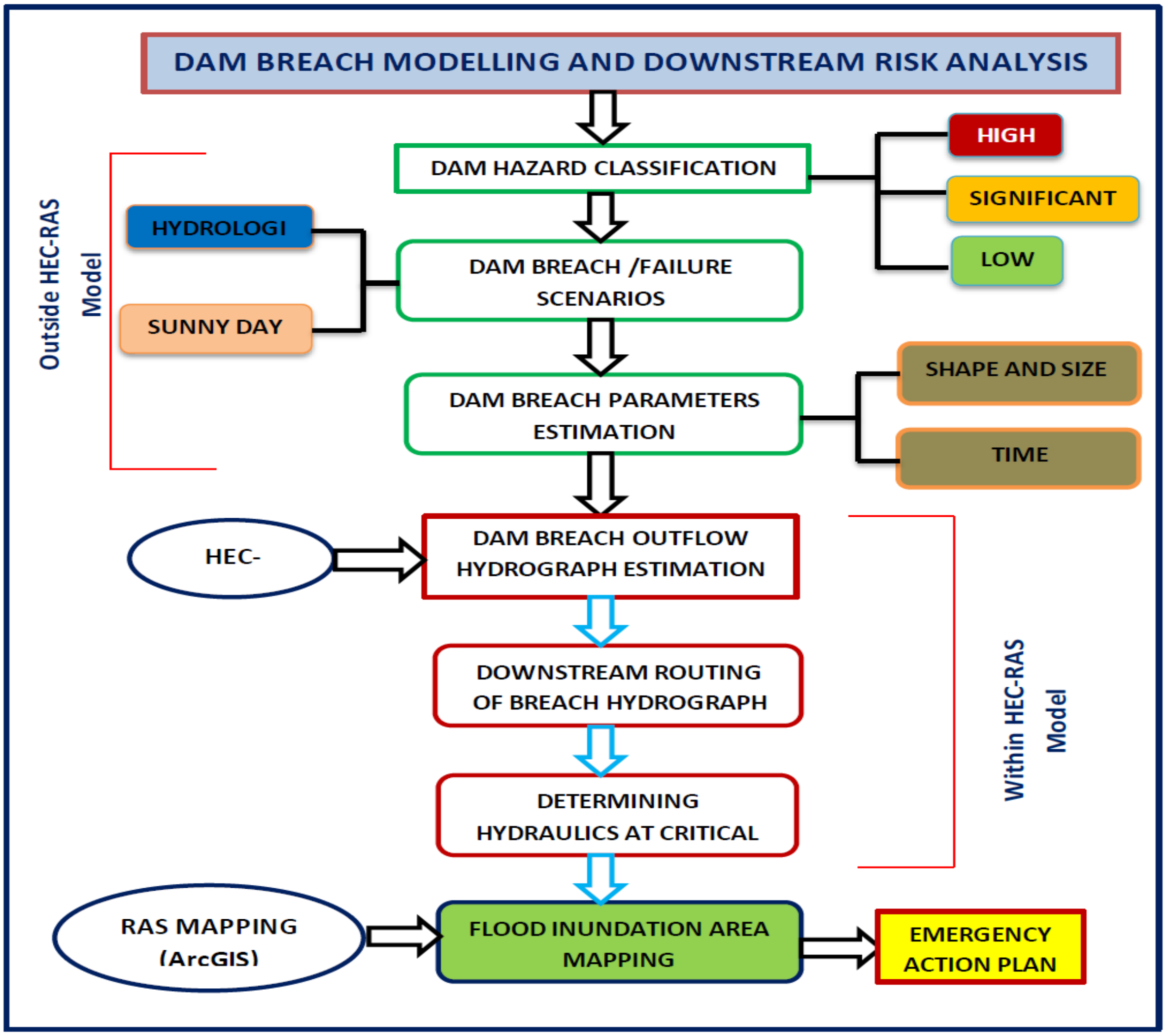


Figure 3.10.4 Flow diagram of General Dam Breach Modeling and Analysis Process

Ribb Irrigation and Drainage Project Dam has a reservoir capacity of 234 Mm3 at Full Reservoir Level (FRL) and 295 Mm^3 at maximum water level.. It has also a dam height of 73 m and 20,000 ha irrigable area downstream. Near the downstream end of the dam, there is a town named Woreta and Kemkemworeda in left and right side of the river respectively.Also the downstream area has different structures, farm lands both sides of the river and population size which can be affected due to flood-flow in case the dam failure should occur. Therefore, according to the hazard potential classification guidelines and depending on the downstream population size and property damage due to dam failure flood-flow, Ribb Dam was classified as a “High Hazard Dam” in this study.

One-dimensional full dynamic flow routing is used for RibbDam breach analysis. This earth-rock-fill embankment dam is modeled for overtopping and piping dam breach mode of failures. For an overtopping failure mode, the failure location is assumed at centerline of channel river bed in the downstream side of the embankment dam; whereas for piping mode of failure, the failure location assumed to be at an elevation of 1946 m at top of the dam and at 1892 where intake of canal outletwasfound.

The main parameters that must be estimated in this analysis are the breach formation time and the maximum size of the breach opening (breach width). These parameters are estimated by empirical (regression) equations and entered to HEC-RAS 4.1 as an input for the prediction of outflow hydrograph from the breach.

The regression equations applied for this purpose are Macdonald and Langridge Monopolis-1984, Von Thun and Gilette -1990 and Froehlich -2008, 1995.

3.9HYDRAULIC MODEL DEVELOPMENT

Here in this sub topic, the dam breach analysis model was discussed in detail. The dam breach analysis model involves the prediction of the dam breach hydrograph and the routing of this hydrograph downstream at critical locations. For this analysis process, hydraulic modeling (HEC-RAS 4.1) was used. In addition to this hydraulic modeling, ArcMap software is used for all GIS related tasks and HEC-GeoRAS (ArcGIS extension tool) serves as the interface between GIS and the hydraulic modeling (HEC-RAS).

For the use of ArcGIS in this study, the Environmental System Research Institute (ESRI) Arc-map software version 10.1 was used. ArcMap is the main component of ArcGIS which is geospatial processing software. In this paper, most of the ArcGIS tasks were performed using other ArcMap extension tool HEC-GeoRAS. This GIS extension tool was used to prepare a reliable model input for hydraulic model, HEC-RAS 4.1.0 within the ArcMap software environment. In addition to this, HEC-GeoRAS was used to visualize hydraulic modeling results in the form of inundation depth maps.

3.9.1 HEC-GEO RAS DEVELOPMENT

HEC-GeoRAS is a set of tools specifically designed to process geospatial data to support hydraulic model development and analysis of water surface profile results (HEC, 2009). HEC-GeoRAS.10.1 which is compatible with Arc Map 10.1 is used in this study in creating RAS Layers in ArcGIS to extract information essential for hydraulic modeling (HEC-RAS). It is used to extract elevation data from DEMs in the TIN format.

For developing each of the RAS layers, DEM were projected into a coordinate system. The stream centerline layer, bank lines layer, flow path layer and cross-sectional cut line layer were created. The development of all other RAS Layers is optional based on the data needs for the river hydraulics model.

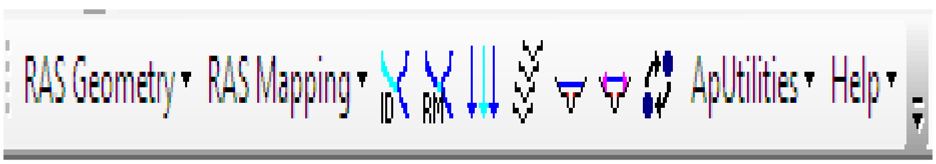


Figure 3.10.5 HEC-geoRAS tool bar used for ARC-GIS 10.1

The Stream Centerline layer is used to identify the connectivity of the river system. It is created in the downstream direction and is used to assign river stations to the cross sections, and other structures to order computational nodes in the HEC-RAS model.

The Cross-Sectional Cut Lines layer is the principal data constructed using HEC-GeoRAS. Cut lines are digitized across the floodplain area to capture the profile of the land surface. Cross sections should be digitized perpendicular to the path of flow in the channel and overbank areas to be consistent with one dimensional flow characteristics. The bank lines and flow path centerlines were created before laying out cut line locations.

After these RAS Layers were created, Geo-RAS tools and menus were assigned and populated attribute data. Lastly, after all the attribute were completed, data were written out to the HEC-RAS geospatial data exchange format and were imported into HEC-RAS 4.1.0.

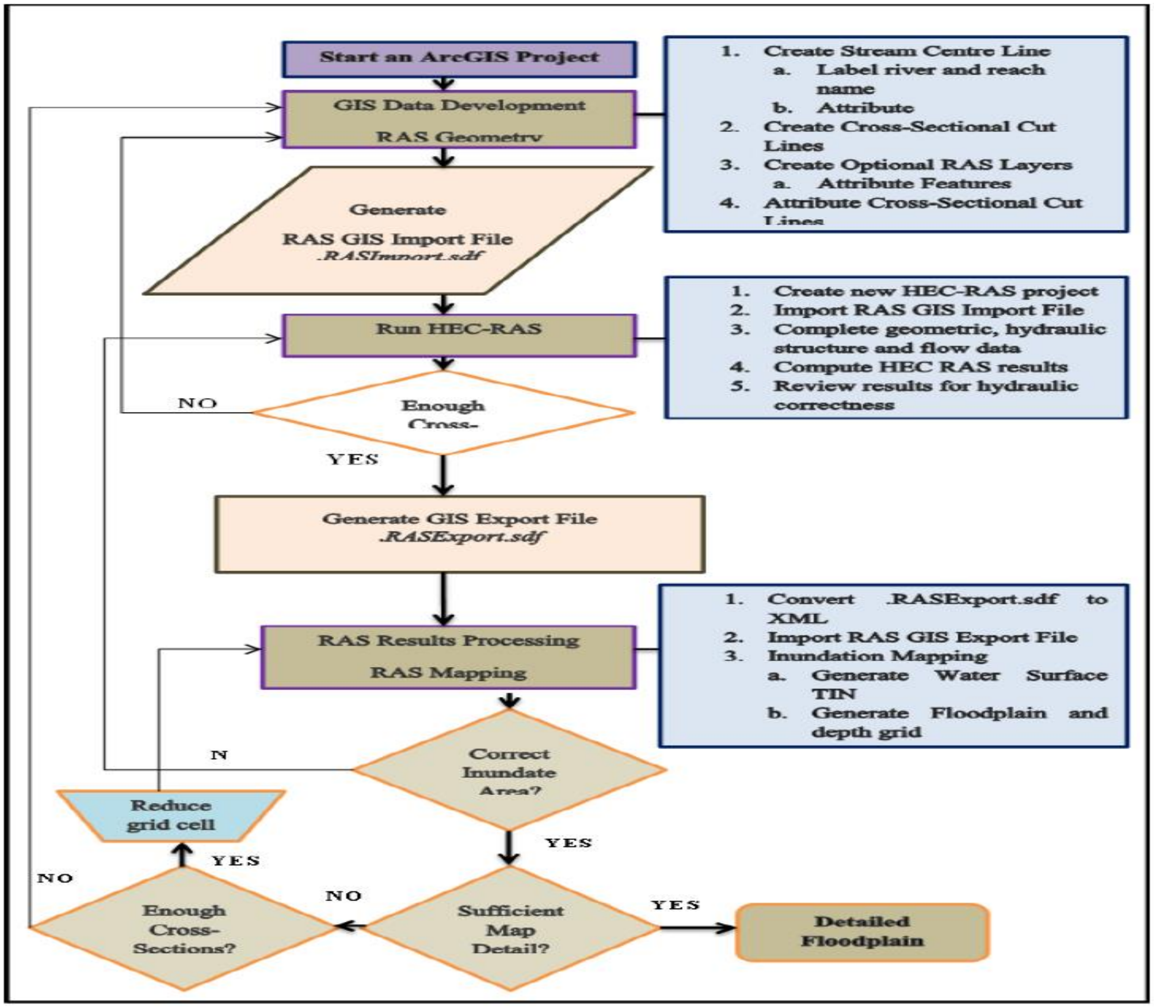


Figure3.10.6. Process flow diagram for using HEC-Geo RAS

Cross Sections

Cross sections are used to define the shape of the stream and its characteristics, such as roughness, expansion and contraction losses, and ineffective flow areas. HEC-RAS requires cross-sections for water surface computation. Approximately 70 cross sections were created for the model using HEC-GeoRAS at about 500m spacing. Elevation data then were extracted from Digital Elevation Model (DEM) and imported to HEC-RAS. Each cross-section ideally is perpendicular to flow direction, and can intersect the main channel only once and may not intersect another cross-section. The cross sections were located to adequately describe geometric features such as roughness changes, grade breaks, expansions and contractions. The cross sections were drown to remain perpendicular to the expected maximum flood wave flow line.

