Addis Ababa University

School of Graduate Studies

Addis Ababa Institute of Technology

Dam Breach Analysis and Remedial Measure for Gilgel Gibe dam

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Degree of Master of Science in Civil Engineering

(Major in Hydraulic Engineering)

By

Million Teshome Legesse

Advisor: - Dr.Ing. Asie Kemal

December, 2016

Certification

This is to Certify That This Thesis Entitled **Dam Breach Analysis and Remedial Measure for Gilgel Gibe Dam** Is Genuine, Done and Submitted By **Million Teshome Legesse**

In Partial Fulfillment of the Requirements for the Award of Degree of Master of Science (M.Sc) In Civil Engineering (Major in Hydraulic Engineering)

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Addis Ababa University

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Addis Ababa Institute of Technology

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ABSTRACT

This study mainly focuses on dam breach analysis and remedial measure of Gilgel Gibe dam. Gilgel Gibe Dam is a rock fill dam with 37 m height and 1,704m crest long and designed to generate hydro electric power. Generation of hydroelectric power has crucial part in the development of the country. In order to have medium power generation of Gibe as it is, it is necessary to study dam breach analysis and take remedial measures if it is needed. Generally this work focus on predicting the breach outflow hydrograph of Gilgel Gibe and routing it through downstream valley by applying computer programs to determine consequences/risk of dam failure and mitigate it.

In analyzing dam breach the essential breach parameters involved in reservoir routing and river routing techniques were estimated manually outside the software. Breach parameters involved in routing process includes time to failure, side slope of breach, bottom breach width, manning roughness coefficient, shape of breach and boundary condition. The unsteady hydraulics of the dam breach due to piping and overtopping failure mode was modeled using U.S. Army Corps of Engineers HEC-RAS 4.1 software. The model results show a peak flow of 10,938.43 m^3/sec and 8,700.57 m^3/sec at the dam for both overtopping and piping failure, respectively. The failures of dam occur through piping as well as overtopping. The overtopped depth is 57 cm.

To route the downstream valley both hydrologic and hydraulic routing were undertaken. Hydrologic routing employs the continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the end where as hydraulic routing employs the continuity equation and both energy and momentum balances to calculate open channel flow profiles. Floodplain mapping for the downstream of Gibe Dam was performed using the water surface elevations on the XS cut lines, within the limits of the bounding polygon. Global and local sensitivity analysis was performed at downstream valley and dam site, respectively. Global sensitivity analysis was performed using manning roughness coefficient and channel bed slope. whereas the local sensitivity analysis was performed using time of the dam breach, side slope of beach, the bottom breach width and relative effect one on other. The model result show that breach formation time is highly sensitive than side slope of breach and bottom *breach width. Similarly the manning roughness coefficient is more sensitive than channel bed slope for downstream reaches.*

Overtopping and piping failure reduction measure had been conducted to reduce the risk occur due to the failure of dam. Covering crest and d/s face of dam, u/s watershed management, vegetation and provision of stone pitching, increasing spillway capacity and crest width and free board increasing were the measures undertaken to reduce risk of overtopping dam failure. Piping dam failure mode reduction were rock toe, chimney drain, u/s face impervious blanket, impervious cutoff, relief walls drain trenches and grouting affected zone. Generally ,the failure of Gibe dam affect the government, community living around the dam ,environment through loss of life, economy loss ,damaging property and inundating the flood prone area.

Generally, the process of dam breach analysis and remedial measures for Gilgel Gibe dam as study case are discussed in this paper.

Key Words: - Dam Breach, Remedial Measure, Dam Breach Parameters, Hec-Ras, Inundation Map

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LIST OF ACRONMYS

MW	Mega Watt
USBR	United State Bureau of Reclamation
FEMA	Federal Emergency Management Agency
DEM	Digital Elevation Models
EAP	Emergency Action Plan
FERC	Federal Energy Regulatory Commission
FEMA	Federal Emergency Management Agency
HEC RAS	Hydrologic Engineering Center River Analysis System
NID	National Inventory of Dams
PMF	Probable Maximum Flood
TIN	Triangulated Irregular Network
USACE	United State Army Corps of Engineers
GIS	Geographic information system
IDF	Inflow Design Flood
DSIG	Dam Safety Interest Group
DSS-WISE	Numerical Simulation-Based Decision Support System for Water Infrastructural Security
USDOI	United States Department of the Interior
HEC-HMS	Hydrologic engineering center Hydrologic Modeling System
SDF	Spillway Design Flood
ITCZ	Intertropical Convergence Zone

MoWIE	Ministry of Water, Irrigation and Electricity
EEPCO	Ethiopian Electric Power Corporation
HEC-GeoRAS	Hydrologic Engineering Center Georeferenced River Analysis System
1-D	One Dimensional
RCC	Roller Concrete Compacted
ICODS	Interagency Committee on Dam Safety
NDSP	National Dam Safety Program
NDSRB	National Dam Safety Review Board
NRCS	Natural Resources Conservation Service
MWL	Maximum Water level
Km	Kilometer
M	Meter
Hr	Hour
NPH	No Public Hazard
RS	River Cross section

1 INTRODUCTION

1.1 Background

Dams have been playing a vital role in the development of any country by storing water at the back for different purpose such as for water supply ,irrigation, power generation, flood protection, navigation and recreation etc, but floods resulting from failures of dam have produced catastrophic disasters on the downstream as well as dam itself. When dams fail, property damage is common, but loss of life can vary dramatically with the extent of the inundation area, the size of the population at risk, and the amount of warning time available (Bureau of Reclamation, 1998). Hydrologic risk is the probability of failure occurring on any hydraulic structure attributable to extremely low or high water flux. Structural failure and performance failure are the two dam failure group. Embankment dam Failures which are caused by dam break/breach is Structural failure where as the failure of dam which is caused by a flood (water excess) or drought (water shortage) is categorized as performance failure. Dams are always subjected to probability of failure in achieving the intended objectives during their life span. One of the main causes for dam failure is flooding (e.g., overtopping attributable to inadequate spillway design), which can be considered a performance failure (Baker et al. 1988).

In order to reduce the potential damages owing to dam breaches, several hydraulic modeling programs have been developed that can simulate downstream water levels in response to a dam breach. When dam‘s failure large amount of water suddenly released and produces large flood waves. Depending on the quantity of water and the higher pressure, these flood waves have enough capacity to damage nearest inhabitants as well as the infrastructure in the downstream side (Singh, 1996). Some extreme flood situations may destroy the strong structure like power lines, industrial plants and causes several environmental impacts.

In dam breach analysis there are two main tasks. These are reservoir outflow hydrograph prediction and the routing of that hydrograph through the downstream valley to determine dam failure consequences. Prediction of the reservoir outflow hydrograph is especially important when the population at risk is located close to the dam, where peak attenuation and other flood routing effects have not yet taken place. This determined flood during failure of dam is routed through the downstream valley to analyze the peak discharge, time to arrival, sensitive parameters at the location of concern to identify the consequence.

In order to mitigate these risks, dam owners and regulators carefully analyze and inspect dams to identify potential failure modes and protect them from the failure. There is no program for preventing failure and because the potential for loading exceeding design limits can never be eliminated. Another important part of risk mitigation is analyzing potential failures of the dam and planning mitigation measure before the dam breach happens. Mitigation measure plans can include public education programs, development of warning systems and procedures, and development of effective evacuation procedures. Therefore understanding of the dam breach analysis and remedial measure are necessary for effective reservoir impounding, basin management and service life of the dam.

This paper discusses about dam breach analysis and remedial measure of Gilgel Gibe rockfill dam as case study. Gilgel Gibe rock fill dam is located about 260 km from Addis Ababa and the dam site is located 70 km from Jimma in province of keffa (MoWIE, 2004). The failure of this dam is particular concern because the failure of a large dam has the potential to cause more death and destruction than the failure of any other man-made structure. This is because of the destructive power of the flood wave that would be released by the sudden collapse of a large dam. Due to this, Modeling Gilgel Gibe dam breach and analyzing the effect of breach outflow hydrograph on downstream floodplain is necessary.

1.2 Statement of Problem

Designing, operating and constructing dam for desired purpose is the duty of designer, operator and contractor with its specification. Even though the dam is constructed as its specification with appropriate design drawings for purpose storing water (i.e. water supply, irrigation, power generation and flood protection). If dams are not properly designed, constructed and operated their risk of failures are higher which leads release of impounded water behind a dam. This impounds water which is released from the dam failure cause catastrophic disaster on community living over downstream valley. As it is analyzed, dam failures are complex with many historical dam failures not completely understood. 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).To reduce flood risk over downstream areas it needs properly designed, operation and construction of dam. This impounded water causes risk to downstream areas and affect the population and damages the economic resources of country. There are several potential causes of dam failure including hydrologic, hydraulic, geologic, seismic, mechanical, and operational (FEMA, 2013).

Significant hydrologic and non-hydrologic events are varies cause of dam failure. When dam fails, Water stored behind a dam can create a hazard to life and property located downstream of the dam. At all times the risks associated with the storage of water must be minimized. Nowadays, there are many embankment dams which are under construction and planned to be constructed in Ethiopia. Among constructed dams, Fentale, Timila and Qoga etc (for purpose of irrigation), Maqa dam (water supply) and Gibe, Tekeze, Beles etc (hydro power) are well known. In order for these dams to safely fulfill its intended purposes, the dam must be constructed, operated and maintained properly. Therefore, proper design, construction and operation of these dams are essential in reducing dam failure and risk analysis. With this context, Dam failure analysis and modeling study must be undertaken in addition to proper design and construction method for the sake of dam safety. Gilgel Gibe Dam is a rock-fill embankment dam consists of impervious clay core. This dam is proposed to impound the water within the reservoir to generate 184 MW power. For Gilgel Gibe 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 overtopping and piping failure.

Therefore, Gilgel Gibe Dam Breach analysis shows the downstream community to be risked due to the breach of dam and also tells the stakeholder to take remedial measure. This remedial measure use the community and government for further plan to mitigation area under pressure, responds for failures and identify consequence. In order to solve this problem , the study address the problem concerned with the proper modeling of Gilgel Gibe Dam breach and remedial measure which is important to all community living around the dam and downstream valley. In addition to this, the government and stakeholder are the concerned bodies to take appropriate measure and awareness about the scenario.

1.3 Objective of the Study

1.3.1 General Objective

The main objective of this thesis work is to analyze Gilgel Gibe rock fill embankment Dam Breach and analyze its remedial measures to reduce the risk level occurring on the downstream.

1.3.2 Specific Objectives

The specific objectives are:

- To estimate breach parameters through four empirical methods and select the appropriate empirical equation used
- To simulate Gilgel Gibe Dam failure under different water level condition
- To predict the maximum breach outflow on dam for overtopping and piping failures
- To route the outflow through the downstream reaches.
- To address the potential consequences of dam failure on downstream community
- To show downstream flood inundation map of Gibe
- To estimate sensitivity analysis of breach parameters
- To prepare the emergence action plan
- To address mitigation measure either prevent or reduce risk on the d/s area.

1.4 Significance of the Study

The study envision provide relevant information about the dam breach analysis and remedial measure of Gilgel Gibe dam for government, society, experts and stakeholders. This information helps these bodies for taking appropriate decision making and under take effective emergence action plan to minimize failure of dam and increasing the power production for the return period of the dam.

Thus studying about dam breach analysis and remedial measure of Gilgel Gibe has significant due to the following reasons:

- If dam failures suddenly, the possible consequences result include loss of human life, economic, social, environmental impacts, damage to national security installations , political and legal ramifications.
- The flood that comes after failure not only affects the downstream community and infrastructure also affects the economy of country as well as the social interaction in Gilgel Gibe valley.
- Failures of dam not only affect downstream community, economic, social and environmental impacts it also affects the natural resources and fauna and flora which live in reservoir.

Similarly the study outcome will be duplicated with minor change to other hydroelectric power, water supply and irrigation dam facing identical problems.

1.5 Purpose and Scope of Study

The flood propagation, risk assessments and flood mitigation models for already constructed dams is becoming a necessity for a variety of reasons such as decreasing life loss, environmental destruction and economic damage. The breach of dam is a complex process that depends on various factors such as hydraulic, hydrologic, geological, geotechnical. This thesis mainly focuses on generating breach hydrographs and routing of that hydrograph through the downstream valley. It proposes approach that copes with hazards caused by structural failure events by decreasing their consequences. If dam breach occurs, catastrophic scenarios and enormous socio–economic impact will happen. This addresses the problem of dam breach analysis as well as simulation. Due to this mitigation of the flood water caused by failure of structures are important in reducing the risk. In this thesis, the framework and techniques for modeling dam failure and propose several modeling tools for dam breach modeling has been worked out. This technique offers advantages of simultaneous evaluation of different flood mitigation scenarios.

Two conditions were analyzed: (1) a sunny day piping dam failure event, and (2) a dam failure coincident with the PMP flood. These dam failure scenarios will be used as the basis for the development of emergency action plans and are intended to serve as support for the dam security and emergency action plan. Generally the purpose of this study is as follows:

- To analysis the Gibe dam breach in different hydraulic condition using HEC RAS and justify the result.

- To estimate the dam breach out flow hydrograph of downstream area which happen during the failures both by the overtopping and piping mode of failures.
- To estimate sensitivity analysis of breach parameters
- To estimate downstream inundation extent and show risked area.

The purpose of this paper is the issue of dam break analysis and remedial measure for Gigel Gibe Dam using an unsteady U.S. Army Corps of Engineers HEC-RAS (USACE, 2010) model for the dam and the downstream river to the Omo Gibe. Scope of the thesis work will cover the upstream reservoirs, dam which happen in entire catchment area and downstream valley of Gilgel Gibe till it joins the Omo Gibe River. The study will cover 64 km till the river joins Great Gibe River.

1.6 Thesis Outline

This thesis contains six chapters organized as follows.

Chapter One gives a general introduction about dam breach analysis and its remedial measure with its statement of problem, objective, significance, purpose, scope and limitation of the study.

Chapter Two describes the Reviewed literature related to the study.

Chapter Three deals with the Description of Study Area and Methodologies.

Chapter Four describes about the Remedial Measure.

Chapter Five describes the Results and Discussion.

Chapter Six describes the Conclusion and Recommendation of the entire study.

2 LITERATURE REVIEW

2.1 Introduction

Floods resulting from failures of constructed dams have produced some of the most devastating disasters of the last two centuries. In order to reduce the potential damages owing to dam breach due to flooding, several hydraulic modeling programs have been developed through different researchers. The actual failure mechanics of dam failure have not been well understood for either earthen or concrete dams. In earlier attempts to predict downstream flooding due to dam failures, it was usually assumed that the dam failed completely and instantaneously (Abinet, 2010). Even though, the actual failure mechanics are not yet understood the failures of dams cause an economic loss that transcends immediate property damage and loss of life.

A dam break is the partial or catastrophic failure of a dam which leads to an uncontrolled release of water (Fread, 1993). The potential catastrophic failure and the resultant downstream flood damage is a scenario that is of great concern. The mitigation of the impacts to the greatest possible degree requires modeling of the flood with sufficient detail so as to capture both the spatial and temporal evolutions of the flood event (FEMA, 2004), as well of the velocity field. The selection of an appropriate model to correctly simulate dam break flood routing is therefore an essential step. Traditionally, one and two-dimensional models have been used to model dam break flooding, but these models are limited in their ability to capture the flood spatial extent, in terms of flow depth and velocity and timing of flood arrival and recession, with any degree of detail. The development in the last years has led to several numerical models aimed at solving the so-called dam break problem (USACE, 2010).

Hydraulic models such as HEC-RAS are often used for the analysis of downstream impacts resulting from potential dam failures. One-dimensional dam breach hydraulic model of HEC-RAS is used frequently to predict the flood inundation area due to a dam breach flood through the downstream valley. It was found that HEC-RAS performed well, with relatively good agreement between predicted and measured water levels (Yochum et. al, 2008) & (Gee, 2010). Regression equation used for estimation of the dam breach parameters has been developed by researcher from historically fail dam.

Under this section different literatures works including recent scientific journals, embankment dam breach modeling books, breach parameter estimation methods, different computer software which are related to the study has been reviewed widely.

2.2 History of Dam Breach

A dam is a barrier made of earth material, rock, or concrete or a combination thereof that is constructed across a river for impounding or diverting the flow of water. The history of dam construction dates back to 2900 B.C. with the oldest dam in the world believed to be constructed in Wadi el- Garawi, 30 km south of Cairo, Egypt (Singh, 1996). Among dams constructed in the world highest dam for 300 years was Alicante Dam in Spain. Nowadays, The Nurek Dam of Tajikistan is the highest where as Three Gorges Dam of China is the largest dam in the world. Low probability of failure has been considered when dams are designed during their design, operation and construction. Despite this, dams do frequently fail and failure of dams can be partial or complete. Dam failure because release of huge amount of water to downstream areas this cause greater chance of loss of life or damage to valuable property. According to Cost (1985), major causes of failures are overtopping due to inadequate spillway capacity is 34 %, foundation defects is 30 %, and piping and seepage is 28 %. In history of dam failure the Vaiont arch dam in Italy in 1963 was overtopped with wave estimated to be 100m high. The failure of this dam caused the loss of 2,600 human lives over downstream area. The failure of Teton Dam in Idaho through piping cause flooding of Sugar City and cities in 1976. The failure of Teton dam killed 14 people and caused over $1 billion in property damage.