Roughness Values

Estimation of Manning’s roughness coefficient (or Manning’s n) is very important to simulate open channel flows. As an empirical parameter, the roughness coefficient actually includes the components of surface friction resistance, form resistance, wave resistance and resistance due to flow unsteadiness (Y.WANG, 2005). Direct determination of the roughness coefficient is almost impossible in studying natural river flows, including unsteady channel network flows.

The roughness coefficient (n) in natural channels is difficult to determine in field. Various factors affecting the values of roughness coefficients were presented by (Chow V. T., 1959). The friction slope may thus be seen as a very important parameter whose value must be chosen very carefully.

From the assessment of land use land cover, major land cover types identified include intensively cultivated, settlement, shrub land, grazing land , bare land, Afro-Alpine, wood land and open grassland(From WWDSE).

Based on these characteristics of flood plain, the Manning’s n values for the stream channel downstream of the Ribb dam ranges from 0.025 to 0.04 to reflect the dynamic and extreme nature of a dam breach flood wave as well as different material types within the channel. The left and right overbank n values also ranges from 0.025 to 0.04 reflecting riverine forests intensively cultivated, etc. along the flood path. Manning’s n-values were based on published values for similar conditions (CHOW, 1959).

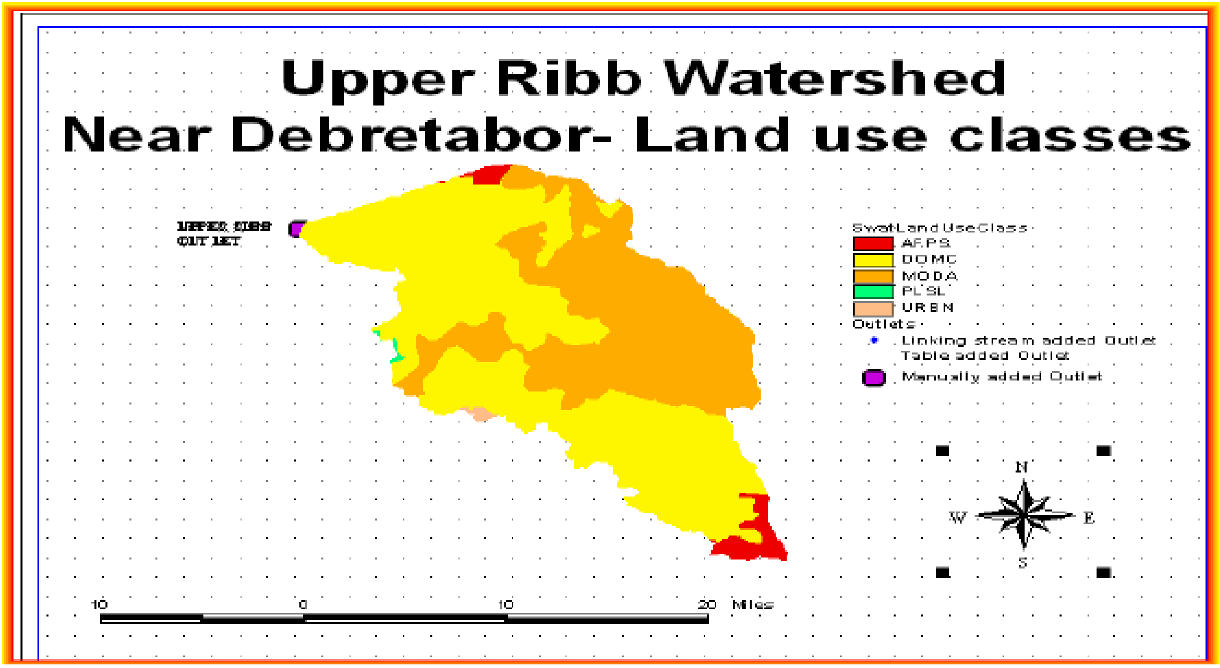


Figure 3.10.7.Upper Ribb Land use classes

3.9.2 HEC-RAS MODEL DEVELOPMENT

Purpose of the Model

The USACE HEC-RAS program released in 1995 is a one-dimensional steady- and unsteady-flow modeling program. The 4.1.0 version of the program is used in this study, which can perform functions like: 1D- steady-flow routing, 1D-unsteady-flow routing, movable-boundary flow for sediment transport analysis, water quality analysis and so on with a graphical user interface. . HEC-RAS also has the capability of modelling dam breach events under a wide range of scenarios.

Cross sections, stream centrelines, and other geometric features of the stream were extracted from GIS using HEC-GeoRAS and ArcGIS. Dam failure scenarios were analyzed for the Sunny Day or Non-Hydrologic and Hydrologic events or during flooding conditions. The objective of this modeling effort is to evaluate the impact of a dam breach on the downstream population and property damages.

In this study, the one dimensional unsteady flow analysis of HEC-RAS model was discussed for downstream dam breach outflow hydrograph routing along Ribbriver channel.

HEC-RAS 4.1.0 is used to model overtopping and piping failure breaches for earthen-rock dams. The resulted flood wave is routed downstream using unsteady flow equations. Inundation mapping of the resulting flood can be done with the HEC-GeoRAS when GIS data (terrain data) are available.

Dams are modeled within HEC-RAS by using the inline structure Editor. It allows to put in an embankment, define overflow spillways and weirs. The upstream reservoir of the dam can be modeled with cross-sections or by using a storage area. In using cross-sections, HEC-RAS will perform full unsteady flow routing through the reservoir and downstream of the dam whereas in using a storage area, HEC-RAS uses level pool routing through the reservoir then unsteady flow routing downstream of the dam.(HEC-2010).

Full dynamic wave (unsteady flow) routing through the reservoir pool which was performed for Ribb Dam breach analysis is the most accurate methodology. To model the Ribb Dam reservoir using full dynamic wave routing with HEC-RAS 4.1.0, the pool with one dimensional cross section throughout the reservoir was modeled. The Ribb Dam was modeled with the Inline Structure in HEC-RAS 4.1.0

External Boundary Condition

Boundary conditions need to be defined to solve unsteady flow equations in HEC-RAS. In the case of subcritical flow, boundary conditions in the form of a specified flow are required at the up- and downstream end of the modeled reach. For the routing of a flood hydrograph through a river reach, the upstream boundary condition is the flood hydrograph (HEC-RAS 4.1, 2010).

External Boundary Condition (EBC), which is defined at the most upstream and downstream ends of the river system. An EBC can have either an inflowing or out flowing river reach connected to the node. An external boundary includes an inflow hydrograph, a stage hydrograph, a rating curve or normal depth.

There were two external boundary conditions for Ribb dam breach analysis in this paper. Those are upstream and downstream boundary conditions. For unsteady flow models, upstream boundary conditions are inflow hydrographs, which was a 24 hour half PMF flood event and Downstream boundary condition was set to normal depth slope of 0.002 (0.002 percent slope) about 40kilometers downstream of the Ribb dam.

Internal Boundary Condition

Internal Boundary Condition (IBC), which is defined as internal nodes, when two or more reaches meet. It is a node without a hydraulic structure which is used to connect two or more river reaches with different roughness or bed slopes, or canal expansion occurs. This node is denoted as a junction boundary condition.

For Ribb dam breach modeling there is no river with considerable amount of flow added to the river Ribb until the end of 40 kilometers or the final extent of distance for the study. So no internal boundary conditions are used here in the study.

Initial Condition

The initial conditions in the unsteady routing model for river reach in the steady case can be specified by two of the variables describing the reach’s the water stages at the reach’s end, or one of the stages and the flow discharge in the reach. In the case that the simulation starts from unsteady conditions, the initial conditions must be defined as the combination of the stages at the reach’s end and the flow at one of its ends, or the flows at the reach’s ends and the stage at one of its ends.

There were two initial conditions that entered in hydraulic modeling, HEC-RAS for unsteady flow analysis of Ribb Dam breach. These are Initial flow and initial elevation of water level in the reservoir at the time of starting breach. The initial flow estimated as the design capacity of the spillway, since the dam breach is analyzed for the worst case flood. While, the initial water level was taken as the elevation of maximum pool level (crest of the dam).

Model Limitation

The assumption that the flow in the river channel as well as in the overbanks only occurs in the direction of the longitudinal channel axis limits the applicability of the model to situations, where this assumption is true to a large extent. 1-D flow further implies that for any cross section along the modeled reach, the velocity is constant and the water surface is horizontal. Additionally, HEC-RAS Model has a limitation of labor intensive, time consuming process and stability problems may arise during the analysis.

3.9.3. FLOODPLAIN MAPPING

In this section the description of the aerial extent of flooding downstream area for Ribb dam breach analysis was analyzed also two failure mode scenarios were used; overtopping and piping modes of failure. The reservoir conditions, normal pool (spillway crest level) and maximum storage elevation (top of the dam crest) were considered for downstream consequence of Ribb Dam breach.

Once a dam breach simulation completed in HEC-RAS 4.1.0, it was exported to Arc-GIS and stored for the next inundation delineation process. Arc-GIS tool USACE HEC-GoeRAS 10.1 Arc Map extension and RAS Mapper automatically delineate flood plains or inundation as a post processing function of HEC-RAS 4.1.0.

Floodplain mapping for the downstream of Ribb Dam was performed using the water surface elevations on the XS cut lines, within the limits of the bounding polygon. A polygon refers to a TIN (Triangulated Irregular Network) layer in Arc-GIS that will define a zone that will connect the outer points of the bounding polygons.

4 BREACH PARAMETER ESTIMATION AND MODEL SETUP

4.1. INTRODUCTION

The events being evaluated in this paper are a hydrologic (flood event) and a sunny day or non-hydrologic event failure modes. The necessary information required for the Ribb dam breach estimations are given below:

Reservoir Data:

Table 4.1: Reservoir Levels and capacity at Each Level

Pool Levels	Elevation (m)	Volume (Mm^3)
Stream Bed Level	1873	0.00
Full Supply Level	1940	234
Top of Flood Control	1943	268.30
Top of Dam Crest Level	1946	294

Ribb Dam Embankment Data:

The crest length of Ribb dam is 800 m with 10m width and maximum Height of the dam above river bed is 73m. The average Upstream and downstream embankment Slopes are 2.5H: 1V and 2H: 1V respectively. The embankment material of the dam is earthrock-fill embankment dam with impervious clay core. For Ribb Dam, a chimney drain adopting the core side slopes and composed of fine and coarse filters between the core and the shell have been provided. To safely discharge the seepage water from the chimney filter/drain and to protect erosion of fines from the foundation alluvium, a horizontal drain composed of fine and coarse filters is also provided also, an 8m wide berm is provided at elevation 1910 m asl on the upstream slop for a reason of stability of the upstream slope.

4.2. ESTIMATING DAM BREACH PARAMETERS

In this study as it mentioned before, two modes of failures were analyzed for the study of Ribb Dam breach analysis. Dam breach parameters were estimated using three regression (empirical) equations. These are: - MacDonald &Langridge-Monopolis (1984), Van Thun and Gillette (1990) and Froehlich (2008), are the most commonly applicable regression equations here in dam breach parameter estimation process.

4.2.1. OVERTOPPING MODE OF FAILURE

Here, the failure location for overtopping failure is assumed to be at the centerline (at an elevation of 1873 meters) 0f main channel. From the calculation of breach parameters using those methods mentioned the following results were obtained.

Table 4.2 Breach parameter Results for overtopping

Methods	Bottom Breach Width (m)	Breach side slope (H:1V)	Breach Failure Time (hrs)	Peak Outflow (Q_{max} , M^3/s)
Macdonald - Langridge and Monopolis (1984)	298.79	0.5	3.4	68,835
Froehlich (2008)	256.43	1	1.75	495,214
Von Thun and Gillette (1990)	229.9	0.5	1.71	-

4.2.2. PIPING MODE OF FAILURE

Breach parameters are calculated Using Macdonald and Langridge -Monopolis (1984), Froehlich (2008) and Von Thun and Gillette (1990), these values are summarized in table below.

Table 4.3 Breach Parameter Estimation Results for Piping

Methods	Breach Bottom Width (m)	Breach Side Slope (H:1V)	Breach Failure Time (hrs)	Peak Outflow (Q_{max}, M^3/s)
Macdonald -Langridge and Monopolis (1984)	257.24	0.5	3.32	65, 980
Froehlich (2008)				
At 1946	245.98	0.7	1.67	415,263
At 1892	195	0.7	1.05	423,112
Von Thun and Gillette (1990)	223.23	0.5	1.62	-

4.3.REPRESENTATION (SETUP) OF RIBB RIVER AND DAM IN HEC-RAS MODEL

4.3.1. RIBB RIVER

The Ribb River, which is some 130 km long, has a drainage area of about 1,790 km² and an average annual discharge of 11.6m³/s. The catchment area at the dam site is 685 km^2. The river, which flows generally in a westerly direction and empties into Lake Tana, is one of the main streams flowing into Lake Tana from the east. The Ribb River, with its tributaries, drains the western slope of the high mountainous area east of the town of Debre Tabor, with a peak elevation of approximately 3050 m. In the low and middle reaches of the river, especially in the extensive alluvial plains bordering the lake, 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.