In context of Africa, there are many dam constructed for the purpose of hydro power, water supply and irrigation etc. Among dam constructed in Africa, old aswan dam (Egypt), Merowe (Sudan), Megech (Ethiopia), Nyabarongo (Rwanada), Bujagali (Uganada) and Semliki (Democratic Republic of Congo) are few of them. These dams are constructed for the purpose of food security, reducing the lack of electricity and scarcity of potable water. In Ethiopia, traditional small scale irrigation schemes have existed centuries ago, particularly in the eastern, central, north western parts of the country (Kamal Eldin Bashar M. K., 2005) for the irrigation and water supply purposes. Modern small scale irrigation using micro-dams were given great emphasis after major droughts have stricken the country late 1970‘s and 1980‘s during Megistu H/Mariam regime.

According to Ministry of Water Resources, micro and medium dams constructed in 1970‘s and 1990‘s in Amhara and Tigray regions (MWR, 2003). The common problems for dam failure were overtopping and piping due to inadequate spillway capacity flood estimation problem; seepage through foundation, abutments, and reservoir area site selection problem; cracking or structural failure- geotechnical problem; sedimentation design problem and lack of watershed management; less inflow in the reservoir-hydrological analysis problem; and lack of proper maintenance and rehabilitation work (Kamal Eldin Bashar M. K., 2005).

Many embankment dams are constructed in Ethiopia for the purpose of irrigation, water supply and hydropower generation. However, their capacity reduces frequently before their design life time due to a number of reasons. The main causes of capacity reduction are Hydrological, Structural and Hydraulic failure of which hydraulic failures contributes 58% in Amhara region (Mekonnen, 2008). Even though the dam breach occurrence in current time is low, the consequence the dam breaches in case it happens may be catastrophic and has multi-dimensional damages on the downstream areas (Andrew Charles, 2011). In this thesis investigation of embankment dams‘ failure, in particular of Gilgel Gibe embankment dam has been undertaken.

2.3 Major Steps in Dam Breach Analysis

2.3.1 Information Gathering

At this Stage, information about the reservoir, components of dam and downstream valley are reviewed. The following data are required to complete the study (Hwang, 2000):

- Hydrologic information such as precipitation patterns, precipitation characteristics, watershed characteristics, and topographic data.
- Reservoir characteristics such as storage capacity, surface area, normal and maximum pool elevations, inflows, etc.
- Physical characteristics of the dam and spillway such as construction type, height, crest length, crest elevation, toe elevation, spillway type, spillway width, elevation and capacity, etc.
- Type, size and characteristics of downstream hydraulic structures such as dams, bridges, culverts, railroads or highway crossings, etc.
- The location and type of downstream development such as schools, highways, railroads, parks, etc.

2.3.2 Size and Hazard Classification of Dam

Depending on the size and hazard potential, dams can be classified into three categories: small, intermediate, and large. The height of the dam and the volume of the reservoir determine the size category of a certain dam.

Table 2-1 : Classification of dam based on height and reservoir volume (Hwang, 2000).

Size of dam	**Dam height (feet)**	**Volume of the reservoir (acre-feet)**
Small	Less than or equal 40	Less than or equal 1000
Intermediate	Less than or equal to 100	Less than or equal 50,000
Large	100 plus	50,000 plus

Dams can also be classified according to their hazard potential. These are high, significant, low and No human hazard dam. This depends on the number of human lives that are threatened, and the economic loss inflicted on the downstream areas in the event of dam failure (FEMA, 2013).

Table 2-2: Recommended IDF requirements for dams using prescriptive approach (FEMA, 2012)

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

PMF = Probable maximum flood

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

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

2.3.3 Inflow Design Flood Hydrologic Analysis

The main objective of this part is to estimate the inflow design flood (IDF) to reservoir and to determine the most probable type of failure. At this step, different modes of dam failure will be assumed based up on the amount of IDF. When inflow hydrograph exceeds the crest of the dam the overtopping failure occur where as if internal erosion occurs inside the dam body the ―sunny day" failure mode shall be assumed failure. The design floods need to be verified and used as IDF hydrographs and routed through the reservoir using HEC RAS to determine the dam Breach hydrographs.

Table 2-3: Recommended inflow design floods (Hwang, 2000).

Hazard	*Size*	*Inflow Design Flood (IDF)*
Low	Small	50 - 100 year
	Intermediate	100 - 0.5 PMF
	Large	0.5 PMF - PMF
Significant	Small	100 - 0.5 PMF
	Intermediate	0.5 PMF - PMF
	Large	PMF
High	Small	0.5 PMF - PMF
	Intermediate	PMF
	Large	PMF
Where PMF is Probable Maximum Flood		

2.3.4 Breach Mechanisms of Embankment Dam

According to the study undertaken by MacDonald & Langridge-Monopolis(1984) there are two breach formation mechanisms: these are includes breaches formed by the sudden removal of all or a portion of the impounding structure and breaches formed by erosion of embankment material (which addresses overtopping and internal erosion failures of embankments). Although breaching in embankment dams may occur for a variety of reasons, breaches in embankment dams are most often modeled as overtopping or piping failures.

2.3.4.1 Overtopping Failures

The overtopping failure is occur when the reservoir's water surface elevations exceeds the top of the dam due to significantly large amount of IDF inflows into the reservoir. Overtopping is believed to have been responsible for about half of worldwide embankment dam failures and most of the deaths (ICOLD, 1997). Researcher like Foster et al (2000) evaluated and support the investigation statically, that 46 % of embankment dam failures are attributable to overtopping. The main cause of dam failures is the inability to safely pass flood flows.

The overtopping failure is occurred when ; Insufficient capacity of spillway design , Partly or fully blocked spillway , Losses of storage capacity of the dam and Huge water displacement due to earthquake (E.Costa and L.Schuster ,1988).In case of excess rainfall, the upstream water level increases instantly. When this level exceeds the maximum drainage capacity of the dam, water started to flow over embankment. This over flowing water causes the breaching followed by slide at downstream slope of the embankment as a consequence of external erosion (Kjaernsli et al. 1992).

Overtopping failures of earthen dams typically begin with head cutting at the downstream toe and advance upstream until the erosion reaches the dam crest and reservoir surface. According to a study by Ralston (1987), a small head cut typically formed due to external erosion forms on the downstream face of a cohesive soil embankment and progresses upstream as shown in Figure 2.1. According to Powledge et al. (1989) Erosion on the downstream face of a cohesive soil embankment dam shown as follows.

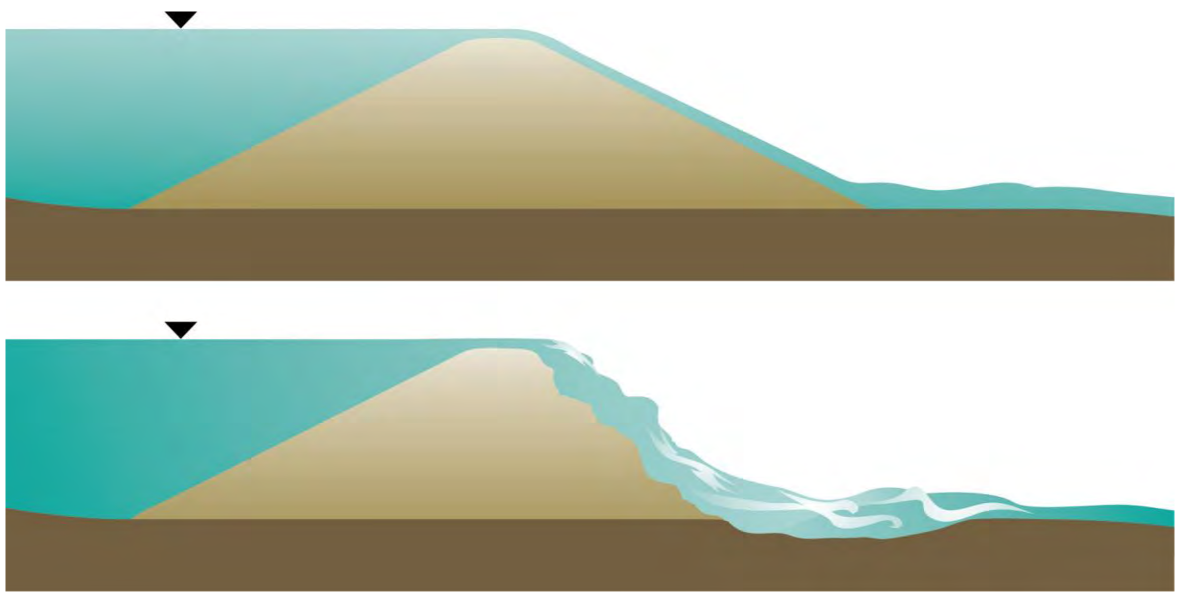

Figure 2-1: Erosion on the downstream face of a cohesive soil embankment dam (Powledge et al. 1989)

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. The breach should be modeled as initiating at the maximum section typically located at the centerline of the downstream main channel. A generalized trapezoidal breach progression is illustrated in Figure 2.2.

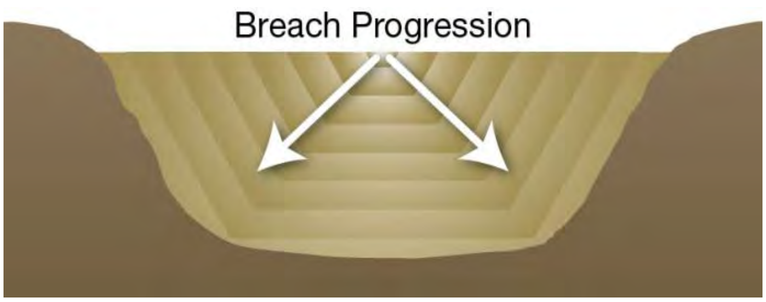

Figure 2-2: Overtopping trapezoidal breach progression (Gee, 2009)

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:

- **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.

Both the linear and sine wave progressions are appropriate in this situation. Like with overtopping failures, the breach progression defines the rate at which the piping hole or breach width enlarges with time. HEC-RAS models allow the user to choose a linear progression, sine wave progression (starts out slowly, with a more rapid increase in the center of the progression and then ending more slowly), or a user defined (manual) progression.

According to the State of Colorado Department of Natural Resources (2010), both progressions should be evaluated and the progression with the more conservative results should be utilized. It is hypothesized that HEC-RAS‘s ability to account for the tail water effects through its hydraulic calculations may contribute to the lower peak discharges calculated when the linear progression is used. Using the linear progression in HEC-RAS may be similar to accounting for the tail water effects twice which results in a less conservative peak discharge. Because of this, the linear progression is recommended for the analysis use in HEC-RAS models using parameters based upon empirical methods.

2.3.4.2 Piping and Internal Erosion Failures

Piping failure also referred to as a ―sunny day” failure because it occurs independent of rain events, even on sunny days, is initiated by erosion of material due to piping, earthquakes, slope instabilities, foundation weaknesses, or other structural weaknesses. Piping is a result of soil erosion which takes place through the embankment because of the seepage water flow (Fell et al. 2003). The water flow exerts force on particles and washes out them through an unexpected seepage discharge point. This discharge point undergoes further erosion towards upstream side and form an open like ―pipe” through the embankment. If left unchecked, both internal erosion and piping will progress until the flow path is large enough to empty the reservoir, sometimes in a breaching type event.

According to State of Colorado Department of Natural Resources, Division of Water Resources (2010) Figure 2-3 shows a schematic of a fully formed piping hole. 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. For fair weather breach analysis, an initiation time should be specified regardless of pool elevation (Gee, 2010).

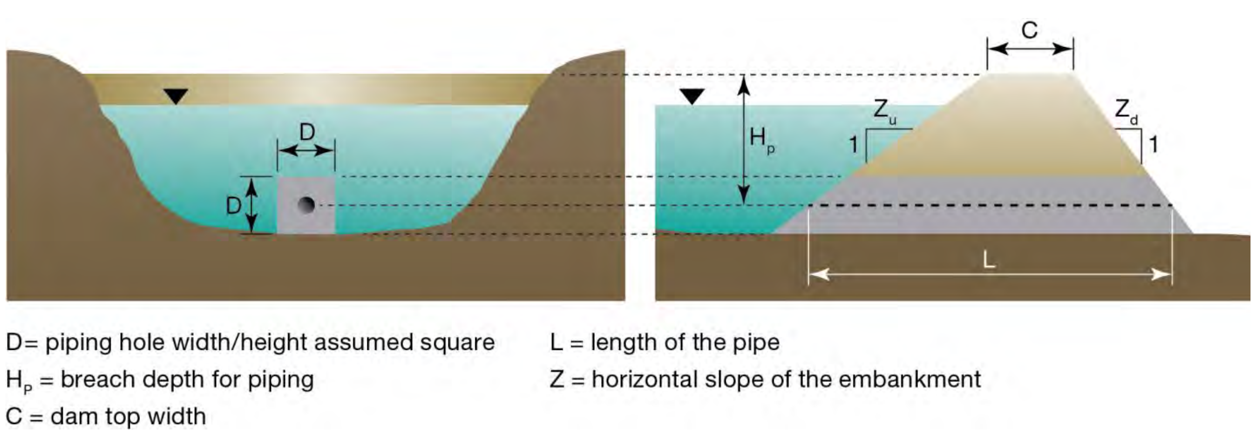

Figure 2-3: Schematic of piping hole (Gee, 2010)

2.3.5 Dam Breach Parameters and Analysis

The objective of this step is to calculate and verify the outflow breach hydrograph resulting from a dam failure. The most typical breach characteristics are the shape, breach depth, breach width, side slopes and breaching time (failure time) at which the breach develops. Table 2-4 shows the parameters in the development of breach for hypothetical failure of dam. The following information's in the table are commonly accepted for use in evaluating and selecting dam breach parameters**.**

Table 2-4: Dam-break breaching parameters (FERC, 1993; Hwang, 2000)

Breaching Parameter	Values	Type of Dam
Average Width of Breach (BB)	0.5 HD <BB <3 HD	Earthen, Rock fill
	BB = 0.8 * Crest Length	Slag, Refuse
	BB = Multiple of Monolith Widths	Masonry, Gravity
	BB = Crest Length	Concrete, Arch
Saide Slope of the Breach (Z)	0 < Z < 2	All
	Z= 0	Masonry, Gravity
	0.25 <Z < 2	Earthen, Rock fill
	1 < Z < 2	Slag, Refuse
Time of Failure in Hours (TFH)	0.1 < TFH <3.0	All
	0.1 < TFH < 0.3	Masonry, Gravity, Slag, Refuse
	0.1 < TFH < 0.5	Earthen, Non Engineered, Poor Construction
	0.3 < TFH < 3.0	Earth, Engineered, Conmpacted

Note: TFH – time to dam failure in hour

BB - Average breach bottom width (ft)

HD – Dam height (ft) and Z or SS – Side slope of the dam breach

In most case analyzing dam breach is not as such easy to determine the shape of breach but shape of the breach is usually approximated as geometric shape such as rectangle, triangle, trapezoidal or parabola. Johnson and Illes (1976), after analyzing the data from approximately 100 case studies concluded that the breach develops initially in ‗V' shape, three to four times wider than deep, later developing in the lateral direction, once the apex reaches the hardest material of the dam core or its foundation.

The breach shape geometric is mostly represented as simple trapezoidal shape, as shown in figure below. In this simple trapezoidal breach shape essential parameters are the bottom breach width(B), average breach width(B_{avg}), breach height(H_b) ,breach side slope ratio Z:1 (H:V) and the depth of water above the eventual breach bottom at the time of failure, H_w. In most case the breach height (H_b) is equal to the height of water (H_w) for overtopping failure mode. The following figure shows the trapezoidal shape of breach. Therefore, Trapezoidal breach shape is considered in this study because the case study is rock fill embankment dam.

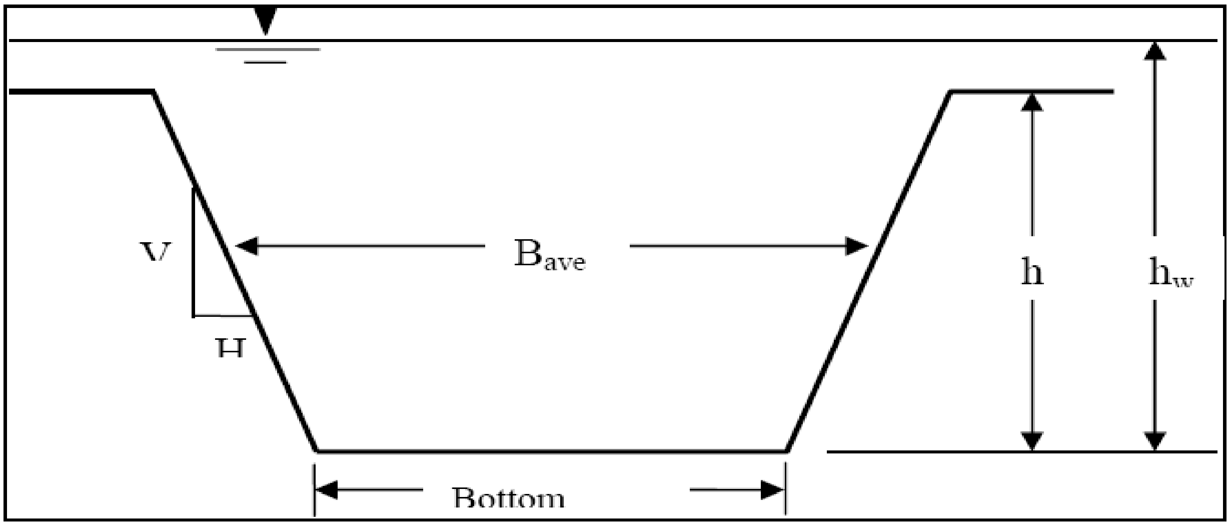

Figure 2-4: Idealized dam breach geometry.

A. **Breach Depth** (Breach depth or h_b): Also referred to as breach height in many publications. This is the vertical extent of the breach, measured from the dam crest down to the invert of the breach.
B. **Breach Width:** The breach width is the average of the final breach width, typically measured at the vertical center of the breach. The ultimate breach width and the rate of breach width expansion can dramatically affect the peak flow rate and resulting inundation levels downstream from the dam.
C. **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). Accurately predicting the breach side slope angles is generally of secondary importance to predicting the breach width and depth.

The dam breach process can and should be divided into two phases. These are breach initiation and breach formation, and it is important to make a distinction between the times associated with each phase (FEMA, 2013& Wahl, 1998). These two phases can be discussed as follows:

Breach initiation: The breach initiation phase begins with the first flow of water over or through a dam that is sufficiently large to initiate warning, evacuation, or heightened awareness of the potential for dam failure. The breach initiation phase ends when the active erosion front reaches the upstream face of the dam, thereby producing a rapidly accelerating breach outflow and unstoppable failure of the dam.

Breach formation: Breach formation 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. The breach formation phase continues until the breach has enlarged to its approximate maximum dimensions.

D. Rate of Failure

The rate of breach expansion, increase in depth and width, is called rate of failure. This rate can progress at a linear or non-linear rate. Dam breach rate of failures defined by breach parameters such as the size, shape, and timing of the breach that modify by inline structure, but are coded into the HEC-RAS model as plan data. The parameters needed for the HEC-RAS dam breach model are breach shape, breach width, time to failure, pool elevation at time of failure, breach side slope and peak discharge. The estimation of these parameters allows the accurate out flow hydrograph and downstream inundation mapping of the study area. The following are points to be reviewed.

MacDonald & Langridge -Monopolis (1984)*:* MacDonald and Langridge-Monopolis (1984) proposed a breach formation factor, defined as the product of the volume of water coming out of dam (including initial storage and concurrent inflow) and the depth of water above the breach invert at the time of failure. Further, they concluded from analysis of the 42 case studies cited in their paper that the breach side slopes could be assumed to be 1H:2V or 0.5H :1V in most cases; the breach shape was triangular or trapezoidal, depending on whether the breach reached the base of the dam.

The data that Macdonald and Langridge –Monopolis used for their regression analysis had the following ranges Height of the dam 4.27 – 92.96 m (with 76% < 30 m, & 57 % < 15 m) and Breach out flow volume 0.00037 – 660x10^6 m^3(with 79% < 25x$10^6$$m^3$,& 69% < 15x$10^6$ m^3). According to the Wahl (1998) report the following is the Macdonald and Langridge –Monopolis equation for volume of material eroded and breach formation time. For earth fill with clay core or rockfill dams:

$$V_{eroded} = 0.00348(V_{out} * H_w)^{0.852} \quad \text{...Eq. (2-1)}$$

For earth fill dams

$V_{eroded} = 0.0261(V_{out} * H_W)^{0.769}$ ………………………………………………… Eq. (2-2)

$T_f = 0.0179(V_{eroded})^{0.364}$ …………………………………………………………. Eq. (2-3)

Where: V_{eroded} = volume of material eroded from dam embankment (cubic meters) ; V_{out} = volume of water that passes through the breach (cubic meters) ; H_w = depth of water above the bottom of the breach (meters) ; T_f = breach formation time (hours)

The base width of the breach can be computed from the dam geometry with the following equation (state of Washington, 1992).

$W_b = [V_{eroded} - H_b^2 (CZ_b + H_bZ_bZ_3/3)] / [H_b(C + H_bZ_3/2)]$. ………………………. Eq. (2-4)

Where: W_b = bottom width of the breach (meters); h_b = height from the top to the dam bottom of the breach(meters); C = the crest width of the dam (meters); $Z_3 = Z_1 + Z_2$; Z_1 = Average slope (Z_1:1) of the u/s face of dam ; Z_2 = Average slope (Z_2:1) of the d/s face of dam ; Z_b = side slope of the breach (Z_b: 1), 0.5 for the Macdonald method case

Generally, Wahl study states Macdonald method will over predict times in some cases while many equation will under predict.

Von Thun and Gillette (1990): They proposed that breach side slopes be assumed to be 1H: 1V except for dams with cohesive shells or very wide cohesive cores, where slopes of 1H: 2V or 1H: 3V may be more appropriate. The data that von Thun & Gillete for regression analysis includes height of dams to be in the range of 3.66 to 92.96 m(with 89% < 30m ,& 75% < 15 m) and also volume of water at breach time is 0.027 to $660x10^6$ m^3(with 89% < $25x10^6m^3$, & 84% < $15x10^6m^3$).

Von Thun and Gillette proposed the following relationship for average breach width:

$B_{ave} = 2.5h_w + C_b$ ………………………………………………………… Eq. (2-5)

Where; B_{ave} = average breach width (meters); Hw= being the depth of water at the dam at the time of failure; C_b = coefficient, which is a function of reservoir storage size.

Table 2-5: Coefficients as a Function of Reservoir size

Reservoir Size, m3	C_b, meters	Reservoir Size, acre-feet	C_b, feet
$< 1.23*10^6$	6.1	< 1,000	20
$1.23*10^6$- $6.17*10^6$	18.3	1,000-5,000	60
$6.17*10^6$- $1.23*10^7$	42.7	5,000-10,000	140
$> 1.23*10^7$	54.9	>10,000	180

Von Thun and Gillette proposed two methods for estimating breach formation time. The first two equation show breach formation time as function of depth of water above the breach bottom.

$$\mathrm{Tf} = 0.020\mathrm{hw} + 0.25 \quad \text{……….……………………....Eq. (2-6)} \quad \text{[Erosion Resistant]}$$

$$\mathrm{Tf} = 0.015\mathrm{hw} \text{…………………………………………Eq. (2-7) [Easily Erodible]}$$

Where t_f is in hours and h_w is in meters.

Von Thun and Gillette also developed equations for breach formation time based on observations of average lateral erosion rates (the ratio of final breach width to breach formation time) versus depth of water above the breach invert. They found a stronger correlation between the lateral erosion rate and depth than for the total breach formation time versus depth.

Using lateral erosion rate data, Von Thun and Gillette put forth two additional equations:

$$\mathrm{Tf} = \mathrm{Vw}/4\mathrm{hw} \text{…………………………………………Eq. (2-8) (Erosion resistant)}$$

$$\mathrm{Tf} = \mathrm{Bav}/(4\mathrm{hw} + 61) \text{ ………………………………Eq. (2-9) (Easily erodible)}$$

Where h_w and B_{ave} both given in meters. Each of these equations requires an assumption or prediction of the average breach width. Since the study case is rockfill dam the equation used for the determination of the breach formation times are Eq. (2-6) and (2-8).

Generally, Von Thun &Gillette suggest the breach formation time is presented for both erosion resistant and easily erodible dams which is stated as the upper and lower boundary corresponding to well constructed dams (erosion resistant) and poorly constructed dams (easily erodible).beside that the breach formation time calculated above depends on depth of water above the breach bottom and reservoirs size.

Froehlich (1995): According to the research under taken by Dr.Froehlich 63 earthen, zone earthen with a core wall (i.e. clay) and rock fill data sets to develop a set of equation to predict the breach parameters (i.e. breach width, side slope, failure time). Dr.Froehlich (1995a) used the data for regression equation analysis includes the height of dams to be in the range of 3.66 to 92.26 m and also volume of water at breach time is 0.00130 to 660x10^6 m^3 (with 87% < 25x$10^6$$m^3$, & 76 %<15x$10^6$$m^3$).

Having all these assumptions and analysis Dr.Froehlich (1995) states Regression equation for average breach width and failures time as follows.

$B_{ave} = 0.1803\ K_o V_w^{0.32}\ h_b^{0.19}$ ……………………………………………………Eq. (2-10)

$T_f = 0.0025 V_w^{0.53}\ h_b^{-0.90}$ ……………………………………………………....Eq. (2-11)

Where: B_{ave} = average breach width (meters); K_{o} = Constant (1.4 for overtopping & 1.0 piping or seepage failure); h_b = height of the final breach (meters); C = the crest width of the dam (meters); V_w = Reservoirs volume at time of failure (cubic meters); t_f = breach formation time(hours);

According to research conducted by Dr.Froehlich the average side slope should be 1.4H: 1V for overtopping failure and 0.9H: 1V for piping or seepage failure mode. Even though Dr. Froehlich don't clearly state the height of breach ,the height of breach is calculated by assuming the breach goes from the crest of dam to natural ground elevation at the breach location.

Froehlich (2008) : Dr.Froehlich (2008) updated his regression analysis after addition 11 new data study on the previous investigation utilized 74 earthen, zone earthen with a core wall(i.e. clay) and rockfill data sets to develop a set of equation to predict the breach parameters(i.e breach width, side slope, failure time). Dr.Froehlich (2008) used the data for regression equation analysis includes the height of dams to be in the range of 3.05 to 92.26 m (with 93% < 30 m, & 81% < 15m) and also volume of water at breach time is .0139 to 660x10^6 m^3 (with 86% < 25x$10^6$$m^3$, & 82% < 15x$10^6$$m^3$).

After all the analysis Dr.Froehlich (2008) comes up with regression equation for average breach width and failure time.

$B_{ave} = 0.27\ K_o V_w^{0.32}\ h_b^{0.04}$ ……………………………………………………Eq. (2-12)

$T_f = 63.2\ (V_w/gH_b^2)^{0.5}$ ………………..……...……………………………………...Eq.(2-13)

Where: Bave = average breach width (meters); K_o = Constant (1.3 for overtopping & 1.0 piping or seepage failure); h_b = height of the final breach (meters); V_w = Reservoirs volume at time of failure (cubic meters); g = gravitational acceleration (9.80665 meters per second squared); t_f = breach formation time(second);

The 2008 Froehlich equations have been changed slightly from the 1995 equations according to Wahl's study. Froehlich recommends breach side slopes of 0.7:1 (horizontal: vertical) for piping and 1.0:1 for overtopping.

E. Determining outflow hydrograph

There are a few recommended computer programs to calculate the breach outflow hydrograph, in addition to manual calculations. 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. Wahl (1998) evaluated these equations by comparing predicted peak discharges to actual peak discharges and found significant scatter between observed data and that predicted by the equations. The Froehlich equation had the best correlation, but still could significantly over-predict or under predict the peak flow. In general the peak flow equations should be used for comparison purposes (USACE, 2014). Shown below is a summary of some of the peak flow equations that have been developed from historic dam failures (USACE, 2014):

- Froehlich (1995b): $Q = 0.607Vw^{0.295}hw^{1.24}$
- Kirkpatrick (1977): $Q = 1.268\ (hw+0.3)^{1.24}$
- Costa (1985): $Q = 1.122\ (S)\ 0.57$ & $Q = 1.776\ (S\ hd)^{0.44}$
- Froehlich (2008): $Q = 3.1B_{avg}\ H_w^{1.5}\ (\frac{y}{y+Tf\sqrt{Hw}})^{3}$, where, $y = 23.4\ A_s/B_{avg}$

- Soil Conservation Service (SCS, 1981): $Q = 16.6\ hw^{1.85}$
- MacDonald and Langridge-Monopolis (1984): $Q = 3.85(Vw*hw)^{0.411}$
- USBR (1982): $Q = 19.1\ (hw)^{1.85}$

Where: Q = Peak breach outflow (m3/sec); h_w = Depth of water above the breach invert at time of breach (m); V_w = Volume of water above breach invert at time of failure (m^3); S = Reservoir storage for water surface elevation at breach time (m3);h_d = Height of dam (m)

The above empirical equations are used to calculate the peak out flow discharges that depends on the storage amount of reservoirs, height of dam and Volume of water above breach invert. Therefore, the maximum breach outflow discharge that will be obtained from the analysis should be checked for its reasonableness. As the Literatures recommended that one can check the reasonableness of the maximum breach outflow obtained by empirical method with that of the model.

F. Dam-Break Maximum Breaching Outflow Verification

After the analysis is completed and the hydrograph developed, it is necessary to check the reasonableness of the maximum breaching outflow Q_{max}. There are a few commonly known techniques to check Q_{max}: historical predictor equations, parametric models, physically based erosion methods, direct comparison techniques, customized prediction equations, classical equations and FERC recommended equations.

2.4 Causes of Dam Failures

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 as one or more monolith sections formed during dam fail, whereas concrete arch dams tend to fail suddenly and completely (Canadian Dam Association, 2007).

According to the study undertaken by different researcher, Embankment dams do not usually have a complete or sudden failure, 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. Causes of Dam failure are flood event , piping or seepage, land slide ,earthquake ,foundation failure ,equipment failure ,malfunction gates ,structural failure ,upstream dam failure ,rapid draw down of pool and sabotage. Dam failures accounts 35% failed due to overtopping, 38% due to piping & seepage, 21% from the defects in foundation & 6% from other failure modes (Cost, 1985). Hydrologic, geologic, structural, seismic and human-influenced are mechanisms that lead causes of dam failures through different mode of failures. The following Table 2-6 shows five causes of dam failure.

Table 2-6: Typical Dam Failure Modes (FEMA, 2004)

Failure Mode	Examples of dam failures
Hydrologic	Overtopping due to: • Inadequate spillway design • Blocked spillway • Loss of freeboard* due to embankment settlement or erosion • Structural overstressing of dam components
	Surface erosion due to: • High velocity water • Wave action
Geologic	Piping and internal erosion caused by: • Internal cracking, hydraulic fracture, or differential settlement • Inadequate filters • Outlet pipeline failure • Pipes through the embankment formed by roots or animal/insect burrows
	Slope instability and hydraulic fracturing: • Load exceeds sliding resistance at base or at joints of structure
Structural	Embankment dam: Failure of the upstream or downstream face
Seismic	Earthquakes/ground movement; also liquefiable foundations or embankment materials
Human influenced or caused	Misoperation: • Sudden rise in reservoir level causes flow through transverse cracks in embankment • Incidents including gate failures, power interruption etc.
	Terrorist activities: • Purposeful misoperation of the dam • Impact of object that removes part of the dam crest

2.4.1 Hydrologic Failure Modes

Many dam failures have resulted because of an inability to safely pass flood flows. Failures caused by hydrologic conditions can range from sudden, with complete breaching or collapse, to gradual, with progressive erosion and partial breaching. The most common modes of failure associated with hydrologic conditions include overtopping, erosion of earth spillways, and overstressing the dam or its structural components.

Hydrologic dam failures are induced by extreme precipitations which cause probable maximum flood or (PMF) 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.

2.5 Dam Breach Analysis Approach of the Study

Dam breach analysis in both overtopping and piping failures are based on hydrological and non hydrological failure approach, respectively. According to this study the scenario included overtopping failure and piping failure modes. There are two primary dam breach study approaches these are a consequence based approach and an event based approach. These are discussed as follows.

2.5.1 Consequence Based Approach

The Consequence based approach is sometimes called risk based approach. A risk-based approach is commonly used for dam design purposes to establish the SDF or IDF for a dam. For a risk-based approach, the downstream consequences for a range of hydrologic dam failure events are evaluated (FEMA, 2007). It sometimes occurs that two or more dams are constructed on a watercourse and the failure of an upstream dam may cause a sequential failure of a downstream dam. The consequences evaluation is not based on the probability of failure, but instead on the potential loss of life or increase in economic losses caused by a potential dam failure.