The river flows through several deep and zigzagging valley sections interspersed with wide, flat reaches over this interval. These aspects of the river make the dam breach modeling a little bit challenging to determine the appropriate number and placement of cross-sections for the model as well as the best model time step for the simulations.

The stability of the HEC-RAS model is function of the distance between the cross sections and the time step used in the simulation. The minimum value allowed by HEC-RAS is one minute time step is used for the Ribb dam model to achieve stability of the calculations. Other, larger time steps were tested, but time steps greater than 1 minute are not found to result in stable calculations.

New cross-sections with a shorter distance between them are interpolated between the initial sections at locations where the HEC-RAS solution became unstable. First, all of the initial river cross-sections were spaced by 500m and then further interpolation to 80 m is carried out.

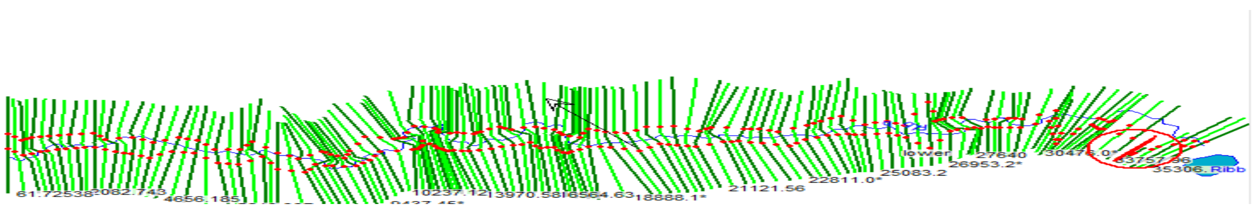


Figure 4.1 Interpolated cross sections.

4.3.2. RIBB DAM BREACH DATA

HEC-RAS allows the modeling of the breach process by entering key data and assumptions regarding the dam, the reservoir and the breach most probable characteristics (see table 4.4 below). Here in this study, Macdonald and Langridge-Monopolis (1984) over estimated and Von Thun and Gillettie under estimated breach parameter, but in the cause of Froehlich- 2008, both the average breach width and time to failure are found in between the two method results, but it is not appropriate method to be selected in cause of Ribb earth-rock fill dam because of shorter time to failure. Since the dam is constructed with impervious clay core it takes time to breach

,thus Macdonald and Langridge -monopolis method is selected as more appropriate method of breach estimation in the cause of Ribb dam. Breach formation time for Ribb embankment dam was estimated to be 3.4 hours for overtopping mode of failure (Macdonald and Langridge Monopolis,1994). The bottom width of the breach is 298.78m with side slopes of breach 1m. In order to have a stable model, the river was considered wet at the beginning of the simulation with an initial flow of 1 m3/sec for HEC-RAS model.

Table 4.4Ribb Dam breach model data for overtopping mode of failure

Item	Value
River station of Dam	34330.54 m
Pilot Flow	1 m3/s (Assume; the river was considered wet at the beginning of the simulation)
Center Station	461.5 m
Final bottom Width	298.79 m
Final Bottom Elevation	1873 m
Left Side Slope	3H:1V
Right Side Slope	3H:1V
Full Formation Time	3.4 hour
Failure Mode	Overtopping
Trigger Failure	WS Elev.

4.4. HEC-RAS UNSTEADY FLOW ANALYSIS PARAMETERS

To model the dam breach process in HEC-RAS, an unsteady flow calculation is performed with asimulation period of 24 hours with the dam breach initiated at the start of the simulation. The

Ribb Dam is modeled as an “inline structure” in the HEC-RAS model and the dimensions of the dam are found from the plan Fig.4.2 and the data in Table 4.3, which is calculated.

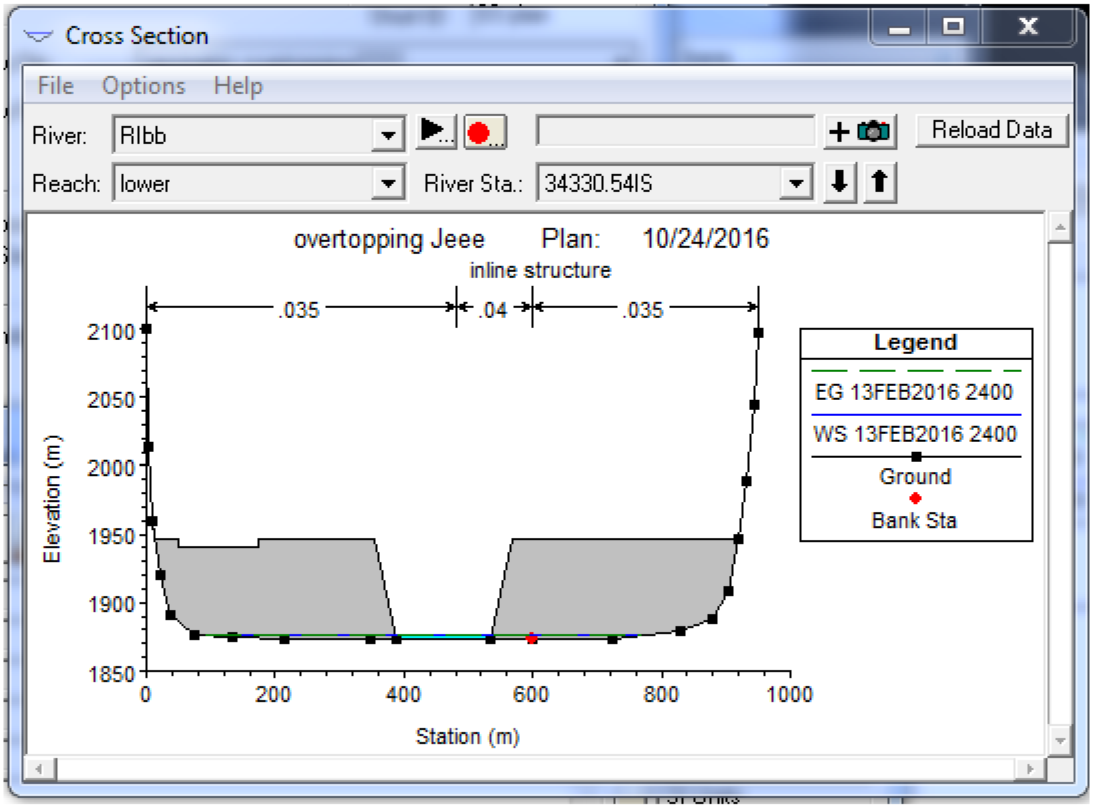


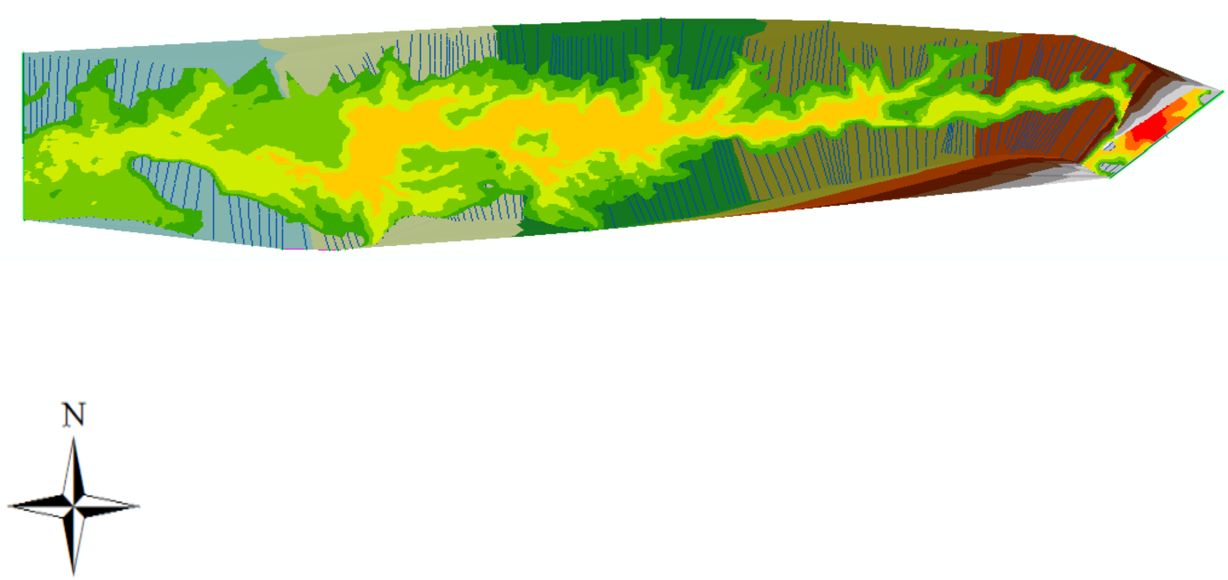
Figure4.2: Dam Breach Plan of Ribb Dam, on Ribb River (HEC-RAS 4.1.)

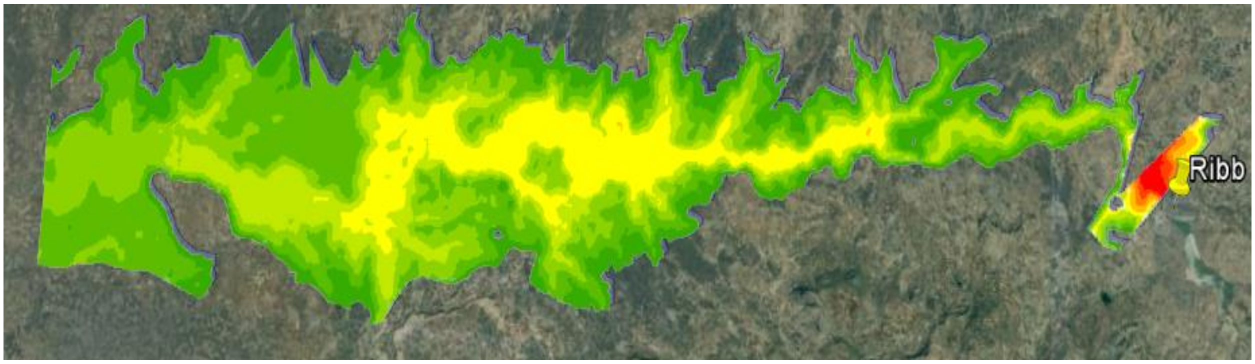
Boundary condition and initial condition data must be entered for the unsteady flow analysis. Boundary conditions at the upstream river reach above the dam was entered as inflow hydrograph of the Ribb River. The downstream boundary condition downstream of of the dam at 40 km was as a normal depth boundary condition with a slope of 0.002 m/m. Also, the initial flow in the basin at the start of the simulation period was set to 1060 m^3/sec for the Ribb River.

4.5 FLOOD INUNDATION MAPPING

Dam-breach flood-inundation maps indicate areas that may be flooded as a result of a dam failure. The maps are used by wide range of end-users for planning and as a response tool to determine the effects of dam failure in downstream areas. In addition, the incremental areas flooded as a consequence of dam failure were considered for a dam classification exercise. For this study, flood inundation maps were generated using HEC-GeoRAS and Arc-GIS. Inundation maps can incorporate elements beneficial to dam safety officials, emergency responders, and mitigation planners. The maps can be used to facilitate communication during an event while at the same time convey relevant information regarding at-risk areas useful for effective long-term mitigation planning. For example, an inundation map may highlight the most vulnerable population areas. Such information is useful for mitigation planners, who may be able to minimize future flood damage via infrastructure projects and rezoning/relocation efforts.

In this study, the PMF and fair weather (Maximum Pool) scenarios maximum water surface elevations obtained from the HEC-RAS model were combined with topographic data extracted in Arc-GIS to create a flood inundation map from Ribb Dam and downstream along Ribb River using the Arc-GIS extension, HEC-GeoRAS. Travel times for the PMF and fair weather breach were tabulated. Finally, the inundation maps for the two scenarios of dam breach would be mapped.