2.5.1.1 Inflow Design Flow and the Incremental Hazard

According to FEMA 94, IDF is defined as ―the flood flow above which the incremental increase in water surface elevation downstream due to failure of a dam or other water retaining structure is no longer considered to present an unacceptable additional downstream threat." Therefore, incremental hazard evaluation and the establishment of the IDF is a risk-based approach. The selection of the IDF is based on the evaluation of the magnitude of several flood events and 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.

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(improve) the incremental increase in unacceptable additional consequences a failure may have on areas downstream.

2.5.1.2 Population at Risk

Population at risk is defined as the number of people occupying the dam failure floodplain prior to the issuance of any warning. Probable loss of life is an important factor used in hazard potential classification systems and emergency action planning. 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 of the factors described as follows. These factors are: 1) The number of people occupying the dam failure flood plain, 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.

Generally dam breach is likely to do permanent damage to loss of life, loss of housing and commercial property, loss of dam and power generation station ,loss of fields and agricultural infrastructure and others, near the dam whereas only temporary damage is likely further downstream.

2.5.2 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. 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 (FEMA, 2013).

The greatest advantage to use 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. Typically, several hydrologic and non-hydrologic (fair weather) events are evaluated as part of an event-based dam safety analysis. For hydrologic failure events, an extreme flood event ranging from the 50-year event for low-hazard dams up to the PMF for high-hazard dams is selected based on the potential for loss of life due to a dam failure or for significant economic and environmental losses.

The degrees of hazard potential classification of the dam lead to the selection of 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).

2.5.2.1 Non-Hydrologic Failure

Non hydrologic events describes a situation when the dam break happens unexpectedly due to collapse of dam body, internal piping, stability loss (Dr.Greg & Darre, August, 2007).The non hydrologic failure is called as a fair weather (Sunny Day) or non-precipitation conditions breach. Sunny day breach is analyzed by establishing an initial reservoir water level and commencing a breach analysis without additional inflow from a storm event and also used to analyze piping failures for hydrologic, geologic, structural, seismic, and human-influenced failure modes (FEMA, 2013). Because of the small discharge and volume compared to that of a dam breach the Base flow conditions for a fair weather failure are typically ignored. According to FEMA the three most common initial water level elevations for fair weather breach analyses are as follows:

A. Normal Pool Elevation

At the beginning the reservoir water level is equalized as normal pool level. As a practical matter, the resultant dam break flood for the normal pool condition is relatively insensitive to the magnitude of reservoir inflow and outflow because the inflow/outflow are typically very small by comparison to the dam break flood. 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 sunny day conditions. This type of event is modeled as piping/internal erosion failure and a monolith collapse resulting from sliding, foundation instabilities, or a seismic event for embankment and concrete dams respectively.

B. Invert of Auxiliary Spillway

The Invert of Auxiliary Spillway is also called as lowest uncontrolled spillway. The reservoirs water level set as Invert of Auxiliary Spillway. Dam failure during a flood generally produces a larger dam break flood than a failure at normal pool because of the larger quantity of stored water. This type of event 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. Guidance for conducting dam break inundation analyses for flood conditions is more complicated than analyses for the normal pool condition because of the need to account for the magnitude of flood inflow and spillway outflow at the assumed time of failure.

C. Maximum High Pool (Top of dam)

At the end the reservoir level set to the top of the dam to represent the maximum amount of volume that may be stored in the reservoir. The condition is applicable in evaluating 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. In this particular case, the magnitude of the dam break flood is relatively insensitive to the regulated reservoir inflow and outflow because the inflow/outflow quantities are usually very small compared to the dam break flood.

2.5.2.2 Hydrologic Failure

Hydrologic breaches that occur with extreme precipitation and runoff are termed ―rainy day" or hydrologic failures. Put simply, a dam failure is more accurately defined as a hydraulic process than a hydrologic one. Because modeling a dam failure using hydraulic principles is usually more accurate than a hydrologic model because the modeler can more accurately simulate the shape of the reservoir, tail water effects, and drawdown effects.

Hydrologic failures that cause dam breach events are generally 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 regulations, could range from the 1-percent-annual-chance flood event (often called the 100-year flood) to a percentage of the PMF.

2.5.3 Tiered Dam Breach Analysis

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

A tiered study approach was developed by the USDOI and is presented in their report titled Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report (USDOI, 2011). The NDRSB EAP Workgroup also stated that although detailed studies often provide a more precise representation of potential flooding for a given set of assumptions, a more accurate representation of dam failure flooding is not necessarily provided (FEMA, 2013).

In their effort to increase the number of EAPs for dams, a tiered approach in dam inundation modeling has gained popularity with many State and Federal dam safety programs. Unlike the risk and Event based approaches discussed in Sections 2.5.1 and 2.5.2, the tiered approach is not used to determine the appropriate flood event to use in a dam failure analysis. Instead, the tiered approach is used to determine the appropriate level of complexity in the assessment, modeling, and mapping of a dam failure based on a dam‘s hazard potential, size, and the complexity of the downstream area under investigation.

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

In general, as the sophistication of the modeling increases, so does the level of effort, time, and cost necessary to conduct the analysis. Table 2-7 provides guidance to determine the tier level for analysis for dam failure inundation modeling and mapping. This table is arranged similarly to some State-developed tiered analysis structures, providing a logical combination of methods to perform an analysis. The dam failure analysis should be continued downstream to a point where the breach flood no longer poses a risk to life and property damage, such as the confluence with a large river or reservoir with the capacity to store the flood waters.

Table 2-7: Tiered Approach Dam Breach Inundation mapping for use in EAPs (FEMA, 2013)

Tier Level	Applicable to	Breach parameter prediction	Peak breach discharge prediction	Downstream routing of breach hydrograph
Tier 1: Basic level Screening And Simple Analysis	1.Low-hazard potential or small size 2.First level screening for significant- or high-hazard dams	Empirical Equations	Simplified Models (SMPDBK, GeoDam-BREACH, or Technical Release [TR]-66) or HEC-HMS	GeoDam-BREACH, SMPDBK,DSAT,1D HEC-RAS Steady State, or HEC-HMS Hydrologic Routing
Tier2: Intermediate	1.Significant hazard potential intermediate size 2.High-hazard dams with limited population at risk	Empirical Equations	HEC-HMS or HEC-RAS Unsteady Model	HEC-RAS (Steady or Unsteady Modeling) 1-D or 2-D models
Tier 3 Advanced	1.High-hazard potential / large size dams with sufficient population at risk to justify advanced analyses	Empirical Equations, NWS BREACH, or WinDAM	HEC-RAS Unsteady Model	HEC-RAS Unsteady Model or 2-D models

Tier 1 and 2 analyses are most appropriate for low-hazard potential / small sized and significant-hazard potential / intermediate-sized dams with a limited number of structures. More detailed surveying or modeling may be warranted for Tier 3 analyses for high-hazard potential / large-sized dams, those with a large population in the evacuation area, or those with significant downstream hydraulic complexities, such as major diversion structures, split flows, or potential for a series of dam failures such as a ―domino effect” (NDSRB, 2009).

2.6 Breach Parameter Estimation Methods

The main points in dam breach hydrograph analysis are estimation of dam breach parameters such as the geometry and timing (e.g., width, depth, shape, and time of failure) of the breach formation. If input breach parameters cannot be predicted with sufficient accuracy, more conservative parameters and associated increased costs may be required (Wahl, 1997). 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.).

Many and different methods are used by dam safety professional to estimate dam breach parameters and come up with the resultant dam breach peak discharge and timing. There are four critical elements in dam breach analysis of any types. These are: 1) estimation of breach parameter (i.e. breach bottom width, side slope of breach and time of failure), 2) estimations of breach peak discharge and breach hydrograph, 3) flood routing during failure and for d/s, and 4) estimation of the hydraulic conditions at critical locations. Unless these breach parameters are estimated properly it is difficult to come up with reliable peak outflow and resulting downstream inundation in close proximity to the dam. Due to this accurate prediction of breach parameters is necessary to make reliable estimates of breach peak outflow discharge and resulting downstream inundations for failure. In most case the important thing that has to been considered is the variation of breach parameters with reservoir size. For large reservoirs the peak discharge occurs when the breach reaches its maximum depth and width. In these cases, accurate prediction of breach geometry is most critical. For small reservoirs, there is significant change in reservoir level during the formation of the breach, and as a result, the peak outflow occurs before the breach has fully developed.

Therefore, different factors must be considered in selecting appropriate breach parameters estimation methods. Among these factors a few of them are dam type, dam dimensions and dam materials of construction. Other pertinent information such as historical records of seepage or foundation problems should also be considered (Gee, 2009). The most commonly used approaches for the required elements of the analysis are as follows.

2.6.1 Physically Based Erosion Methods

A physically-based model (also referred to as a ―process" or ―causal" model) utilizes generally accepted relationships based on physical principles to establish the framework of a model. 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. The model takes into account several components of a dam and reservoir that are not considered in the empirical methods, such as area versus elevation, dam dimensions, soil properties of the dam, and tail water effects downstream. A modified form of the Meyer-Peter and Muller sediment transport equation is used in this model. The model takes into account several components of a dam and reservoir that are not considered in the empirical methods, such as area versus elevation, dam dimensions, soil properties of the dam, and tail water effects downstream. It is relatively simple to run and is widely used within the United States. Acceptance of this model for hazard classification studies will be allowed with reasonable justification. The results must be validated with the other recommended methods.

2.6.2 Parametric Regression 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 2-8, adapted from USBR (1998), provides the most common parametric regression equations developed based on information from case studies of historic dam failures available at the time of this publication.

Table 2-8: Published Parametric Regression Equations for Predicting Breach Parameters (USBR, 1998)

References	Number of studies	Relation proposed(SI,units meter ,m3/s)
Johnson and Illes (1976)		$0.5h_d < B < 3h_d$ for earth-fill dams
MacDonald and Langridge-Monopolis (1984)	42	Earth-fill dams: $Ver = 0.0261(Vout^{*}hw)^{0.769}$ $T_f = 0.0179(V_{er})^{0.364}$ Non-Earth-fill dams: $V_{er} = 0.00348(V_{out}*_{h}w)^{0.852}$
Reclamation (1988)		$B = (3)hw$ $tf = (0.011)B$
Froehlich (2008)	63	$B_{ave} = 0.27\, k_o v_w^{0.32}\, h_b^{0.04}$ $T_f = 63.2\, (v_w/gh_b^{2})^{0.5}$
Froehlich (1995)	74	$B_{ave} = 0.1803\, k_o v_w^{0.32}\, h_b^{0.19}$ $T_f = 0.0025 v_w^{0.53}\, h_b^{-0.90}$

These empirical regression equations were developed to predict the average breach width, breach depth, and time-of-failure or formation time. Wahl (2010) suggests that one of the main advantages of using empirical parametric regression equations is that the user can exhibit some control over the breach parameters used in the model, and thus account for site-specific factors.

2.6.3 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. 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.

2.6.4 Comparative Analysis

Comparative analysis is the simplest approach to dam breach flood estimation. This method compares a given dam of interest with those in a database of well documented dam failure case histories. A given dam geometry, height, slope angles, and reservoir areas and 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.

2.7 Hydrologic Analysis

The hydrology for dam breach inundation studies involves determining the peak discharge and volume for extreme hydrological and meteorological events. The flood event to be evaluated in a dam breach analysis of Gilgel Gibe I is the recurrence-interval-based event (i.e., 25-, 50-, 100-, 500-, 1,000-year event) or a ratio of the PMF, depending on specific State dam safety requirements.

Perform full breach analyses for the following hydrologic conditions at a minimum:

- Sunny-day breach: Reservoir at its maximum normal operating pool level.
- Overtopping breach: Inflow design flood set to the percentage of the PMF or design flood that equals the top of the dam. If the dam passes 100 percent of the design flood without overtopping, this scenario does not need to be run.

Compare barely overtopping and design-flood breach runs to runs for the same event assuming that the dam does not fail. The simplified breach method for existing dams only reviews the impacts from a breach occurring with the reservoir at the effective crest of the dam. If the design-flood breach overtops the dam, the analysis will need to either assume flow over the top of the dam or not. The former adds complexity to the model as the length of the dam that is overtopped is reduced by the breach width.

2.8 Dam Breach Failure Modeling Tools

The unsteady (rapidly time varying) river flow hydraulics problems are solved through Dam break modeling. Since mid-1960s dam breach prediction models have existed. Due to several critical dam failures further development of dam breach models was realized in the 1970s. There are three categories of dam failure modeling techniques. Regression analysis which utilizes the available dam failure data (i.e. outflow hydrograph data and dam geometry) is the first technique. Analytical modeling the dam failure process through characterizing the physical processes with failure process to make predictions is the second category. The last technique is numerically modeling dam failure, overtopping, and flood wave routing with a computer model (Steininger, 2014).

Dam failure computer modeling can essentially be categorized into two major types of models, those that explicitly model the dam failure mechanism and outflow hydrograph and those that model the watershed scale hydrology and hydraulics in order to quantify the amount of water available to a reservoir and then route an outflow hydrograph from a dam. Coupling of these two types of models can also be done to simulate the entire process at a watershed scale and localized scale. Presented in the following sections are description of the current and past research with each of the aforementioned techniques and examples thereof. This review is not intended to be comprehensive, but rather a general picture of the available dam failure, overtopping, and flood wave routing modeling techniques will be presented.

2.8.1 Dam Breach Empirical Models

Although there have been thousands of man-made and natural dam failures, there is not an abundance of data available concerning dam failure events due either to a lack of downstream gaging or to downstream gages being compromised during the flood event (Steininger, 2014).

However, work has been done to compile the available data in order to estimate flood characteristic parameters as a function of dam geometry and reservoir geometry parameters by regression analysis. The magnitude of the peak discharge from a dam failure and the time to peak discharge are two important parameters due to their direct relation to downstream floodplain management. Several researchers such as Froehlich, Pierce and Singh over the past few decades have compiled both measured and estimated flood hydrograph data and, by regression analysis, related the peak discharge and time to peak discharge to various geometric characteristics of the failed dams or associated reservoirs. Some parameters that have been used in these types of regression analyses are: the maximum height of a dam, the depth of the water behind a dam, the volume of water behind a dam, and the crest length of a dam. Recently the results of many of the regression analyses that have been done over the past few decades were compiled for comparison and review (Thornton et al. 2011). Thornton et al. 2011 summarized the resulting empirical functions of many of these analyses and also presented a multivariated regression analysis utilizing the data sets that were used for the regression analyses. Table 2-9 shows the compiled summary of dam failure empirical relationships.

Table 2-9: Summary of dam breach hydrograph empirical relationships (Thornton et al. 2011)

	Investigator	Type	R^2	Number of case studies: Real	Number of case studies: Simulated	Equation
Height of water equations	Kirkpatrick (1977)	Best fit	0.790[a]	13	6	$Q_p=1.268(H_w+0.3)^{2.5}$
	Soil Conservation Service (SCS) (1981) for dams > 31.4 m	Envelope[b]	Not available	13		$Q_p=16.6(H_w)^{1.85}$
	U.S. Bureau of Reclamation (1982)	Envelope	0.724	21		$Q_p=19.1(H_w)^{1.85}$
	Singh and Snorrason (1982)	Best fit	0.488		8	$Q_p=13.4(H_d)^{1.89}$
	Pierce et al. (2010)—linear	Best fit	0.633	72		$Q_p=0.784(H)^{2.668}$
	Pierce et al. (2010)—curvilinear	Best fit	0.640	72		$Q_p=2.325\ \ln(H)^{6.405}$
Storage equations	Singh and Snorrason (1984)	Best fit	0.918		8	$Q_p=1.776(S)^{0.47}$
	Evans (1986)	Best fit	0.836	29		$Q_p=0.72(V_w)^{0.53}$
	Pierce et al. (2010)	Best fit	0.805	87		$Q_p=0.00919(V)^{0.745}$
Height of water and storage equations	Hagen (1982)	Envelope	Not Available	6		$Q_p=1.205(V_w\cdot H_w)^{0.48}$
	MacDonald and Langridge-Monopolis (1984)	Best fit	0.788	23		$Q_p=1.154(V_w\cdot H_w)^{0.412}$
	MacDonald and Langridge-Monopolis (1984)	Envelope	0.156	23		$Q_p=3.85(V_w\cdot H_w)^{0.411}$
	Costa (1985)	Best fit	0.745[c]	31		$Q_p=0.763(V_w\cdot H_w)^{0.42}$
	Pierce et al. (2010)	Best fit	0.844	87		$Q_p=0.0176(V\cdot H)^{0.606}$
	Froehlich (1995)	Best fit	0.934	22		$Q_p=0.607(V_w^{0.295}\cdot H_w^{1.24})$
	Pierce et al. (2010)	Best fit	0.850	87		$Q_p=0.038(V^{0.475}\cdot H^{1.09})$

[a]This R^2 value was calculated using a portion of the writer's original data set.

[b]Wahl (1998) suggested that this is an enveloping equation even though three data points plot slightly above the curve.

[c]This R^2 value was calculated without the five concrete and masonry dams included in the writer's original data set.

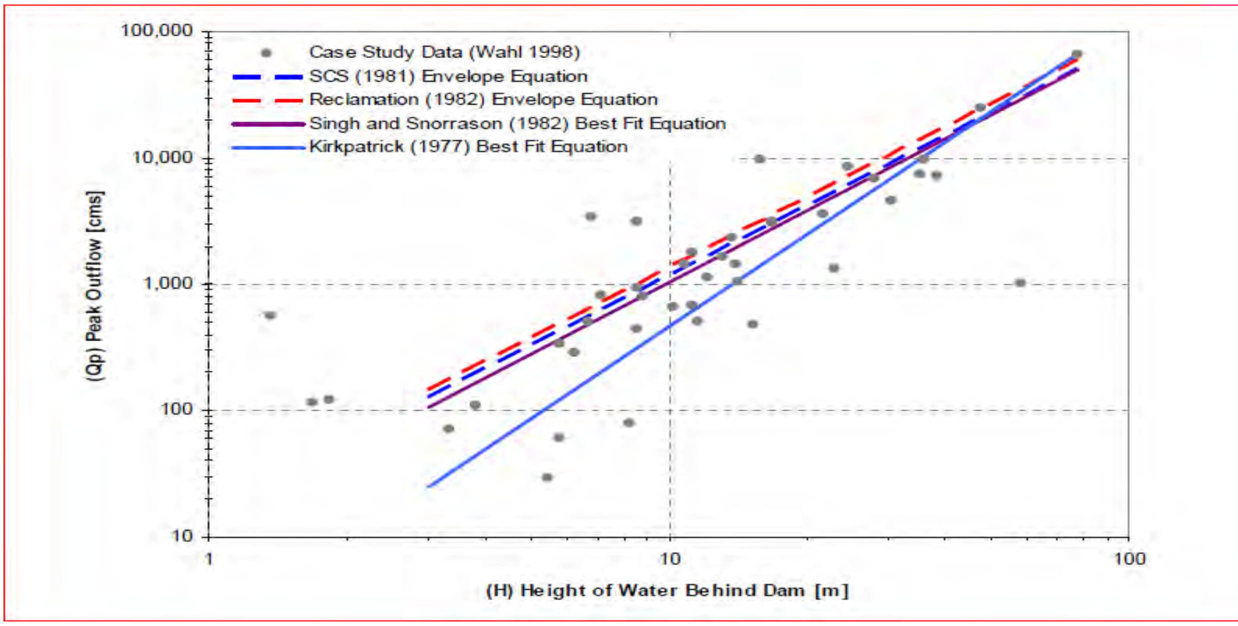

Figure 2-5: Dam Failure Data Sets (Thornton et al. 2011).

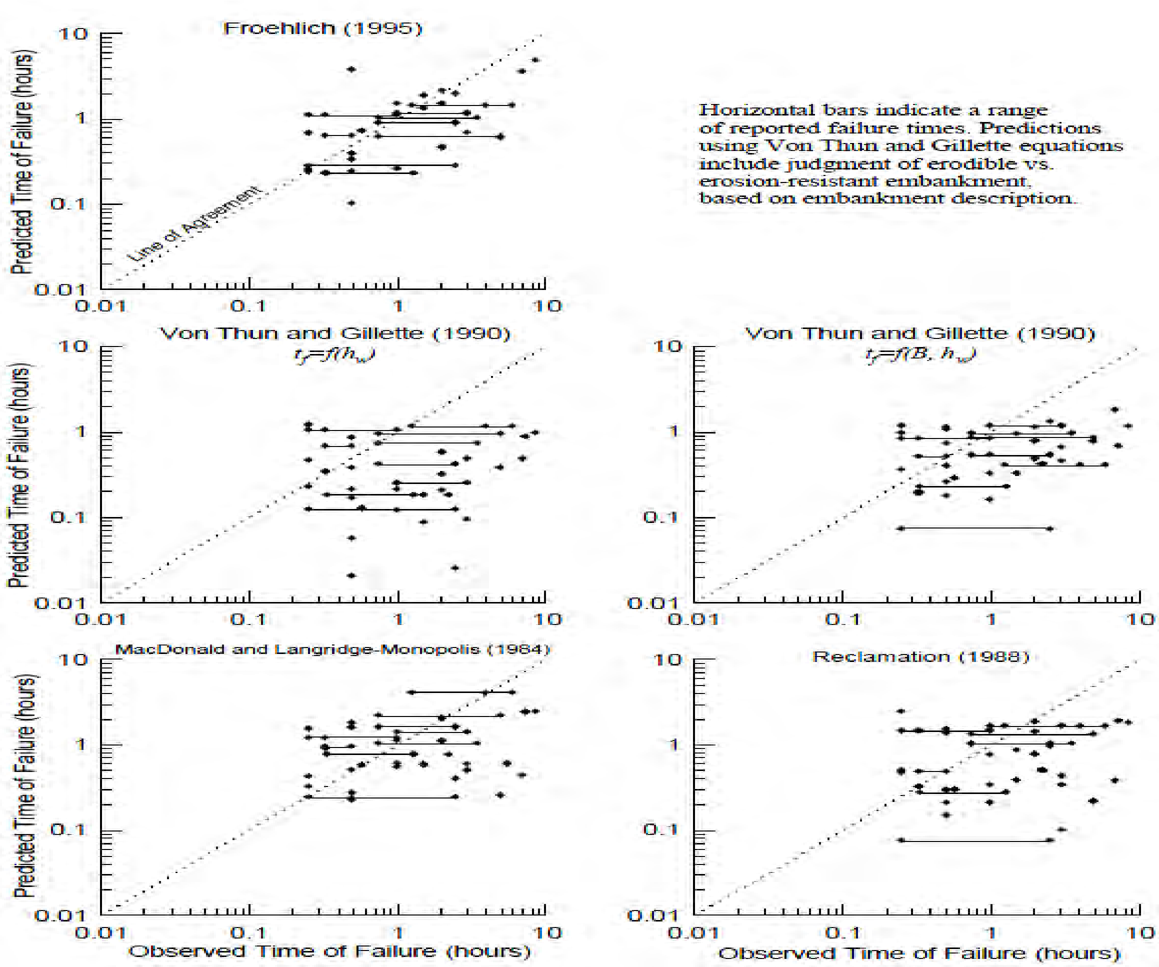

Figure 2-6: Predicted time of failure vs. observed time of failure (Wahl, 1998).

A multivariate regression analysis was performed using not only the volume of water behind the dam (V) and dam height (H), but also the average embankment width W_{avg} as dependent variables for predicting the peak outflow (Q_p) at embankment breach. The following relationships are the multivariate regression equations that were developed for the peak discharge from a dam breach:

$$Q_P = 0.863(V^{0.335}\, H^{1.833}\, W_{avg}{}^{-0.663}) \text{ ....Eq. (2-14)}$$

$$Q_P = 0.012(V^{0.493}\, H^{1.205} W_{avg}{}^{0.226}) \text{ .... Eq. (2-15)}$$

Where: V = volume of water behind the dam; H = dam crest height; W_{ave} = average embankment width (perpendicular to the crest); L = embankment length (crest length)

It was determined that when the number of pertinent dam characteristic variables increased from one to three as in these equations, the coefficient of variation increased slightly and the mean predicted error and the uncertainty bandwidth decreased (Thornton , 2011). Similar regression analyses have been done to develop equations for the time to failure of a breach outflow. Figure 2-6 was taken from the 1998 Department of the Interior Bureau of Reclamation Dam Safety Office report entitled ―Prediction of Embankment Dam Breach Parameters". Figure 2-6 shows predicted versus observed time of failure values as determined by different researchers such as Froehlich 1995, Von Thun and Gillette 1990, MacDonald and Langridge- Monopolis 1984, and Reclamation 1988. The relationships that have been developed by regression analyses of these data sets are tools for rough estimation of flood characteristics which can be helpful in emergency response and flood plain management. These relationships can also be used in conjunction with computer models.

2.8.2 Dam Break Analytical Models

While the water flow and erosion physical processes involved in dam failure are well known, they are still difficult to analytically model and quantify due to complex turbulence and rapidly varying characteristics. However, several analytical models have been developed that determine the discharge from a dam breach from mathematical formulations of the physics of the breach process. Cristofano, in 1965, related the erosion of a breach channel to the discharge though the breach using the shear strength of the dam material and the force of the flowing water. Several assumptions were made about the shape of the breach and an empirical coefficient was used to calibrate the model (Wahl, 2010).

Walder and O'Connor, in 1997, developed a model for the peak discharge from a breach as a function of the material erosion rate, the reservoir size, a breach shape parameter, the breach side slope angle, a reservoir shape factor and the breach depth to dam height ratio (Wahl, 2010). The following ~~–b~~enchmark case" relationships were developed for the peak discharge (Q_P) from a natural or constructed earthen dam breach and the time to peak discharge (t_P) (Walder and O'Connor, 1997).

$Q_p = 1.51(g^{0.5} h_d^{2.5})^{0.06} (k_b V_s/H_d)^{0.94}$, $t_p = 1.24(V_S/k_b^2 (gh_b)^{0.5})^{0.333}$…. for $(k_b/ (gh_d)^{0.5}) (V_s/h_d^3) <\approx 0.6$ ………………………………………Eq. (2-16)

$Q_p = 1.94(g^{0.5} h_d^{2.5}) (D_c/H_d)^{0.75}$, $t_p = h_d/k_b$ …for $(k_b/ (gh_d)^{0.5}) (V_s/h_d^3) >>1$ ……Eq. (2-17)

Where: g = gravitational acceleration; hd = water level drop in reservoir; k_b = mean erosion rate of the breach; Vs = volume of water stored behind the dam; Dc = dam crest height

As it is notice from the study, equation (2-16) applies to reservoirs where the volume stored to dam height ratio is small and equation (2-17) applies to reservoirs where the volume stored to dam height ratio is large. These formulations apply to average reservoir conditions for all parameters that are not present in the equations. This method recognizes the difference in dam failure processes between large and small reservoirs. Table 2-10 summarizes several popular physically based dam breach models that have been developed.

Table 2-10: Physically-based embankment breach parameters (Wahl, 1998)

Model and Year	Sediment Transport	Breach Morphology	Parameters	Other Features
Cristofano (1965)	Empirical formula	Constant breach width	Angle of repose	
Harris and Wagner (1967);BRDAM (Brown and Rogers, 1977)	Schoklitsch formula	Parabolic breach shape	Breach,dimensios, sediments	
DAMBRK (Fread,1977)	Linear predetermined erosion	Rectangular, triangular, or trapezoidal	Breach dimensions, others	Tail water effects
Lou (1981); Ponce and Tsivoglou (1981)	Meyer-Peter and Muller formula	Regime type relation	Critical Shear stress, sediment	Tail water effects
BREACH (Fread,1988)	Meyer-Peter and Muller modified by Smart	Rectangular, triangular, or trapezoidal	Critical shear stress, sediment	Tail water effects; dry slope stability
BEED (Singh and Scarlatos, 1985)	Einstein- Brown formula	Rectangular or trapezoidal	Sediments, Others	Tail water effects, saturated slope stability
FLOW SIM 1 and FLOW SIM 2 (Bodine,undated)	Linear predetermined erosion; Schoklitsch formula option	Rectangular, triangular, or trapezoidal	Breach dimensions, sediments	

2.8.3 Dam Breach Computer Models

In order to model the dam breach process and the downstream flooding process the National Weather Service (NWS) dam-break forecasting model FLDWAV was developed by D. L. Fread. FLDWAV took over for the popular DAMBRK model which has been used since the nineteen seventies (Fread 1984, 1993). FLDWAV utilizes a finite-difference numerical method to solve the complete one dimensional Saint Venant equations for unsteady flow. The model will compute the outflow hydrograph from a dam resulting from spillway flow, overtopping, and/or dam breach and then route the flood wave downstream. Internal boundary conditions can be input to represent man-made control structures such as dams, weirs, and bridges. The flow may be subcritical, supercritical, or mixed and can also vary from Newtonian to non-Newtonian (Fread & Lewis, 1998).

The BREACH dam breach model predicts the outflow hydrograph from an earthen dam using a physically based approach which takes into account various geometric, geotechnical, erosional, and flow characteristics (Fread, 1988). The model uses information about the constituent materials of a dam along with the Meyer-Peter and Müller sediment transport equation and a quasi-steady uniform flow (manning equation) to define the breach opening evolution in time. Subsequently the outflow hydrograph can be determined. The BREACH model code is free.

The United States Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) developed the HEC-RAS Hydraulic channel flow model as part of their suite of hydrologic and hydraulic modeling tools (Brunner, 2010). While primarily used as a flow routing model, a dam breach module has been added to the model to simulate the breach process. HEC-RAS can simulate steady or unsteady one-dimensional flow by solving the full one dimensional Saint-Venant equations. Subcritical, supercritical, or mixed flow regimes can be simulated by HEC RAS.

2.8.4 Watershed Computer Models

Over the past several decades, many watershed scale computer models have been developed within government agencies, academia, and the private sector. Watershed models are often categorized into lumped parameter, semi-distributed parameter, and distributed parameter models. Lumped parameter models are those which assign one parameter value to the whole watershed. Semi-distributed models are those which distribute parameter values by sub-catchments within a watershed. Distributed parameter models divide a watershed into a grid of cells and assign a parameter value to each cell within the watershed domain. Several in-depth comparative reviews of watershed models have been done. The World Meteorological Organization (WMO) has sponsored comparative studies of watershed models in 1975, 1986, and 1992. Singh et al. have written comparisons as well (Singh et al. 2002). The National Weather Service (NWS) sponsored a review of distributed models called the distributed model inter-comparison project (DIMP) (Smith et al. 2004).

The United States Army Corps of Engineers (USACE) has developed a series of lumped parameter watershed models. The most recent version is the Hydrologic Engineering Center (HEC) Hydrologic Modeling System (HMS) (Feldman, 2010). This model simulates watershed scale processes using empirical equations. This model and similar lumped parameter models are simple to use and require far less set up time and field data to run than distributed models. In many cases they can be as accurate as a more sophisticated physically based model. However, they do not represent the runoff characteristics of complex watersheds which have highly varied soil types or land uses as well as distributed parameter models. They will not provide information about the distribution of flow within a watershed. Also they always require calibration, which essentially limits their utility to cases where calibration and validation data are available.

An example of a semi-distributed parameter model is the Hydrologic Simulation Program-Fortran (HSPF) (Bicknell et al. 1997). This model has its roots in one of the oldest watershed models, the Stanford Watershed Model (Crawford and Linsley, 1966). This model simulates many hydrologic, sediment transport, and chemical transport processes. The hydrologic processes are represented as stored water and flow is routed between storages (Velleux et al. 2010). Flow is simulated with the one-dimensional kinematic wave approximation of the Saint Venant equation.

The Kinematic Runoff and Erosion (KINEROS) is another example of a semi-distributed parameter model (Woolhiser et al. 1990). This model is an example of an ―open book" model whereby a watershed is represented by planes which route flow into channels. KINEROS simulates rainfall, interception, infiltration, surface runoff, and erosion. Flow is calculated by the one-dimensional kinematic wave approximation of the Saint Venant equation. The Soil and Water Assessment Tool (SWAT) is another example of a physically based semi-distributed parameter model which simulates rainfall, infiltration, surface flow, groundwater flow, and transmission losses (Neistch et al. 2002). SWAT has been linked with the Arc/Info geographic information system (GIS) (Velleux et al. 2010). All three of these semi-distributed parameter models have publicly available source code.

System Hydrologic European Fortran (SHETRAN) is a fully-distributed parameter, physically based model which simulates interception, infiltration, surface runoff, groundwater flow, evapotranspiration, sediment transport, and chemical transport (Ewen et al. 2000). Surface flow is calculated with the diffusive wave approximation of the Saint Venant equation. This model is two-dimensional in the overland plane and one dimensional in channels. Groundwater flow is three-dimensional. While SHETRAN is not commercially available, there is a commercially available package called MIKE SHE (DHI, 2005). FLO-2D is a two-dimensional physically based model which simulates rainfall, surface flow, interception, and infiltration (O'Brien, 2006). FLO-2D uses the full dynamic wave Saint-Venant equation to route flow in two dimensions. This software is commercially available and there is a free basic version.