Legend

Max. flood depth (m)

<VALUE>
- 0.00012207 - 6.495804132
- 6.495804133 - 13.3162703
- 13.31627031 - 21.43587287
- 21.43587288 - 37.35029393
- 37.35029394 - 61.38431756
- 61.38431757 - 82.82006836

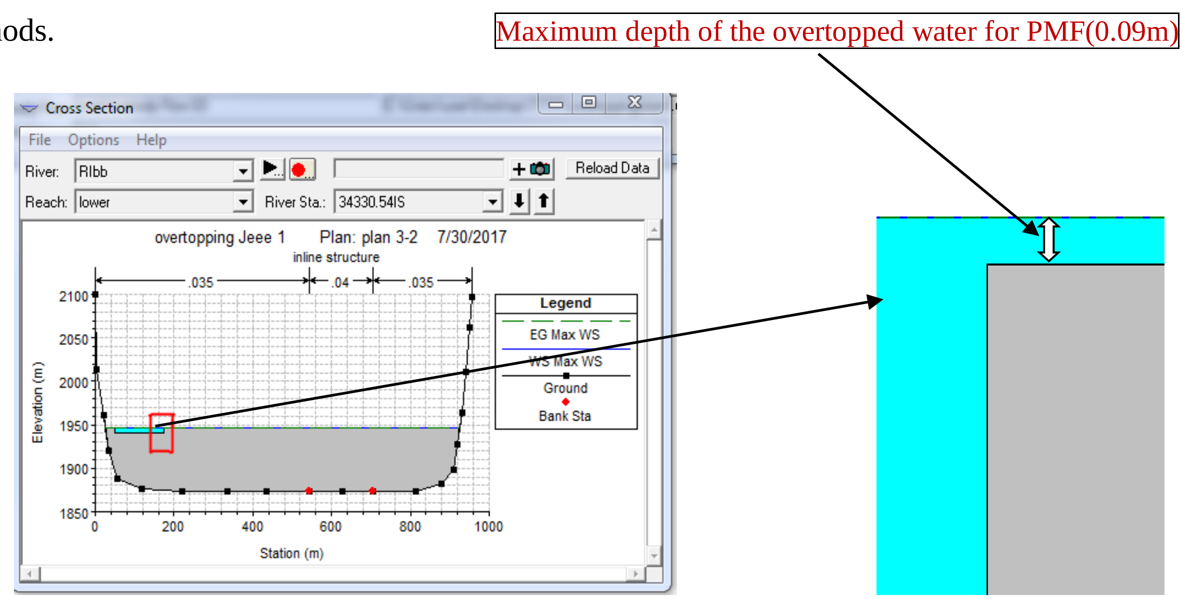
5. RESULTS AND DISCUSSIONS

5.1 RESULTS OF DAM BREACH PARAMETERS

The dam breach parameters were estimated using three most currently used regression equations, i.e. Macdonald and Langridge-Monopolis (1984), Von Thun and Gillette (1990) and Froehlich (2008). For the case of overtopping failure, the bottom breach width for the three methods were 298.78 m, 229.9 m, and 256.43 m respectively. Their breach development times were 3.4 hrs, 1.71hrs, and1 75 hrs. Similarly, the breach width and breach development times for the above three methods in the case of piping failure were: 257.24 m, 223.23 m and 245.98 m and 3.32 hrs, 1.62 hrs, and 1.67 hrs respectively.

5.2 DAM BREACH SIMULATION RESULTS

Ribb River passes any given flow within the range of stages. The shift in stage is a result of the shifts in river meander, channel geometry or bed forms, the dynamic of the hydrograph (how fast the flood wave rises and falls); backwater (backwater can significantly change the stage at a given cross-section for a given flow); Within the distance of 40 km downstream of the dam. From the HEC-RAS model results, the breach outflow hydrographs have been computed for different failure modes (overtopping and piping) breach cases and for flood events at full PMF, 10,000 years return period. The maximum breach outflows and time to peak flow of the two failure scenarios, i.e. overtopping and piping were computed using the results of the above three methods.



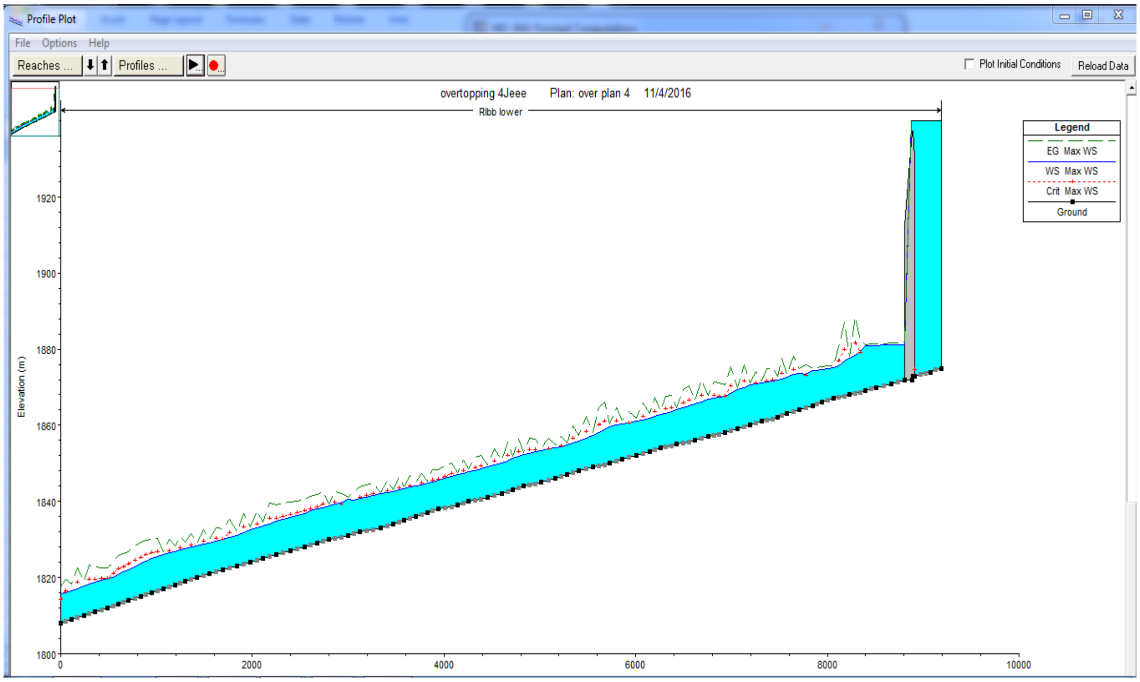


Figure 5.1 Water surface profile at dam

5.2.1 OVERTOPPING MODE OF FAILURE RESULTS

Here in this section the maximum breach outflow and arrival time to peak breach outflow results of critical obtained from HEC-RAS model simulation at dam in case of hydrologic (PMF) breach scenario for are given in tables below .

Table 5.1 Full PMF breach peak outflows for overtopping mode of failure

Methods	Peak flow (cms)	Time to peak (hrs)
Macdonald - Langridge and Monopolis (1984)	22,240.88	17.66
Froehlich (2008)	39,464.46	17.37
Von Thun and Gilletie (1990)	40,672.27	17.4

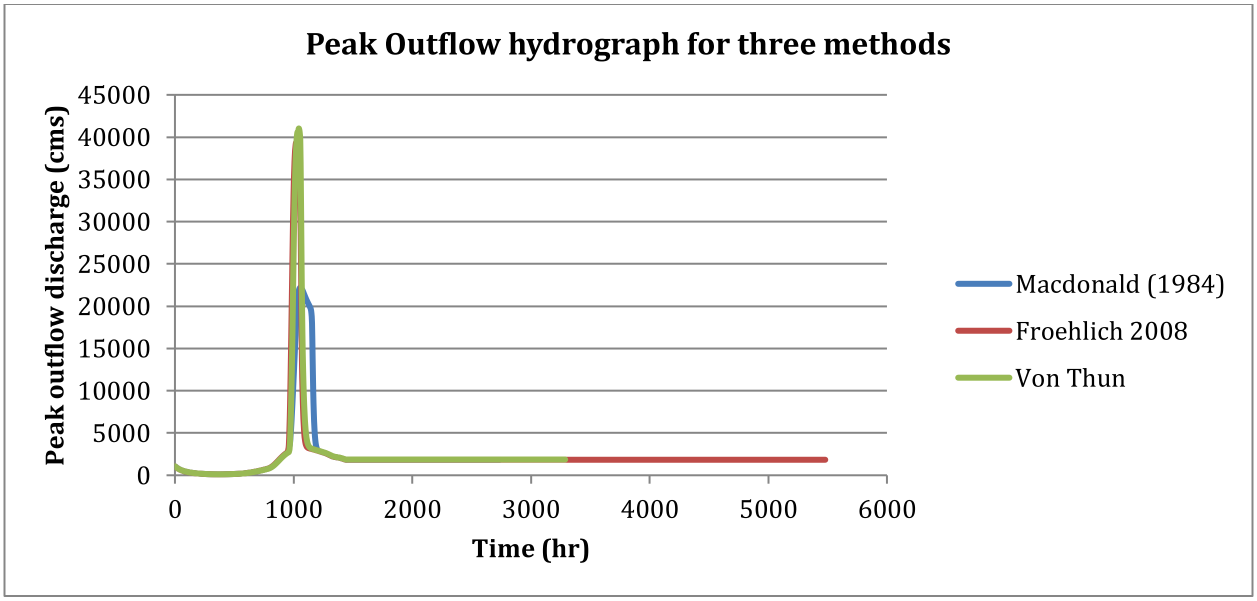


Figure 5.2 Breach outflow hydrographs for the three methods

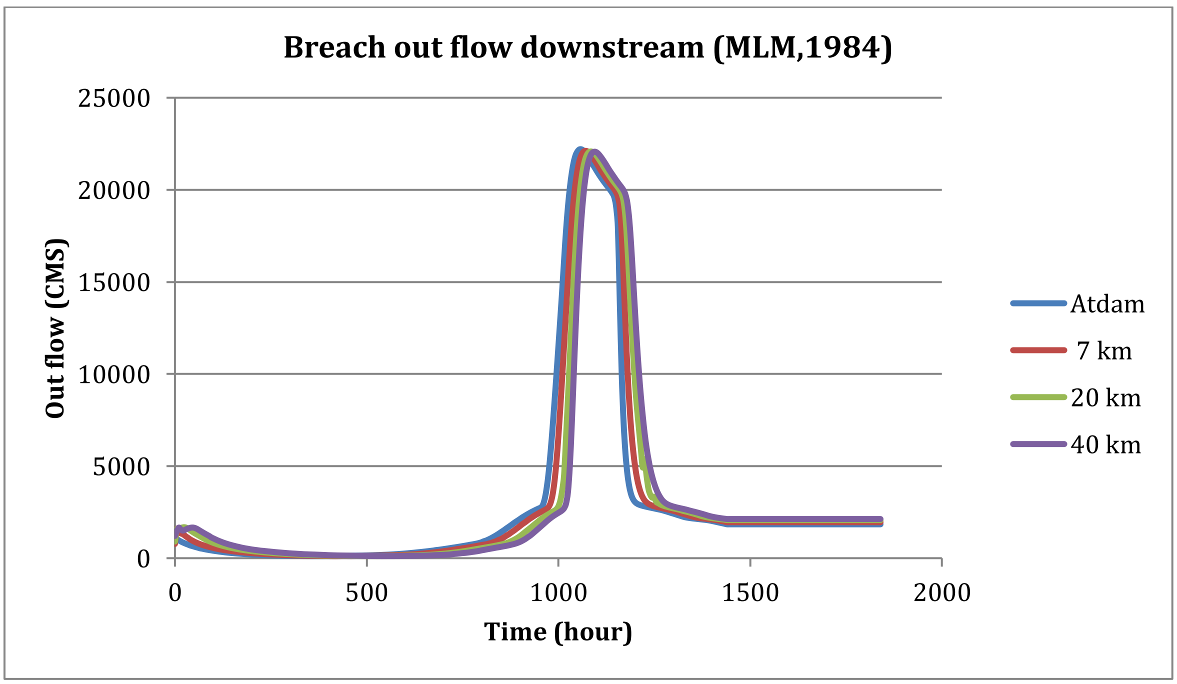


Figure 5.3 Downstream breach out flow using (MLM,1994)

5.2.2 PIPING (SUNNY DAY) MODE OF FAILURE RESULTS

The maximum breach outflow and arrival time to peak breach outflow results obtained from HEC-RAS model simulation at dam for three methods in case the of non-hydrologic (piping) breach scenario are given in tables below.

Table 5.2 Piping failure mode results

Methods	Peak flow (m^3/s)	Time to peak (hours)
Macdonald Langridge-Monopolis (1984)	21,999.93	17.83
Froehlich (2008)	22,604.44	18.5
Von Thun and Gilletie (1990)	39665.24	17.33

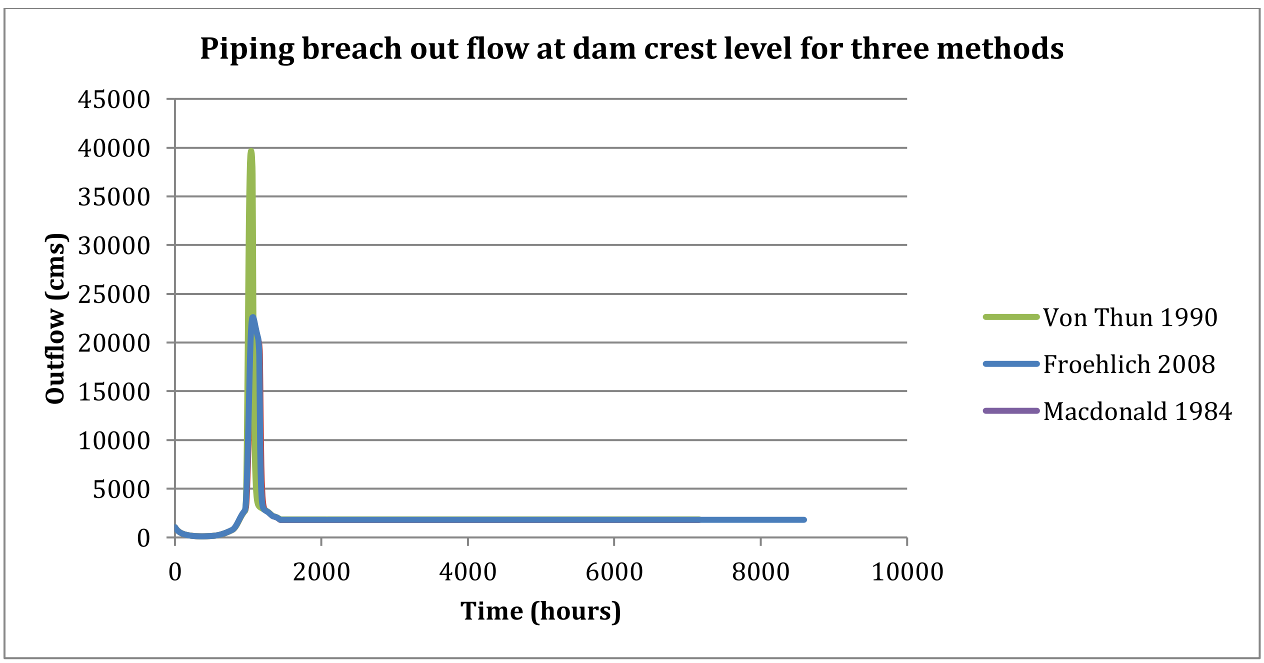


Figure 5.4 Breach out flow for three methods at dam crest level for piping

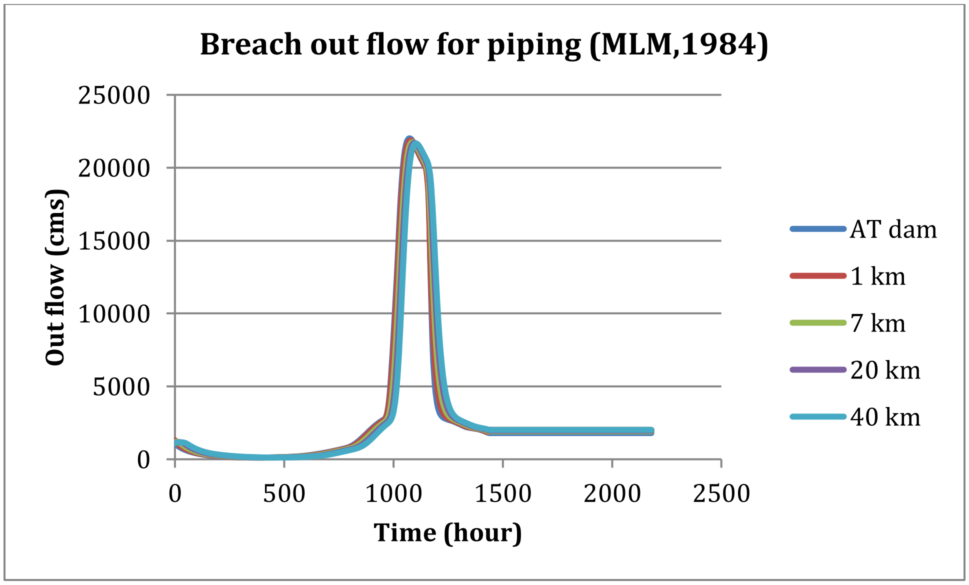


Figure 5.5 Breach outflow downstream for piping

5.3 DISCUSSION

In this paper the breach parameters estimated using three most common methods of regression equations (Macdonald and Langridge Monopolis-1984, Froehlich (2008) and Von Thun and Gillette (1990) are different for both causes of overtopping and piping modes of failures.

For overtopping mode of failure (hydrologic), in cause of the Macdonald and Langridge - Monopolis, this method over estimates the breach bottom width, in case of Von Thun and Gilletie the breach bottom width is less than the two methods in comparison. The result in the case of Froehlich (2008) equations the value becomes in between the two, the minimum during Von Thun equations and maximum during Macdonald and Langridge-Monopolis . The trend of these parametric equations to over-estimate and under estimate the breach size may be attributed to the fact that they are developed based on the assumption of breach forms.

The breach development times for these methods were 3.4 hours, 1.75 hours, and 1.71 hours respectively. When these three results are compared, the result of Macdonald and Langridge-Monopolis the breach width is larger and the breach development time longer than other methods used in this study. Both breach bottom width and breach development time for Von Thun and Gillette method is small.

Ribb dam is high dam and the volume of the water in the reservoir is about 234 and 268.30 million meter cube at FRL and MWL respectively. Therefore, in order to attain its final breach bottom width the breaching process will continue to erode for longer time and may have larger breach width and longer breach development time. Again, in the case of Ribb Dam, since the dam is an earth rock-fill embankment dam consisting of impervious clay core and the amount of water behind the dam is large, and the erosion progress is not suddenly occurred, it can take more time to totally breach to its final width. In the case of piping mode of failure, except a small amount decreasing values in breach bottom width and breach time development, the results were the same with that of overtopping case.

As shown above in table clearly, the result of model simulation in the case of hydrologic or overtopping mode of failure (Probable Maximum Flood) event, the maximum breach outflows and time to peak were computed for three cases. Using Macdonald and Langridge-Monopolis (1984) breach parameter as model input in HEC-RAS , the maximum breach outflow and time to peak which occurred at the dam were 22,240.88 m^3/s and 17.66 hours respectively. From Von Thun and Gillette (1990) also 40,672.37 m^3/s. peak outflows and 17.4 hours time to peak were calculated at the dam location. Again from Froehlich (2008), the maximum breach outflow which occurred at the dam and time to peak were 39,464.46 m^3/s and 17.37hours.

Thus, peak breach outflow results for each of the three methods were different with some amount. Breach peak outflow for Macdonald Langridge Monopolis is smaller than the two because the higher the time to failure corresponding to given breach the smaller the out flow discharge and vice versa. From the time to peak breach outflow of the three methods, one can consider the effect of time to peak in the process of warning and emergency action plan.

6. DAM BREACH SENSITIVITY ANALYSIS

6.1. PURPOSE

Performing a sensitivity analysis of dam breach parameters helps to study the relationships among the parameters that are involved in dam breach and flood propagation processes. The methods applied in conducting the sensitivity analysis of dam breach parameters include hydrologic and hydraulic routing techniques. These techniques help to predict dam-breach outflow hydrographs and flood wave propagation, and provide the information regarding the wave front arrival time, inundated area, and flow depth.

In this study, sensitivity analysis of dam breach parameters using Ribb dam as a testing basis, involved estimating the key parameters, time to dam failure, side slope of breach, downstream Manning coefficients, and channel bed slopes. HEC-RAS 4.1.0 was applied in unsteady flow routing through the downstream reaches to carry out the study for overtopping were used for this analysis.

The sensitivity analysis here in this study was prepared to evaluate the relative effects of dam breach and downstream river parameters on the peak discharges at the dam site and specified locations in the downstream channel.

- To investigate the sensitivity of maximum discharges for given changes in time to dam failure in hours (TFH), and side slope of dam breach (SS) at the dam site.

6.2 RESULTS AND DISCUSSION

The sensitivity analysis using Ribb Dam as a testing basis involved testing a number of dam breach parameters. The parameters defined for the reservoir and river component of the analysis were prepared based on existing data and some empirical formulas developed With the aid of hydrologic and hydraulic modeling software, reservoir and river flow routings were carried out to establish relationships among the characteristics influencing a peak flow at the dam and specified location in the downstream. The findings were discussed in the following sub-sections.

Maximum breach discharge (Q_{max}), breach development time in hours (TFH), and side slope of breach (SS or 1: Z) were the three principal parameters analyzed at the dam site. . The second half, river component, of sensitivity analysis focused on identifying and analyzing the influences of change in downstream reach parameters, i.e. Manning coefficients and channel bed slopes, on peak flows for a given breaching outflow hydrograph.

1. Time to dam failure (TFH) versus maximum breach discharges (Q_{max}) for a given side slope of breach

Table 6.1. Maximum discharge, percentage and time to dam failure at dam site

Time of breach development(hr)	Side slope of breach(1:Z)	Maximum Q(CMS)
0.5 T_o	1:Z_o	41247.97
0.75 T_o	1:Z_o	28804.72
T_o	1:Z_o	22415.38
1.15 T_o	1:Z_o	20502.52
1.18 T_o	1:Z_o	19577.27
Note: T_o and Z_o represent typical values and equal to 3.4 hr and 0.5 respectively.		

In this sensitivity analysis, five different TFH were used for HEC-RAS dam-breach unsteady flow sensitivity analysis and resulted in five dam breaching outflow hydrographs. Figure 5.1. – five dam breach hydrographs for a given side slope of breach (0.5:1) with five different TFH. Table 6.1.Showed the maximum breach discharge varied with TFH with a given side slope Z and maximum discharge percentage.

Based on results in Table 6.1., the peak discharge decreased when TFH increased. A 25 % reduction in TFH (0.5 T_o to 0.75T_o) resulted in 55.5 % increase in peak discharge at the dam site.

Whereas, a 28.5 % increase in discharge found from (0.75 T_0 to T_o), 8.3 % reduction of discharge is resulted from (T_0 to 1.15 T_0) and 4.17 % reduction also resulted from (1.15T_o to 1.18T_o) in peak discharge at the dam site. This trend showed in the following figures.

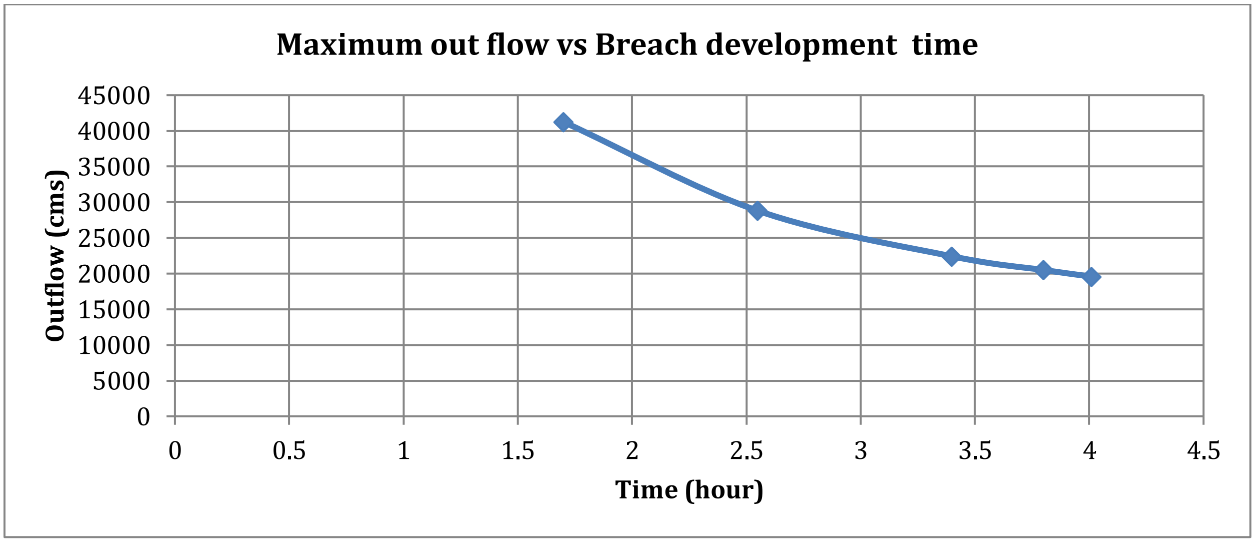


Figure 6.1. – Time to dam failure and maximum discharge at dam site

The above results confirmed that predicting dam breach outflow hydrographs was dependent on and sensitive to a minor change in dam breach development time.