Several features of a computer model are necessary or highly desirable when modeling dam overtopping and large magnitude flood events at the watershed scale. A model must be a fully distributed parameter type in order to analyze the interactions between the floodplain and the channel and to map the distribution of flow within a watershed in time. Floodplain interactions are complex due to highly varied roughness and many possible flow directions. Fully distributed models best capture this detail. Also location specific events like overtopping and dam failure must be modeled with a fully distributed model in order to accurately represent the localized detail at a dam site. A model should also route flow in two dimensions in the flood plain. Large scale flood flows are very multi-dimensional in nature as the flows are not confined by channel walls. Modeling floods that originate from a point within a watershed requires a model that is capable of accepting a user defined point source hydrograph as input. The incorporation of a GIS program into the pre-processing of model input data and post-processing of model calculated, distributed flow depths and velocities is crucial for this type of modeling. It allows for easy modification of watershed elevations for dam representations. It also provides a vehicle for visually interpreting model outputs in the form of maps and animations.

2.9 Dam Breach Modeling Tools

Performing dam breach model involves prediction of the dam breach hydrograph and the routing of that hydrograph downstream (FEMA, 2013). A number of modeling tools are available to perform dam breach modeling, ranging from simple methods to complex models. With advancements in GIS-based modeling, many models can interface with digital terrain data to produce automated dam breach inundation zone delineations.

Dam breach modeling involves tools that generate the dam breach peak discharge and /or hydrograph only and tools that develop a breach hydrograph and perform downstream flood routing using a one- or two-dimensional hydraulic model.

Simplified numerical models typically relate the breach hydrograph (or breach peak flow) to simple reservoir characteristics such as reservoir volume and dam height. These models may or may not include hydrologic modeling to determine the envelope maximum water depths to calculate the breach flow which do not consider complicated downstream conditions such as backwater effects. Table 2-11 shows the most widely used dam breach modeling tools/models and their general capabilities.

There are two types of dimensional hydraulic models. These are one-dimensional models and two- dimensional models. One-dimensional models solve either full dynamic or simplified forms of one-dimensional, cross-section-averaged shallow water equations. 1-D models are more sophisticated than simplified numerical models and typically consider backwater effects; many one-dimensional models are capable of dynamic reservoir routing rather than level pool routing.

One-dimensional routing is best suited for modeling flow through a well-defined, confined channel. For routing over wide, flat surfaces, such as floodplains, one- dimensional models make certain assumptions (such as uniform flow velocity over a cross- section) that are not true and can have significant consequences on the accuracy of the model.

Two-dimensional models 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. Two-dimensional models are capable of routing flow over unconfined floodplains where flood waters are not contained within a defined channel.

2.9.1 Dam Breach Hydrograph and Peak Outflow Generation Tools

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

Table 2-11: Most Widely Used Dam Breach Modeling Tools (FEMA, 2013).

Method	**Computation of Peak Breach Outflow**	**Computation of Ultimate Breach**	**Breach Hydrograph Generation**	**Downstream Routing Capability**			
				Steady	**Unsteady**	**1-D**	**2-D**
Breach Hydrograph Generation Only							
Empirical	✓	✓					
NWS BREACH	✓	✓	✓				
USACE HEC-1 & HEC-HMS	✓		✓	Without downstream hydrologic routing	✓		
Breach Hydrograph Generation and Downstream Hydraulic Routing One-Dimensional Models							
NRCS TR-66	✓		✓	✓	✓	✓	
WinDAM	✓	✓	✓				
NWS SMPDBK	✓		✓	✓		✓	
NWS FLDWAV	✓	✓	✓		✓		

USACE HEC-1 and HEC-HMS	✓		✓	Downstream hydrologic routing		✓	
USACE HEC-RAS	✓		✓	✓	✓	✓	
Two-Dimensional Models							
DSS-WISE	✓		✓		✓		✓
FLO-2D©	✓	✓	✓		✓	✓	✓
MIKE© FLOOD	✓		✓		✓	✓	✓

In order to generate dam breach outflow hydrograph and downstream routing analysis for Gilgel Gibe dam, United State Army of Corps Engineers, Hydraulic Engineering Center River Analysis System (USACE, HEC-RAS 4.1) was selected for this study.

2.10 Routing Breach Outflow Hydrographs through Downstream Reaches

Routing Breach Outflow Hydrographs is needed to compute the discharge, water surface elevation, and velocity throughout the river reach. Dam-break flood hydrograph is a dynamic and unsteady situation with which HEC RAS use for analysis. Therefore, the preferred approach is to utilize a fully developed Unsteady State flow routing model. The implicit formulation of the St. Venant equation is well-suited from the standpoint of accuracy for formulating unsteady flows in a natural channel. The following assumptions are necessary for derivation of the Saint-Venant equations (FEMA, 2008):

- The flow is one-dimensional; depth and velocity vary only in the longitudinal direction of the channel. This implies that the velocity is constant and the water surface is horizontal across any section perpendicular to the longitudinal axis.
- Flow is assumed to vary gradually along the channel so that hydrostatic pressure prevails and vertical accelerations can be neglected.

- The longitudinal axis of the channel is approximated as a straight line.
- The bottom slope of the channel is small and the channel bed is fixed; that is, the effects of scour and deposition are negligible.
- Resistance coefficients for steady uniform turbulent flow are applicable so that relationships such as Manning's equation can be used to describe resistance effects.
- The fluid is incompressible and of constant density throughout the flow.

The process of routing is used to predict the temporal and spatial variations of a flood hydrograph as it moves through a river reach or reservoir. The effects of storage and flow resistance within a river reach are reflected by changes in hydrograph shape and timing as the flood wave moves from upstream to downstream. Figure 2-7 shows the major changes that occur to a discharge hydrograph as a flood wave moves downstream. Routing serves the useful purpose of deriving the hydrographs from rainfall distributions, estimating the water yield at a specified point, developing design elevations of flood.

In general, routing techniques may be classified into two categories: hydraulic routing, and hydrologic routing. Hydraulic routing techniques are based on the solution of the partial differential equations of unsteady open channel flow. These equations are often referred to as the Saint Venant equations or the dynamic wave equations. Hydrologic routing employs the Continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the outlet.

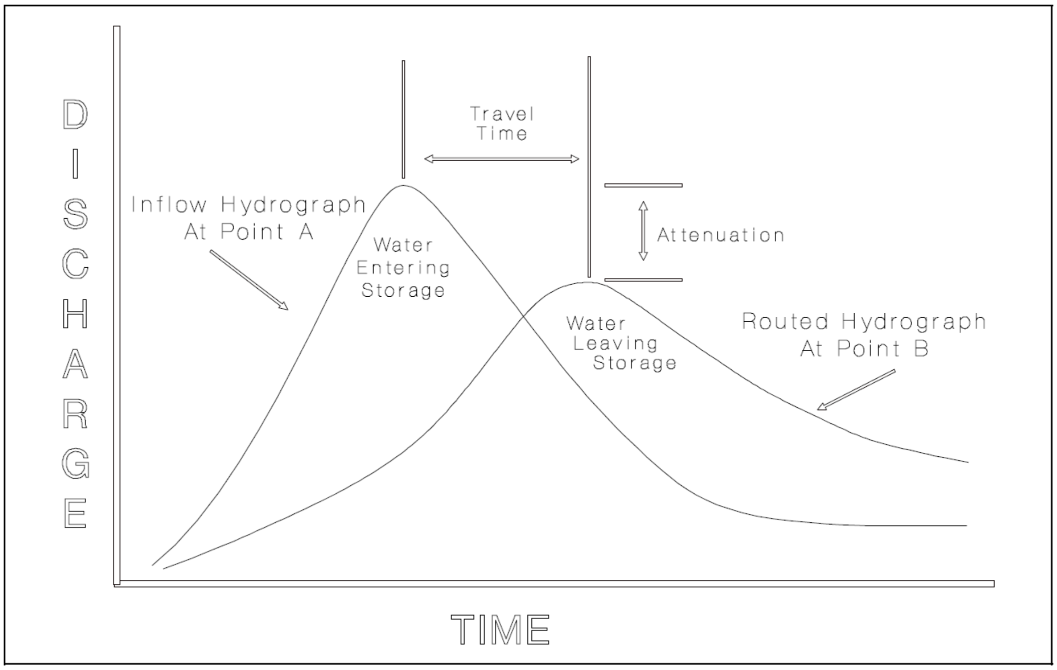

Figure 2-7: Discharge Hydrograph Routing Effects (USACE, 1994).

Therefore, HEC-RAS is chosen for unsteady state flood routing, and this technique simultaneously computes the discharge, water surface elevation, and velocity throughout the river reach. Parameters needed in running HEC-RAS to perform unsteady flow routing are channel geometry and boundary conditions, flood hydrograph routing (hydraulic routing and hydrological routing) and roughness coefficient.

2.10.1 Flood Hydrograph Routing

Hydraulic routing and hydrological routing are the two routing techniques. Hydraulic routing techniques are mostly based up on the solution of the partial differential equations of unsteady open channel flow. This equation is often referred to as the Saint Venant equations or the dynamic wave equations. Hydrologic routing employs the Continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the outlet.

2.10.1.1 Hydraulic Routing Techniques

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 equations (USACE, 1994). 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 consists of steady and unsteady flow analysis. Steady and unsteady flow analyses are different through 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 (USACE, 1993). Among software HEC-RAS can be selective in analyzing steady and unsteady flow. The HEC-RAS computer program uses equations that describe 1-D unsteady flow in open channels, the Saint Venant equations consist of the Continuity equation and the Momentum equation. The solution of these equations defines the propagation of a flood wave with respect to distance along the channel and time.

The Continuity equation originates from the law of conservation of mass. In figure 2-8, q is the lateral inflow rate per unit length of channel. Q and A stands for initial discharge and cross-sectional area. All variables are functions of time and space.

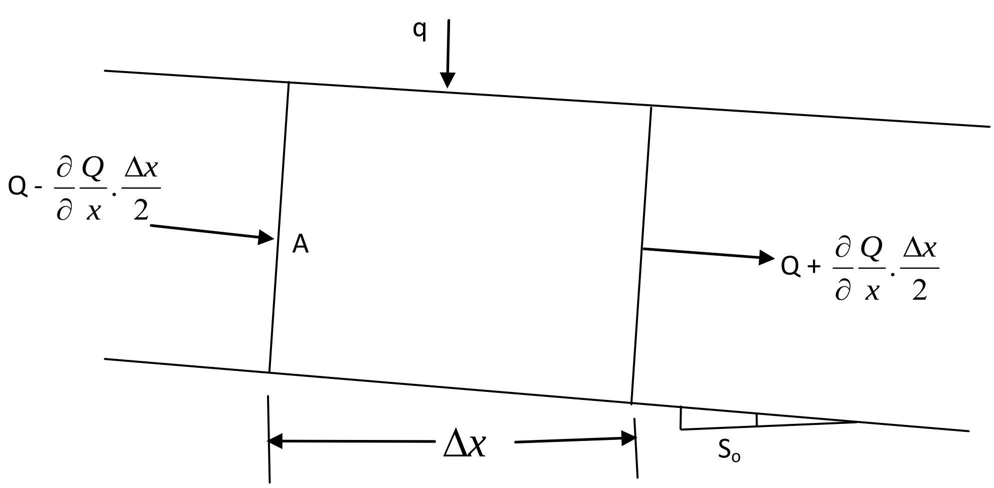

Figure 2-8: Control volume for the Continuity equation (Gupta, 2001).

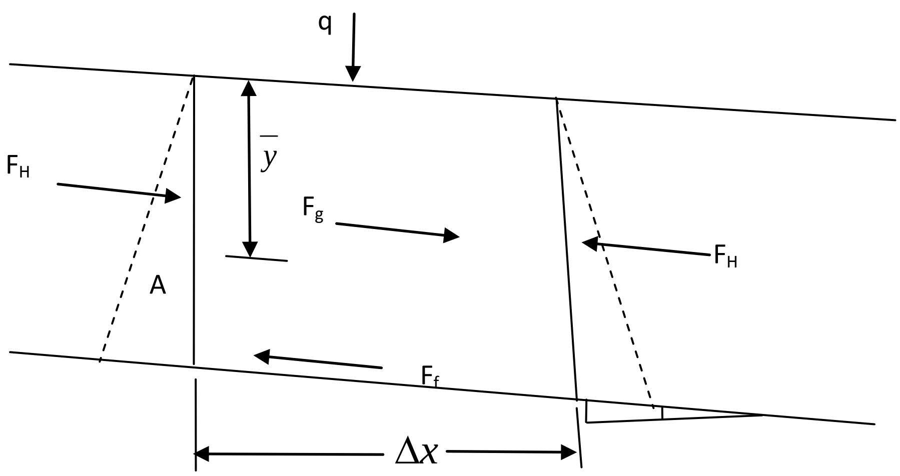

Figure 2-9: Control volume for the Momentum equation (Gupta, 2001).

$$\text{Inflow} = \left(Q - \frac{\partial Q}{\partial x} \cdot \frac{\Delta x}{2}\right)\Delta t + q\Delta x\Delta t$$

$$\text{Outflow} = \left(Q + \frac{\partial Q}{\partial x} \cdot \frac{\Delta x}{2}\right)\Delta t$$

The rate of change in volume stored within the element is equal to the change in cross-sectional area multiplied by the length of section and time, i.e.

$$\text{Storage change} = \frac{\partial A}{\partial t}\Delta x\Delta t$$

According to the conservation law: Input – Output = Rate of change in volume

Substitute the inflow, outflow and storage change in the conservation law and divide by Δx ,

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \quad \text{...Eq. (2-18)}$$

For a unit width b of channel with v average velocity, the continuity equation can be written a

$$\frac{\partial y}{\partial t} + v\frac{\partial y}{\partial x} + y\frac{\partial v}{\partial x} = \frac{q}{b} \quad \text{.....................................Eq. (2-19)}$$

The Momentum equation is the x-direction is produced from a force balance on the river element, according to Newton‘s second law of motion. The following three main forces are acting on area A as shown in Figure 2-9.

Hydrostatic: $F_H = -\gamma \dfrac{\partial(\bar{y}A)}{\partial x}\Delta x$

Gravity: $F_H = \gamma A S_o \Delta x$

Friction: $F_H = -\gamma A S_f \Delta x$

The rate of change of momentum is expressed from Newton‘s second law as:

$$F = \frac{d}{dt}(mv)$$

Where the total derivative of v with respect to t can be expressed

$$\frac{dv}{dt} = \frac{\partial v}{\partial t} + v\frac{\partial v}{\partial x}$$

After equating the sum of the three external forces and make some simplifications for negligible lateral inflow and a wide channel, the equation can be rearranged to yield a complete Momentum equation:

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t} \text{ ……………………………………Eq. (2-20)}$$

Where: γ = specific weight of water; $\bar{y}$ = distance from the water surface to the centroid of the pressure prism; Q = Inflow; A = Cross-sectional flow area; v = average velocity of water; x = distance along channel; b = water surface width; y = depth of water; t = time; q = lateral inflow per unit length of channel; S_f = friction slope; S_o = channel bed slope; g = gravitational acceleration

The most accurate and comprehensive solution to 1-D unsteady flow problems in open channels are the Continuity and Momentum equations. Continuity and Momentum equations have specific assumptions and limitations. The assumptions used in deriving the 1-D unsteady flow equations are as follows:

a. Velocity is constant and the water surface is horizontal across any channel section.

b. All flows are gradually varied with hydrostatic pressure prevailing at all points in the flow, such that vertical accelerations can be neglected.

c. No lateral secondary circulation occurs.

d. Channel boundaries are treated as fixed; therefore, no erosion or deposition occurs.

e. Water is of uniform density, and resistance to flow can be described by empirical formulas, such as Manning's and Chezy's equation.

f. Solution of the full equations is normally accomplished with an explicit or implicit finite difference technique. The equations are solved for incremental times (t) and incremental distances (x) along the waterway.

2.10.1.2 Hydrologic Routing Techniques

Hydrologic routing mostly based on continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the end. The hydrologic routing models offer the advantages of simplicity, ease of use and computational efficiency under the absence of significant backwater effect (USACE, 1994). When Hydrologic routing models used it is easy to attenuated flow hydrographs at desired locations but it is not provide information on water surface elevations or flow velocities.

Hydrologic routing combines the continuity equation with some relationship between storage, outflow, and inflow. These relationships are usually assumed empirical, or analytical in nature. In its simplest form, the Continuity equation can be written as inflow minus outflow equals the rate of change of storage within the reach:

$$I - O = \frac{\Delta S}{\Delta t} \quad \text{...Eq. (2-21)}$$

Where: I = the average inflow to the reach during t*;* O = the average outflow from the reach during t*;* S = storage within the reach

For the purpose of this study, the Modified puls reservoir routing method of HEC-1 was selected to conduct reservoir routing. The Modified puls method applied to reservoirs consists of a repetitive solution of the Continuity equation. It is assumed that the reservoir water surface remains horizontal and therefore, outflow is a unique function of reservoir storage. The Continuity equation, Eq. 2-21, can be manipulated to get both of the unknown variables on the left-hand side of the equation:

$$\left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) = \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 + \frac{I_1 + I_2}{2} \quad \text{…………………… Eq. (2-22)}$$

Since ―I‖ is known for all time steps, and O_1 and S_1 are known for the first time step, the right-hand side of the equation can be calculated. The left-hand side of the equation can be solved by trial and error. This is accomplished by assuming a value for either S_2 or O_2, obtaining the corresponding value from the storage-outflow relationship, and then iterating until Eq. 2-22 is satisfied. However, this iterative procedure can be done using a computer program that would produce fast and accurate results (FEMA, 2008).