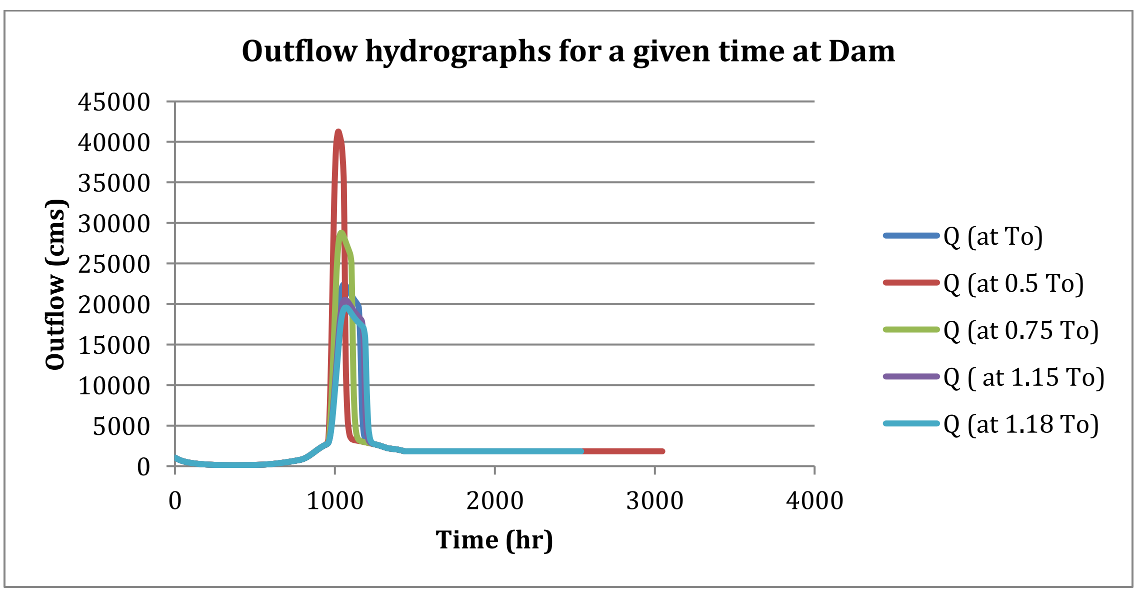


Figure 6.2. Five dam breaching outflow hydrographs for a given side slope

2. Side slope of breach (SS) versus maximum breach discharges (Q_{max}) for a given time to dam failure

The HEC-RAS 4.1.0 for dam-breach analysis method was applied and resulted in the following summarized data for the two variables, Q max and SS, at the dam provided that TFH remained the same. Appendices presented the detailed outputs of HEC-RAS reservoir routings.

The shape parameter (Z) identified the side slope of the breach, i.e., 0.5 vertical: Z horizontal and breach shape is assumed as trapezoidal.

Table 6.2 Side slope of breach (SS) versus maximum breach discharges

Time to breach (hr)	Side slope of breach	Maximum discharge (cms)
T_o	$0.15Z_o$	22575.71
T_o	$0.25Z_o$	22531.79
T_o	Z_o	22419.06
T_o	$1.5\ Z_o$	22019.46
T_o	$2\ Z_o$	21886.04

Note: T_o and Z_o represent typical values equal to 3.4 and 0.5 respectively.

Results indicated that the maximum discharge increased as the side slope of breach decreased at the dam site. A 30 % decrease in side of slope of breach ($0.15Z_o$ to Z_o) resulted in 0.88 % reduction in maximum discharge at the dam relative to original data. Whereas, a 25% increase in SS (Zo to 1.5 Zo) produced 0.48 % decrease in maximum discharge at the dam site. The increments of percent change in peak flow were very small compared to the corresponding percent changes in SS. This observation was further illustrated in Figure below.

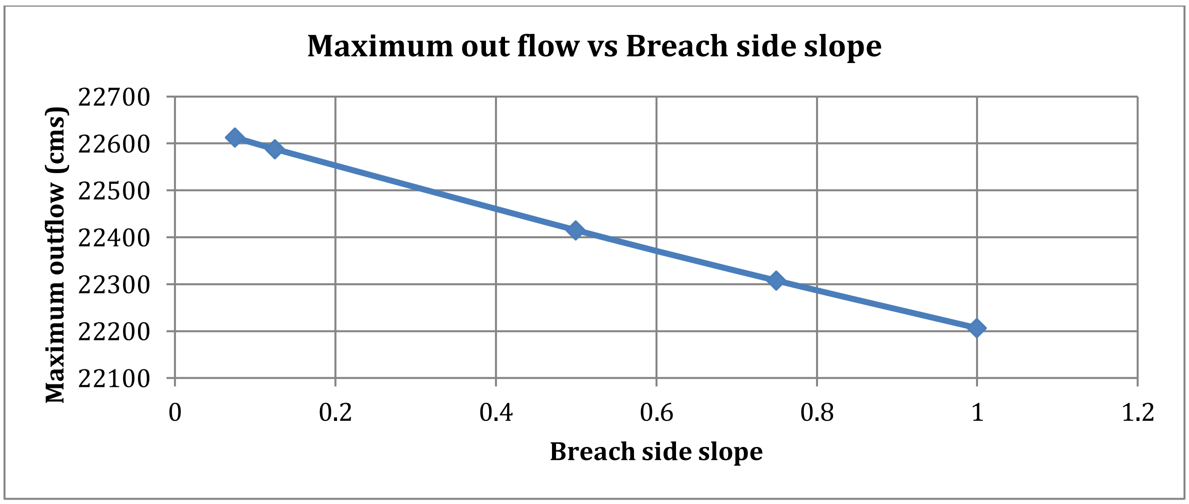


Figure 6.3 – Side slope of breach and maximum discharge at dam site

3. Relative effects of time to dam failure (TFH) and side slope of breach (SS) on maximum breach discharges (Q_{max})

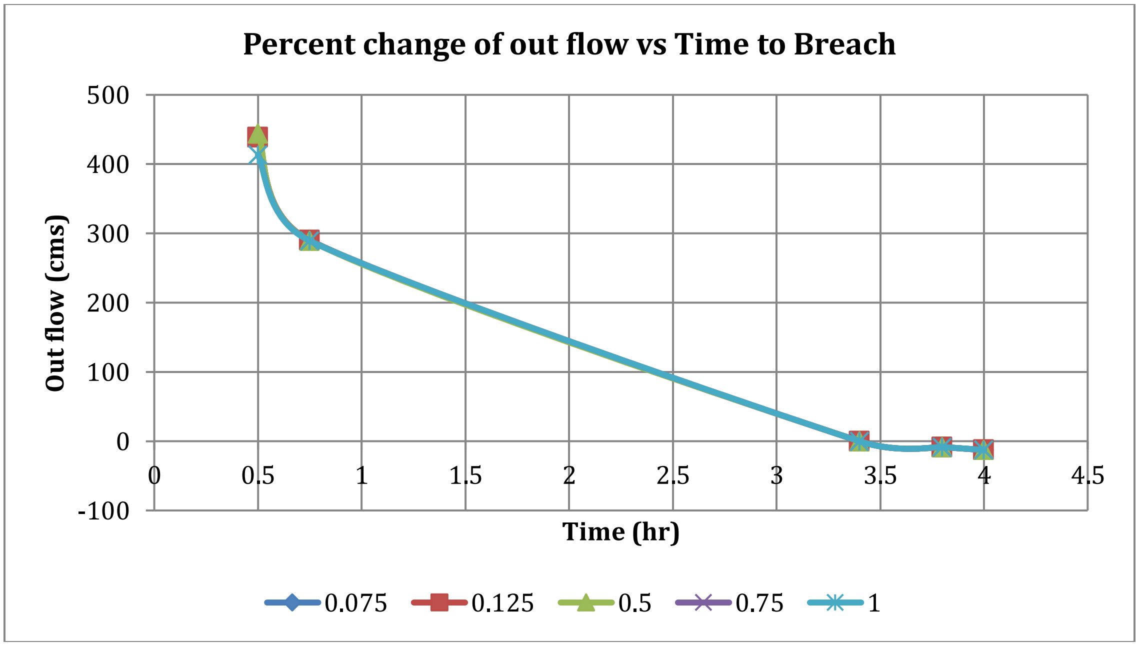


Figure 6.4 Percent change in outflows versus time to failure

4. Manning coefficient versus peak flows in existing channel for specified time to dam failure

The downstream river routings were carried out by HEC-RAS computer program. The HEC-RAS unsteady flow routing was run to investigate the overall impacts of change in time to dam breach on the downstream reach peak flows for given manning coefficients at specified locations between reaches.

Table 6.3 Downstream peak flows for five TFH breaching outflow hydrographs

River station (km)	Assumed time to dam failure for inflow hydrographs					
	0.5 To	0.75 To	To	1.15 To	1.18 To	
	Peak flow Q $_{max}$ (m^3/s)					
	Q(m^3/s)	Q(m^3/s)	Q (m^3/s)	Q (m^3/s)	Q (m^3/s)	Manning coefficient (N_o)
0	41247.97	28804.72	22415.38	20502.52	19577.27	0.04
1	41248.12	28799.42	22417.21	20499.48	19577.27	0.04
1.5	41190.93	28789.65	22418.72	20504.58	19577.34	0.04
2	41112.22	28762.29	22415.13	20508.78	19573.07	0.04
2.5	40981.45	28716.79	22402.07	20506.67	19561.55	0.04
3	41009.82	28676.01	22379.76	20499.24	19539.89	0.04
4	41019.60	28693.38	22340.8	20474.33	19527.10	0.04
5	41037.67	28700.59	22353.66	20457.88	19533.66	0.04

As shown in Table 6.5 five different unsteady flow routings were done using HEC-RAS to produce peak flows at respective river stations. Similarly, Figure 6.5(annex) shows the five peak flow profiles generated from five breaching outflow hydrographs that had five different dam breach development times at the dam site. The tabular and graphical analyses of the outputs revealed that the peak flows starting at the dam run in a similar manner all the way to downstream reach end. It was obvious that smaller time to dam failure resulted in a higher dam breach hydrograph.

7. EMERGENCY ACTION PLAN (EAP)

7.1. PURPOSE

The purpose of the Emergency Action Plan (EAP) for this study is to safeguard lives and secondarily to reduce property damage in the event of Ribb dam failure. To carry out this work, the EAP contains :

1) Procedures to monitor Ribb Dam periodically and during flood warnings issued by the National Meteorological Agency;

2) To notify appropriate body of Amhara Regional State Government a potential dam failure; and

3) Warn and evacuate the isolated residences at risk. These procedures are to supplement and be used in conjunction with Ethiopian Federal Government Concerned Body (Ministry of Water, Irrigation and Energy), the owner of the Ribb dam.

7.2. FLOOD DESCRIPTION

Failure of the Ribb dam could cause significant damage to 20,000 ha irrigation field which is found around 28 km downstream of the dam and some small towns may affected.

7.3 OPERATING PROCEDURE

1. The dam will be inspected periodically each year during the construction time and after construction for maintenance and suffering signals.

2 . The dam observer will inspect the dam when the National Metrological Agency (NMA) issues a flood warning for the area and also will note and record water levels in reservoir and the rate at which the pool is rising. If the dam shows signs of internal piping (muddy seepage exiting the downstream embankment), erosion, slope failures, blocked spillways, or other, the dam observer will call the local or regional concerned body to block downstream roads and warn downstream residences. The dam observer may contact the Ministry of Water, Irrigation and Energy (owner of the dam) or his designated engineer to provide assistance. If the pool level

rises too within small range levels of the dam crest, the dam observer will contact the local concerned body or Ministry of Water, Irrigation and Energy (owner of the dam) or his designated engineer to provide assistance to dispatch police to block downstream roads and warn downstream residences.

Name of Dam :Ribb Irrigation Project Dam

Signatures of Persons Involved in Emergency Action Plan

Dam Owner

By ________________Date

Ministry of Water, Irrigation and Energy

Typed Name: ______________

Title: ________________________

Phone: ______________________

(Day): _________________

(Night)________________

Local Department of Emergency Operations

By: ________________ Date

____

Typed Name:_________________

Title: _________________________

Phone: _______________________

(Day)_________________

(Night)________________

Local or Regional Police

By: _______________Date ____________________

Typed Name: _________________

Title: ________________________

Phone-

(Day):_________________

(Night):________________

Department of the Irrigation and Drainage Projects

By: _____________ Date

Typed Name: _________________

Title: ________________________

Phone-Day):__________________

(Night):________________

Owner's Engineer

By: ______________________ Date____________________

7.4 PREVENTIVE ACTIONS

If time allows, contact the design and consultant party of the project Amhara Water Works and Supervision Enterprise (AWWDSE) or the representative Engineer and the Federal Ministry of Water, Irrigation and Energy, Department of Irrigation and Drainage Projects for advice on preventative actions. Listed below are potential emergency actions which may prevent or delay the failure of the dam. They should be considered based on site-specific conditions, as well as the risk of failure and risk to employees.

Possible actions to be taken in the event of: Imminent Overtopping by Flood Waters:

- Open drain or flood gates to maximum capacity.
- Place sand bags along the dam crest to increase freeboard.
- Place riprap or sandbags in damaged areas of dam.
- Provide erosion protection on downstream slope by placing riprap or other appropriate materials.
- Divert flood waters around dam if possible

Erosion of dam by seepage or piping through the embankment:

- Plug the seepage with appropriate material such as (riprap, grass bundles, sandbags, soil, or plastic sheeting).
- Lower the reservoir level until the flow decreases to a non-erosive velocity or stops leaking.
- Place sand and gravel filter over the seepage exit area to minimize loss of embankment soils.
- Continue lowering the reservoir level until the seepage stops or is controlled. Refill reservoir to normal levels only after seepage is repaired.

7.5 SUPPLIES AND RESOURCES

In an emergency situation, equipment and supplies may be needed on short notice. The following supplies and resources may be needed during an emergency: earthmoving equipment, sand and gravel, sandbags, riprap, pumps, pipe, laborers.

List of Contractors:

It will be the responsibility of the owner (Ministry of Water, Irrigation and Energy) to maintain the list of contractors and consultants that may be contacted during an emergency condition for equipment materials, and repairs, engineers. For each contractor on the list, the following information is needed:

- ✓ Contractor name
- ✓ Contact person.

    -Address.

    -Phone number.

- ✓ Equipment & repair supplies available.-
- ✓ Arrival time to dam

Contractor: Amhara Water Works Construction Enterprise

Contact person: ___________________________________

Phone No: ___________________

Address: _________________________

Services contracted for: _______________________________________

Consultant: Amhara Water Works Design and Supervision Enterprise

Contact person: ___________________________________

Phone No: ___________________

Address: _________________________

Services contracted for: ___________________________

8. CONCLUSION AND RECOMMENDATION

8.1 CONCLUSION

From breach parameters results calculated above using regression methods (Macdonald and Langridge-1984, Von Thun and Gillette-1990 and Froehlich-2008) the Macdonald and Langridge-1984, was more preferred method for Ribb dam breach parameter estimation. Breach bottom width estimated using this method is greater than that of the two methods and also time to failure. The simulated result also showed that the arrival time of the maximum flood from the dam breach is longer because Ribb dam is an earth rock fill embankment dam with impervious clay core it takes time to beach .This also simplifies, it gives some time for the emergency works in case the dam failure, to make warning and evacuate the downstream people. Therefore, for Ribb earth rock-fill embankment dam Macdonald and Langridge Monopolis ,1984 method is more appropriate and reasonable to estimate the breach parameters and to use as an input for hydraulic modeling purpose.

8.2 RECOMMENDATION

Ribb dam is currently under construction and almost 90 % is completed. As a simulation results shown that the Ribb Dam will overtop during the hydrologic event of full PMF . Therefore, the appropriate solution should be given for this problem from now before the construction is completed. The solution may be adding a free board by 1m or it may be increasing the capacity of the spillway. Therefore, the owner of the dam (Ministry of Water, Irrigation and Energy) with the engineering party or consultant (Amhara Water Works Design and Supervision Enterprise), should properly check and take the possible action for the solution.

The breach outflow flood inundates the downstream large irrigation fields, small town and kebeles. Therefore, according to the emergency action plan prepared, every stake holders should participate in emergency works in case the dam will fail. These stakeholders are the owner (MWIE), the contractor, the consultant, National Meteorological Agency, regional or local police, and others.

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

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APPENDIX 1 HYDRAULIC IMPUT DATAS FOR HEC-RAS MODEL

Table 1: Ribb dam reservoir capacity

Elevation (m)	Volume (10^3 m^3)	Change in Storage m^3	Average Elevation (m)	Head (h, m)	Discharge (m^3/s)	hf=Kv^2/2g	H-hf	Discharge (m^3/s)	Period (day)
1940	233,699								
1939	223,919	9,780	1939.5	51.5	202.30	20.84	30.66	20.00	5.66
1938	214,474	9,444	1938.5	50.5	200.33	20.44	30.06	20.00	5.47
1937	205,336	9,139	1937.5	49.5	198.34	20.03	29.47	20.00	5.29
1936	196,427	8,909	1936.5	48.5	196.32	19.63	28.87	20.00	5.16
1935	188,079	8,348	1935.5	47.5	194.29	19.22	28.28	20.00	4.83
1934	179,995	8,084	1934.5	46.5	192.23	18.82	27.68	20.00	4.68
1933	172,249	7,745	1933.5	45.5	190.15	18.41	27.09	20.00	4.48
1932	164,809	7,441	1932.5	44.5	188.05	18.01	26.49	20.00	4.31
1931	157,634	7,175	1931.5	43.5	185.93	17.60	25.90	20.00	4.15
1930	150,697	6,937	1930.5	42.5	183.78	17.20	25.30	20.00	4.01
1929	143,990	6,707	1929.5	41.5	181.60	16.80	24.70	20.00	3.88
1928	137,514	6,476	1928.5	40.5	179.40	16.39	24.11	20.00	3.75
1927	131,263	6,251	1927.5	39.5	177.17	15.99	23.51	20.00	3.62
1926	125,226	6,038	1926.5	38.5	174.92	15.58	22.92	20.00	3.49
1925	119,408	5,817	1925.5	37.5	172.63	15.18	22.32	20.00	3.37
1924	113,811	5,598	1924.5	36.5	170.31	14.77	21.73	20.00	3.24
1923	108,416	5,394	1923.5	35.5	167.96	14.37	21.13	20.00	3.12
1922	103,221	5,195	1922.5	34.5	165.58	13.96	20.54	20.00	3.01
1921	98,219	5,002	1921.5	33.5	163.16	13.56	19.94	20.00	2.89
1920	93,393	4,826	1920.5	32.5	160.71	13.15	19.35	20.00	2.79
1919	88,729	4,664	1919.5	31.5	158.22	12.75	18.75	20.00	2.70
1918	84,217	4,512	1918.5	30.5	155.69	12.34	18.16	20.00	2.61
1917	79,861	4,356	1917.5	29.5	153.11	11.94	17.56	20.00	2.52
1916	75,655	4,205	1916.5	28.5	150.49	11.53	16.97	20.00	2.43
1915	71,584	4,071	1915.5	27.5	147.83	11.13	16.37	20.00	2.36
1914	67,640	3,944	1914.5	26.5	145.12	10.72	15.78	20.00	2.28
1913	63,818	3,822	1913.5	25.5	142.35	10.32	15.18	20.00	2.21
1912	60,119	3,700	1912.5	24.5	139.53	9.92	14.58	20.00	2.14
1911	56,535	3,583	1911.5	23.5	136.66	9.51	13.99	20.00	2.07
1910	53,061	3,475	1910.5	22.5	133.72	9.11	13.39	20.00	2.01
1909	49,692	3,369	1909.5	21.5	130.71	8.70	12.80	20.00	1.95
1908	46,429	3,263	1908.5	20.5	127.64	8.30	12.20	20.00	1.89
1907	43,272	3,157	1907.5	19.5	124.48	7.89	11.61	20.00	1.83
1906	40,215	3,057	1906.5	18.5	121.25	7.49	11.01	20.00	1.77
1905	37,254	2,961	1905.5	17.5	117.93	7.08	10.42	20.00	1.71
1904	34,391	2,862	1904.5	16.5	114.51	6.68	9.82	20.00	1.66
1903	31,630	2,761	1903.5	15.5	110.99	6.27	9.23	20.00	1.60
1902	28,976	2,654	1902.5	14.5	107.35	5.87	8.63	20.00	1.54
1901	26,432	2,544	1901.5	13.5	103.58	5.46	8.04	20.00	1.47
1900	24,007	2,425	1900.5	12.5	99.67	5.06	7.44	20.00	1.40
1899	21,707	2,300	1899.5	11.5	95.60	4.65	6.85	20.00	1.33
1898	19,527	2,180	1898.5	10.5	91.35	4.25	6.25	20.00	1.26
1897	17,466	2,061	1897.5	9.5	86.89	3.84	5.66	20.00	1.19
1896	15,528	1,938	1896.5	8.5	82.19	3.44	5.06	20.00	1.12
1895	13,715	1,812	1895.5	7.5	77.20	3.04	4.46	20.00	1.05
1894	12,018	1,697	1894.5	6.5	71.87	2.63	3.87	20.00	0.98
1893	10,427	1,591	1893.5	5.5	66.11	2.23	3.27	20.00	0.92
1892	8,945	1,482	1892.5	4.5	59.80	1.82	2.68	20.00	0.86
1891	7,580	1,365	1891.5	3.5	52.74	1.42	2.08	20.00	0.79
1890	6,341	1,240	1890.5	2.5	44.57	1.01	1.49	20.00	0.72
1889	5,220	1,121	1889.5	1.5	34.53	0.61	0.89	20.00	0.65
1888	4,216	1,003	1888.5	0.5	19.93	0.20	0.30	15.38	0.76
Total (days required to empty the Reservoir upto 1899 i.e 90%)									122.68
Month									4.09