2.11 Inundation Mapping

Dam failure inundation map is a map depicting the area of the downstream from a dam that would reasonably be expected to be flooded in the event of a failure of dam (Colorado dam safety branch, 2010). 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. Dam breach inundation studies are used for multiple purposes, including:

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

public of the risk living downstream of dams.

These purpose uses has unique information requirements and may be used in different manners as it is desired. This may range from multi- office-based planning efforts by mitigation planners and dam safety officials to field-based emergency responders responding to a developing or imminent dam breach (USACE, 1993).

Two reservoir conditions, normal pool level for piping and maximum pool storage elevation for overtopping, are usually examined in assessing the downstream consequences. The inundation delineation provides the most important dataset for inundation mapping, the inundation polygon. Information boxes are provided on the maps providing specific information related to the dam breach flood wave at various locations. Information provided includes distance from the (km), peak flood elevation (m), peak breach elevation (m), depth of flooding (m), time of arrival of the flood wave (hrs) and time to reach the peak stage (hrs).

2.12 Preparing Emergency Action Plan

An Emergency Action Plan (EAP) is a formal document that identifies potential emergency conditions at a dam and specifies preplanned actions to be followed to minimize property damage and loss of life. In real world, 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.

2.13 Dam Breach Remedial Measures

Remedial measure is primary the backbone for any dam under operation. When dams are not inspected, operated and maintained in appropriate time with appropriate monitoring manner the failure of dam will occur. To reduce such problem it is must to operate and inspect properly.

When dams are not properly operated and inspected the service life of dam will reduce due to failure. Before failure of dam happens it is better for client/concerned body to take appropriate measures. Among failure of dam piping/leakage is one which occur due to foundation crack, lack construction material quality, geologic condition etc. whereas overtopping failure occur mainly due to loss of freeboard, inadequate spillway capacity, lack of sediment management and embankment settlement. The ground conditions and the geological features of the dam site greatly influence the amount of seepage and its relevant effects (ODNR, 1994, 2003; Wiesner and Ewert, 2013). Many seepage problems and dam failures have occurred because of inadequate seepage control measures or incomplete cleanup and preparation of the core, foundation and abutments of the dam.

Therefore, seepage must be controlled to prevent erosion of the dam or foundation, and thus to maintain its global integrity and stability. However, seepage controls that occur after construction are difficult and quite expensive (Dunbar and Sheahan, 1999). It is not usually attempted unless the seepage has lowered the pool level or is endangering the dam or appurtenant structures. The need for seepage control will depend on the quantity, content, and location of the seepage. Measure that reducing seepage, such as use of an upstream blanket, the installation of a cutoff wall, or a grout curtain are typically used.

When PMF or large design floods exceeds the crest of dam the overtopping failure occur which is due to insufficient storage or in inadequacy of spillway capacity. In such case the addition of storage or release capacity becomes impractical or too costly; dam owners must sometimes resort to providing overtopping protection. Based on recent and ongoing overtopping protection research, the Federal Energy Regulatory Commission FERC has decided that embankment protection has sufficient merit to allow consideration on a project-specific basis (Frizell, et al 1991). The use of traditional in situ reinforced concrete has been enhanced as a major choice to achieve reliable stability and performance due to improvements in design methods and placements.

Covering the crest through concrete, vegetation and provision of stone pitching, increasing free board and crest width, increasing spillway capacity etc have recently been used on several embankment dams in the US at a significant cost savings and effective.

3 DESCRIPTION OF STUDY AREA AND METHODOLOGIES

3.1 Description of Study Area

The Gilgel Gibe Dam I is located in the Kefa province about 260 km south-west of Addis Ababa and about 70 km north-east of Jimma ($7^{O}50'$N, $37^{0}20'$ E). The reservoir is located at $7^{o}49`52.45``$N latitude and $37^{o}19`18.79´´$E longitude. Figure 3-1 shows the project area.

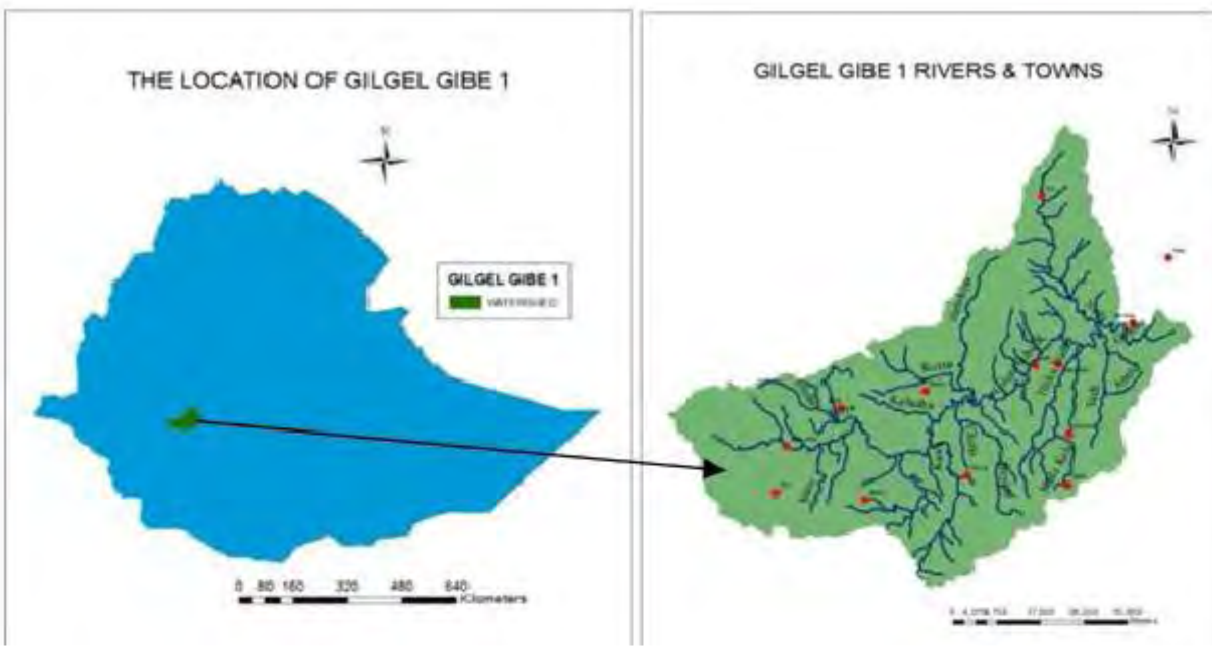

Figure 3-1: Study Project Area Map.

3.1.1 Climate Characteristic

Two-season tropical climate exist in Ethiopia. The dry winter season occurs between October and April and the rainy season (during the summer months) occurs between May and September. The average annual air temperatures of 19.2 °C exist in Gibe. The climate of the Omo-Gibe River Valley varies from a hot arid climate in the southernmost parts of the floodplain to a tropical humid one in the highlands that include the extreme north near Bako, the areas surrounding Jima and around the headwaters of the Gojeb River.

In accordance to average values, 60% of the total amount of annual rainfall occurs within the June-September period, 30% in the February-May period and 10% in the October-January period. Since 1955 there have been at least 10 rainfall stations in operation in the Basin at any one time. Of these, Jimma (with more than 35 years of record) and Bonga (with about the same) have good long term records. There are several other stations with over 10 years of record, though these records are not always continuous.

From this general view, the annual average rainfall varies from a minimum value of about 1,300 mm near the confluence with the Great Gibe stream, to a maximum of about 1,800 mm around the Utubo and Fego mountains. As usual it is possible to note a clear decrease in rainfall going downstream of the catchment area, corresponding to decrease in height above sea level.

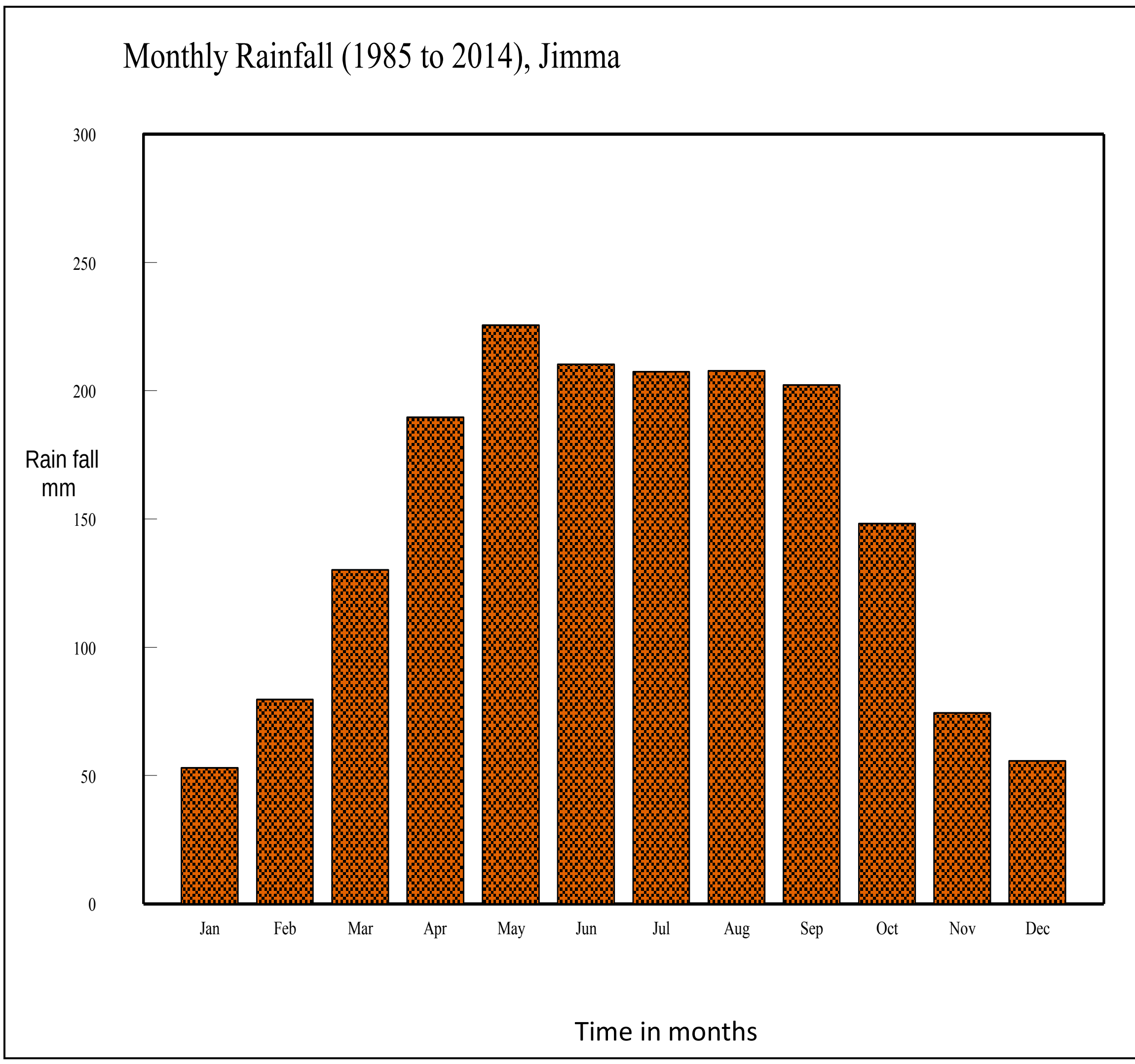

Figure 3-2: Mean Monthly Rainfall of Gilgel Gibe (MoWIE, 2004)

3.1.2 Omo Watershed

Most of Ethiopia is characterized by tropical climate moderated by altitude with a marked wet season. The southwestern Ethiopia is characterized by humid tropical climate with rainfall higher than 1,000 mm. The Gibe River raises just north of latitude 9°N and longitude 37°E on the Ethiopian Plateau and direction of flow is southwards towards the Omo River. The area is a fairly flat plateau about 1,650 m a.s.l with the highest peak reaching 3,359 m a.s.l and consists of a series of gentle sloping low hills and broad plains surrounded by hills or mountains.

The catchment area of the Gilgel Gibe basin is about 5,125 km^2 at its confluence with the great Gibe River and about 4,225km^2 at the dam site. The average annual flow in the area is 56.1 m^3/s. The Gilgel Gibe basin which has tributaries drains in to the Gilgel Gibe I reservoir is located in between 7^o 19´07.15´N and 8^{o}12´09.49´´N latitudes and 36^{o}31´42.60´´ E to 37^{o}25`16.05´´E longitudes. The Gilgel Gibe is a tributary of the Great Gibe River, known as the Omo River further downstream.

Figure 3-3: Sketch Map of the Omo-Gibe River catchment (MoWIE, 2004)

3.1.3 Topography

In the study area, the upper stretch of the river between Asendabo and the Deneba waterfall presented a winding fairly flat course. The right bank was flat or very slightly hilly at most and left bank was steeper. Approaching the Deneba waterfalls, at approximate elevation 1,620m a.s.l., the river banks became steeper. All the structures and Reservoir are located in the Jima zone. The vegetation in the Omo-Gibe Basin has been greatly modified by the effects of land use.

Climatic vegetation of broadleaf forest and wooded grassland or shrub would be existing in the area. The most extensive areas of forest are on high ground, particularly to the west of the Basin.

Generally the topography is characterized by high relief hills and mountains with an average elevation of about 1700 m above mean sea level. In general terms, the Gilgel Gibe basin is characterized by a wet climate with an average annual rainfall of about 1,550 mm and an average temperature of 19^{o}C. The seasonal rainfall distribution takes a uni-modal pattern with its maximum during the summer and minimum during the winter, influenced by the intertropical convergence zone (ITCZ).

3.1.4 Geology

The Gilgel Gibe is situated on the southwestern Ethiopian plateau. The area is characterized by a series of basic and subsilicic effusive volcanic rocks, frequently inter-layered with reddish paleosiols of Tertiary age. The rocks of the area are tentatively ordered as following, beginning with the youngest rocks: Trachytic tuff, Vesicular basalt, Aphyric augite basalt, Welded tuff, Augite basalt and Augite trachyte.

Over the upper reservoir, these rocks are covered with fluvio-lacustrine sediments. The entire volcanic sequence is frequently blanketed by thin, residual, subtropical lateritic soils, which have been formed on hill and ridge foot slopes. As well, they are covered with thick, black, plastic clay deposits on the flatter areas and valley. The hills on the right side of the Gilgel Gibe River, downstream of the waterfalls, are mostly covered to an elevation of about 1,800 m a.s.l. by thick colluviums deposits together with deeply weathered landslide and/or rockslide material.

3.2 Methodologies

3.2.1 Data Collection and Processing

Most of the original data collected and compiled by the Ministry of Water, Irrigation and electric and EEPCO were used in this breach analysis. The main purpose of Collecting data in creating a modeling methodology was for defining the size, type, elevation and storage relations of the subject dam, and the geometries of the downstream river reaches. Data collected for modeling were grouped into the following categories:

A. **Reservoir Characteristics**: The reservoir characteristics consist of reservoir storage elevation curve and reservoir surface area elevation curve. The maximum normal water level and total storage capacity are 1,671 m above sea level and 1,000 Million m^3, respectively. About 860 Million m^3 live storage with catchment area of 51 sq. km. The minimum normal water level and the crest of the main dam are 1,659.8 m above sea level land 1,673 m above sea level. The summary of Gibe reservoir and dam characteristics were presented in Table 3-1.

Table 3-1: Gibe Reservoir and Dam Characteristics.

Gibe Dam and Reservoir description	**Type/size**
Reservoir Characteristics	
Maximum Water Surface Elevation(m)	1,671
Full storage capacity(m3)	$1{,}000*10^6$
minimum Normal water level(m)	1659.8
Live storage(m3)	$860*10^6$
Dam Characteristics	
Type of Dam	Rock fill
Height of dam(m)	37
Elevation of Dam crest(m)	1673
Length of crest(m)	1,704
Elevation of spillway(m)	1663
Dam width(m)	7
Height of spillway(m)	27
Maximum spillway capacity(m3/s)	1,400

B. **Dam and spillway Characteristics**: This category includes data about name of dam, dam type, dam size, location of the dam, elevation of downstream toe of dam, design water storage pool elevation, maximum flood surcharge elevation, spillway crest elevation, crest of dam elevation, and height of the dam measured from downstream toe to the crest, and category of the dam. Gibe dam is 37 m high rock fill structure at crest elevation of 1,673 m above sea level with impervious bituminous facing on the u/s. The spillway crest length, width (upstream to downstream) and height are 69, 24 and 21 m respectively. The structure has been located in the left abutment (i.e. uppermost part) and its foundation is placed on sound basalt. The crest sill level of Spillway is at 1,663 m above sea level.