APPENDIX 2 : DAM BREACH ROUTED OUT FLOW HYDROGRAPHS FOR BOTH FAILURE SCENARIOS

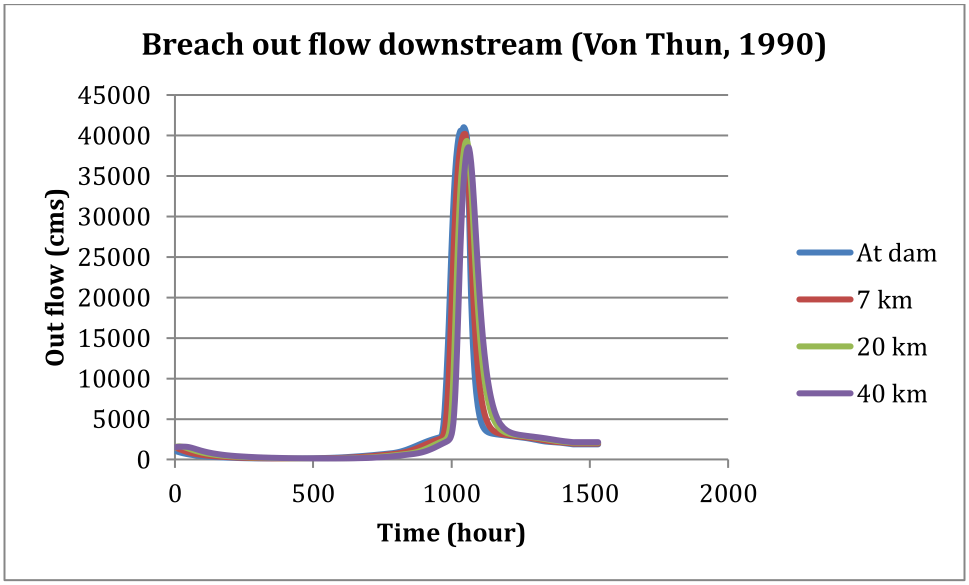

Figure 1: Breach out flow for overtopping

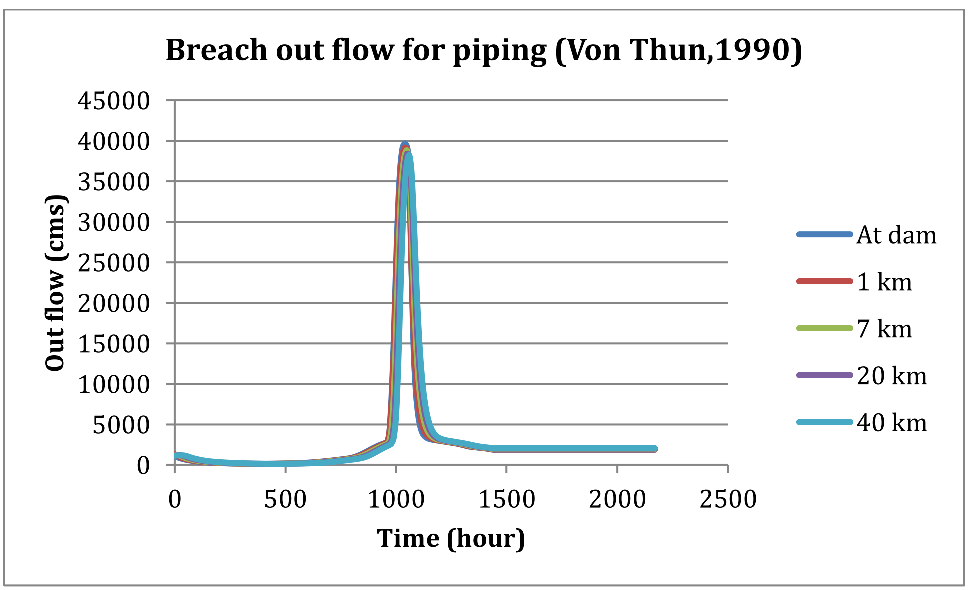

Figure 2 : Breach out flow for piping

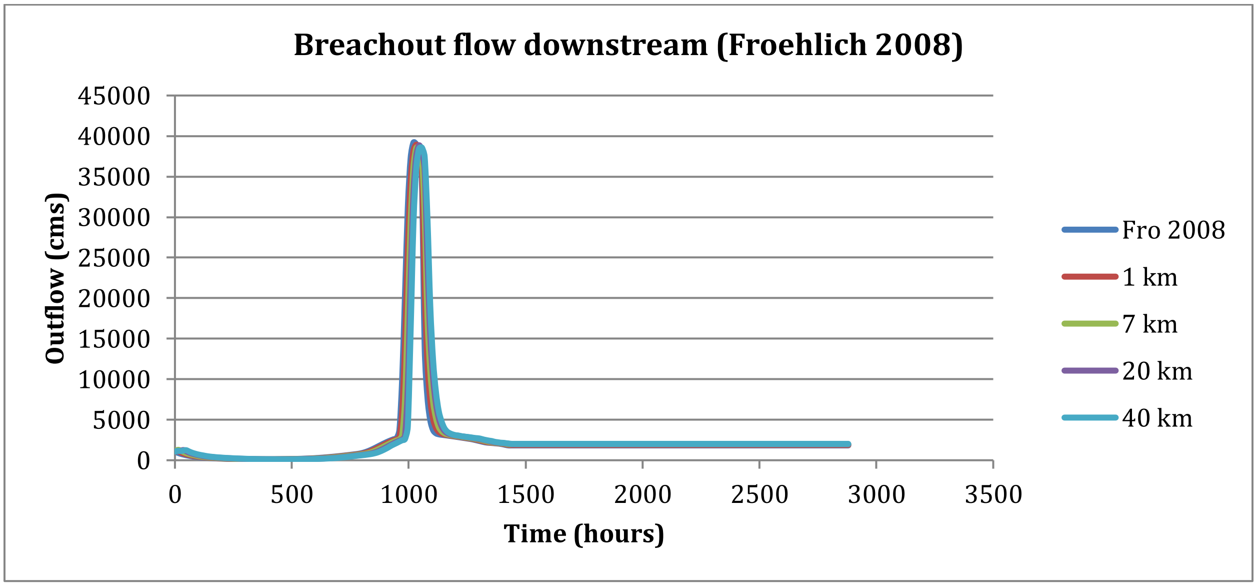

Figure 3: Breach out flow for overtopping

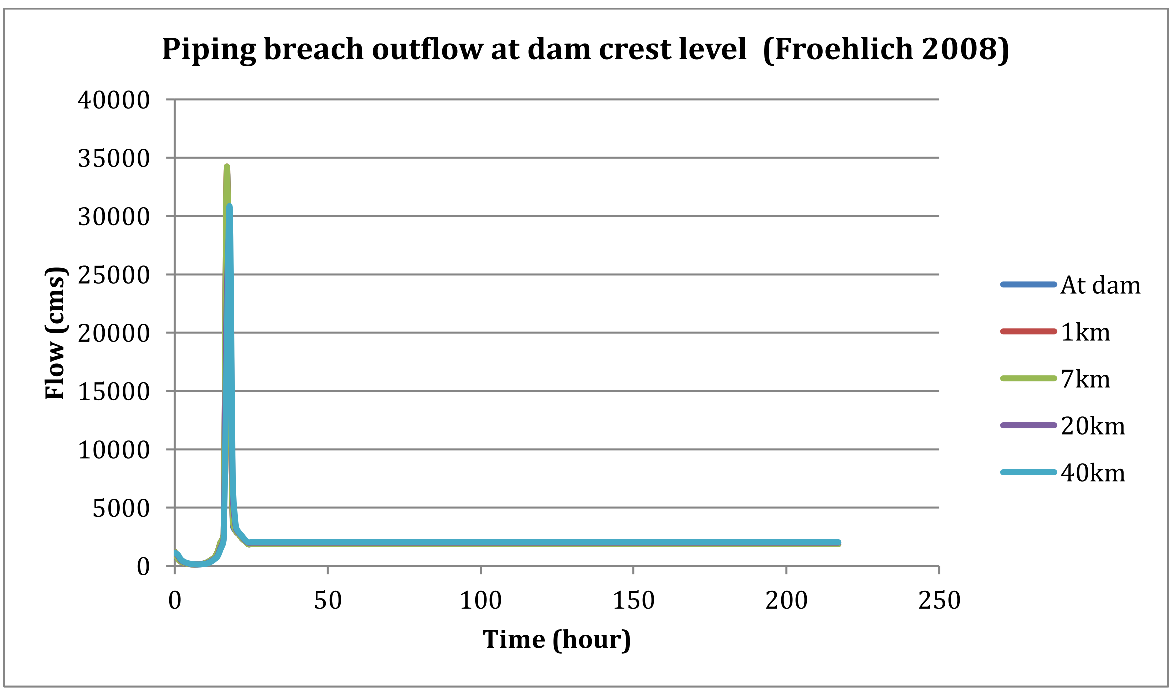

Figure 4 : breach out flow for piping

Table 7.1. Maximum discharge, percentage and time to dam failure at dam site

Time of breach development(hr)	Side slope of breach(1:Z)	Maximum Q(CMS)	Change in Q_{max} (%)
0.5 T_o	1:Z_o	41247.97	84
0.75 T_o	1:Z_o	28804.72	28.5
T_o	1:Z_o	22415.38	0.0
1.15 T_o	1:Z_o	20502.52	-8.53
1.18 T_o	1:Z_o	19577.27	-12.7
Note: T_o and Z_o represent typical values and equal to 3.4 hr and 0.5 respectively.			

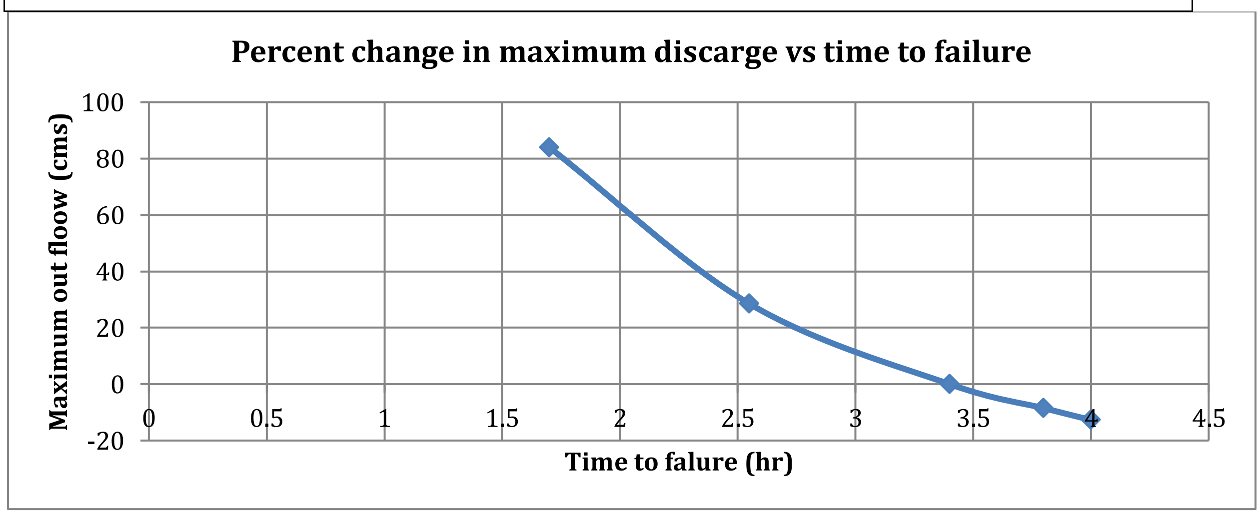

Figure 7.2 – Percent change in max discharge and time to dam failure at dam site

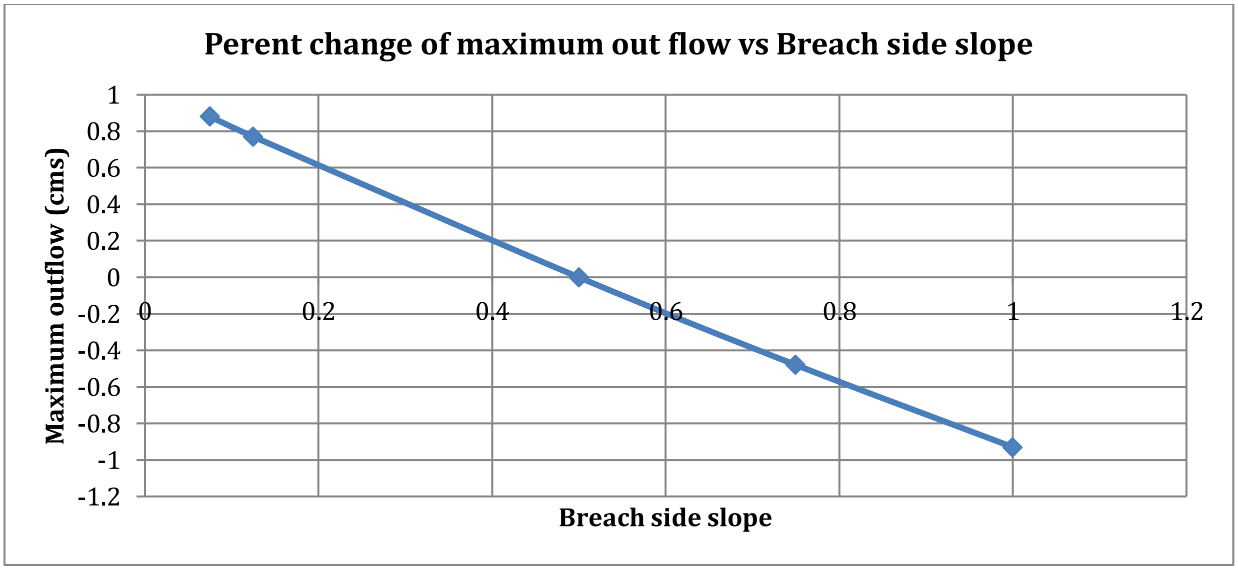

Figure 7.5 – Percent change in max discharge and side slope of breach at dam site

Table 7.3 Relative effects of time to dam failure (TFH) and side slope of breach (SS)

Breach time (hr)	0.15 Zo	0.5 Zo	Zo	1.5Zo	2Zo
	Percentage of Peak outflows (m^3/s)				
0.5To	87.2	87.6	89	87.6	87.7
0.75To	30.5	30.4	29.9	30.2	29.2
To	0.0	0.0	0.0	0.0	0.0
1.15To	-48.6	-48.5	-48.8	-48.5	-48.6
1.18To	-51.2	-50.8	-51	-50.9	-51
Note : To and Zo refers 3.4 and 0.5 respectively					

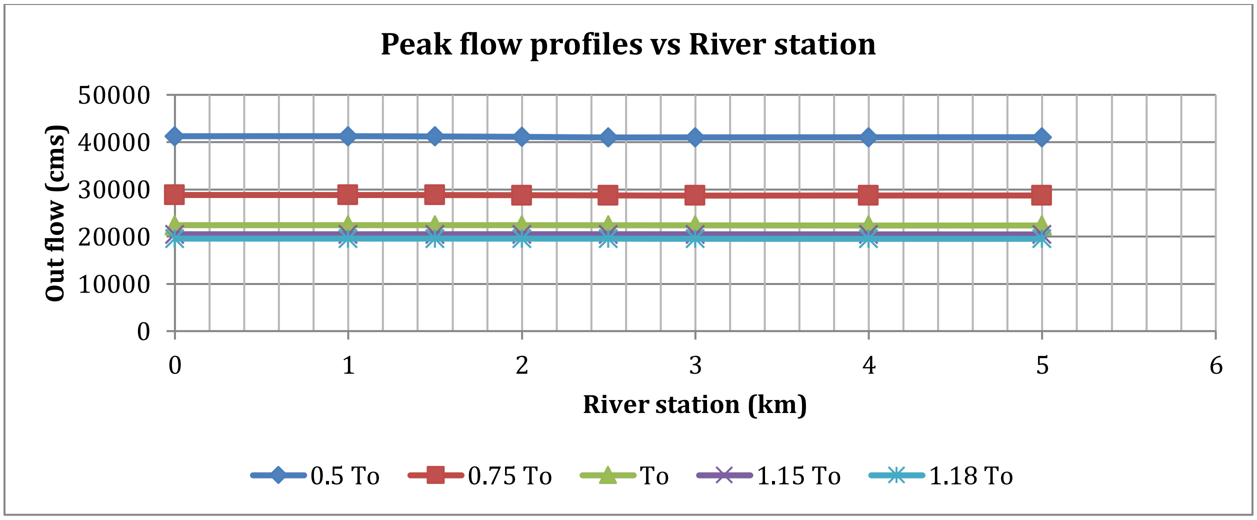

Figure 6.5 Peak flow profiles for five TFH breaching outflow hydrographs