C. **Downstream Information**: Data gathered under this category includes bank stations, reach stations, downstream developments, cross section plots, manning roughness coefficients, other pertinent hydro structures and also Downstream Community.

D. **Inflow Hydrograph**: The inflow hydrograph data category includes the flood events hydrograph provided by the dam owner. In addition, it includes a modified ―sunny day" hydrograph used to breach the Gibe Dam. Generally, for the dam breach model simulation of Gibe Dam, the probable maximum flood (PMF) has been considered as inflow to the reservoirs. The PMF is the 48 hours 0.5PMF on which the analysis of dam breach of Gilgel Gibe has been undertaken with as significances of ensuring degree of hazard classification of failure of the dam. Even though the analysis of dam failure occurs both in overtopping and piping modes of failure the main input in HEC RAS models as reservoir inflow is the PMF of 48 Hours 0.5PMF.

3.3 Dam Breach Analysis Procedures

The parameters of dam breach depend on type of the dam and mode of failure. The shape and duration of the breach, together with the size of the dam and the reservoir, would determine, to a great extent, the characteristics of the breach outflow hydrographs. Piping and overtopping mode of failures are assumed in this study because most of dams have failed due to these modes. Generally performing a dam breach model involves prediction of the dam breach hydrograph and the routing of that hydrograph downstream.

3.3.1 Predicting the Outflow Hydrographs

For flood hydrograph estimation, the breach modeled by defining acceptable dam breach parameters was done as the main part in study dam breach. Predicting the outflow hydrographs at the dam location was done using HEC-RAS under different scenarios. The process of predicting the outflow hydrograph was a multi-steps approach and began with defining the reservoir characteristics, physical description of Gilgel Gibe, and its detailing breach characteristics.

3.3.1.1 Defining the River Geometry

Using HEC-RAS the first task after creating the project is to create the geometry of the river. This can be done by importing the river geometry from ArcGIS by using software HEC-GeoRAS which is modeled to integrate the two software‘s. The geometry of the Gibe River generated from ArcGIS was exported to HEC-RAS as shown in figure below.

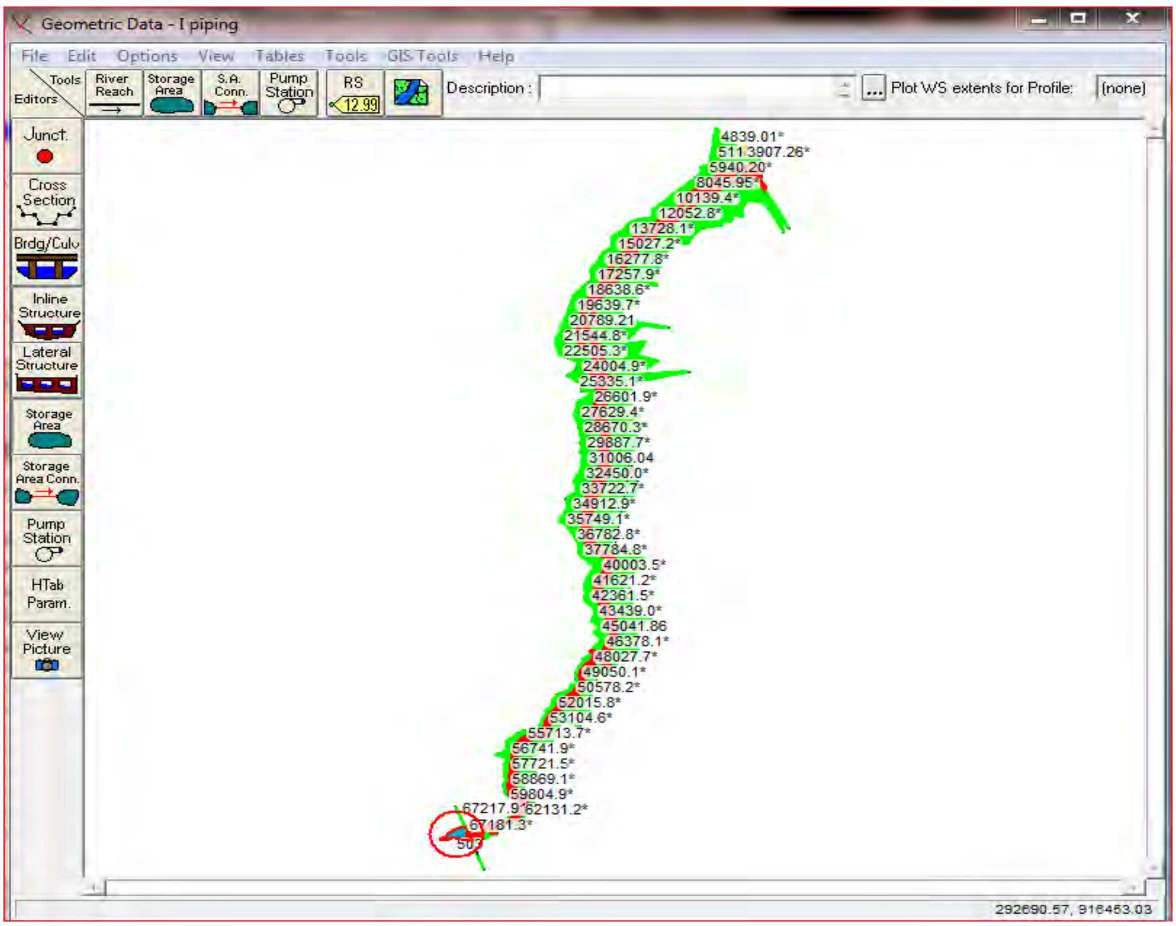

Figure 3-4: Gilgel Gibe River Geometry developed by HEC RAS

The stability of the HEC-RAS model is function of the distance between the cross sections and the time step used in the simulation. To minimize the error in computation and get stable solution of analysis in HEC RAS interpolating new cross-sections with a shorter distance between them were done between the initial sections at locations where the HEC-RAS solution became unstable. Whereas the minimum computation interval time was 1 second and to get stable solution selecting appropriate time had been done.

3.3.1.2 Describing Reservoir Characteristics

Description of reservoir geometry was the first step in the analysis of dam breach. The original data taken from the dam owner was input in the HEC-RAS model. The collected data concerning the reservoir characteristics is Elevation -Volume curves. On HEC RAS the reservoir of Gibe is modeled as elevation vs. volume curve and inflow. The following diagram is the volume vs. elevation curves of the Gilgel Gibe dam which is determined by Ministry of Water, Irrigation and Energy.

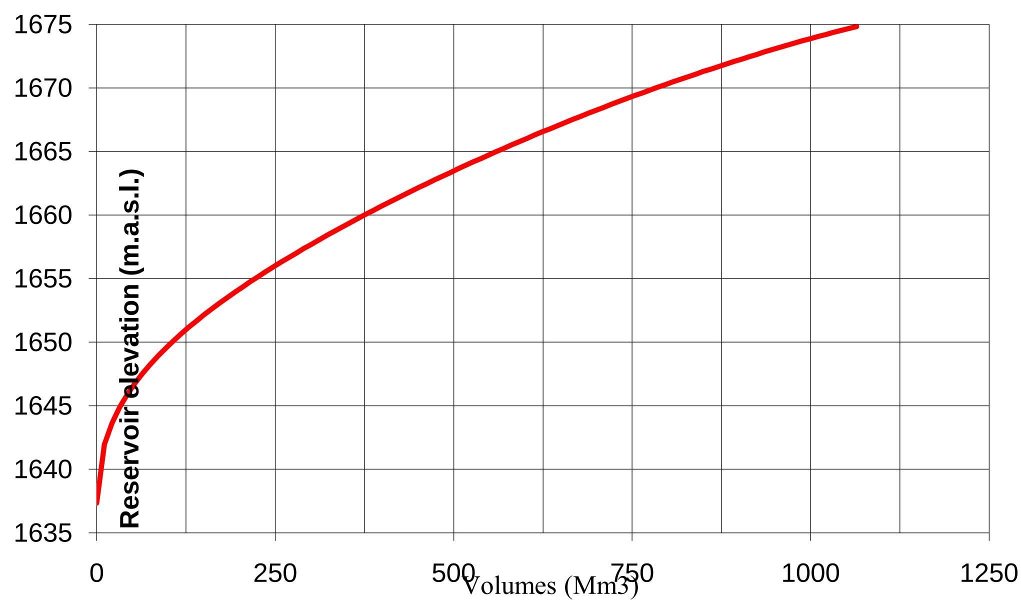

Figure 3-5: Volume versus Elevation Curve of Gilgel Gibe (MoWIE, 2004).

3.3.1.3 Identifying Physical Descriptions of Dam

This step included the identification of dam height, dam crest width, spillway elevation and width, weir flow coefficient and coefficient of discharge. In this study, data about the physical characterizes of the Gibe Dam were imported from the dam owner‘s dam inundation mapping and drawing used as input in HEC-RAS modeling.

Summary of dam characteristics description

- Crest Length: 1,704m

- Crest Width: 7m
- Maximum Height above river bed: 37m
- Average Upstream and Downstream Embankment slope: 2H:1V
- Type of dam : Rock fill
- Storage Capacity: 1,000 $*10^6$ m^3

3.3.1.4 Determining Inflow Hydrograph to the Reservoir

This step involved deriving an inflow design flood from a probable maximum precipitation. The inflow design flood is expected to cause the dam to breach in order to analyze the worst case of dam breach analysis. The inflow PMF which is expected to cause the dam to breach (i.e overtopping or piping) needs to analyze the worst case of dam breach analysis. Inflow hydrographs generated from two days 0.5 PMF was used to design the spillway and this hydrograph was used for the breach analysis. Inflow hydrograph which is used in HEC RAS as input was used in the analysis of Gilgel Gibe dam which is shown as follows.

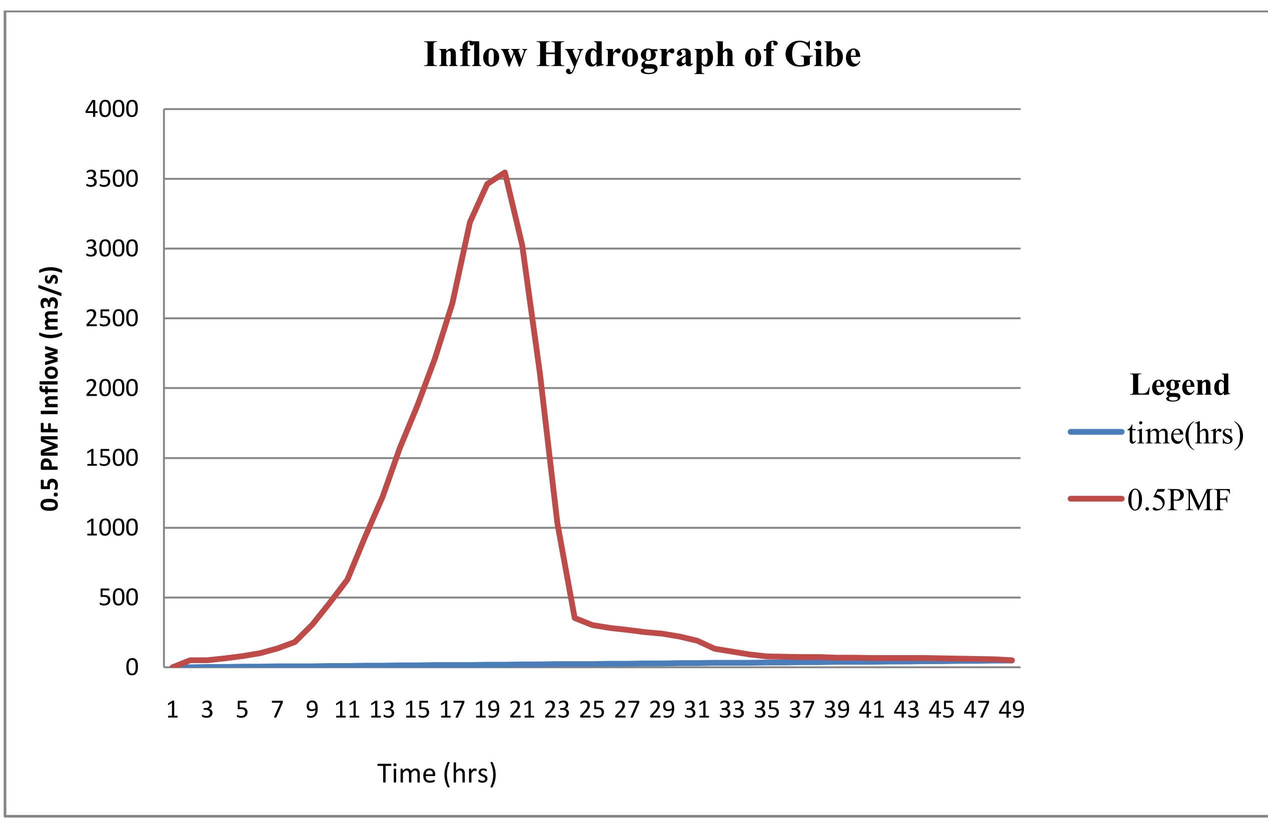

Figure 3-6: Inflow Hydrograph of Gilgel Gibe

3.3.1.5 Estimating Dam Breach Characteristics

Two modes of failures were analyzed for the study of Gilgel Gibe Dam breach analysis. These are: - overtopping failure and piping failure scenarios. Dam breach parameters were estimated using four regression (empirical) equations. These are: - MacDonald & Langridge Monopolis (1984), Van Thun and Gillette (1990), Froehlich (2008) and Froehlich (1995) are the most commonly applicable regression equations in dam breach parameter estimation process.

3.3.1.6 Breach Outflow and Verification

The maximum breach outflow that will be obtained from the analysis should be checked for its reasonableness. Literatures recommended that one can check the reasonableness of the maximum breach outflow obtained through selected method with model.

3.3.2 Routing Breach Outflow Hydrographs through Downstream Reaches

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

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

3.3.2.1 Defining Channel Geometry and Boundary Conditions

During modeling of the downstream channel of the Gibe Dam using HEC-RAS, the crucial step was to establish the external boundary conditions. The Upstream and downstream boundary were selected at locations that were independent of flow conditions below the boundary. The last downstream cross section was set at a reasonable distance and a normal depth was chosen to define the downstream boundary conditions. Upstream boundary condition was inflow hydrograph. A normal depth was used for the downstream boundary condition. For this study the normal depth option can only be used as downstream boundary condition for an open ended reach.

For the purposes of this study, default values of expansion and contraction coefficients were used throughout the unsteady state analysis. The program by default assigns a value of 0.3 0.1 and 2.6 for expansion, contraction and breach weir coefficient, respectively. This decision was made due to geometric similarities among cross sections and manning‘s roughness coefficients provided by the dam owner.

3.3.2.2 Roughness Coefficient Values

Manning‘s coefficient n is used to describe the resistance to flow due to channel roughness caused by sand/gravel bed-forms, bank vegetation and obstructions, bend effects, and circulation-eddy losses. The manning‘s coefficient n values provided by the dam owner for the channel reaches between specified cross-sections were used as a reference basis of the dam breach analysis.

In unsteady state river routing simulation, results were often very sensitive to the manning n values. Selection of the manning n was aimed to reflect the influence of bank and bed materials, channel obstructions, irregularity of the river banks, especially vegetation, and to minimize potential biasness of the results. According to Chow (1959) , the Gilgel Gibe river bed and bank materials and manning n values 0.030 and 0.035 are taken for the banks and flow channel respectively.

3.3.2.3 Dam-Break Outflow Hydrographs

At four different chainage over downstream section of a dam four outflow hydrographs were used as inflow hydrographs for the downstream river routing.

3.4 Flood Hydrograph Routing

Flood routing is an analytical procedure intended to trace the flow of water through a hydrological system, pond, conveyance or porous media, given some runoff even hydrograph as input. Flood routing can predict the temporal and spatial variations of a flood wave through a river reach and/or reservoir. The hydrologic routing and hydraulic routing were used to conduct flood routing through the reservoir and downstream channel. As flood waves travel downstream they are attenuated and delayed. That is, the peak flow of the hydrograph decreases and the time base of the hydrograph increases. The shape of the outflow hydrograph depends upon the channel geometry and roughness, bed slope, length of channel reach, and initial and boundary flow conditions.

The propagation of flood waves in a channel is a gradually varied unsteady flow process, which is governed by conservation of mass and momentum equations. The solution of these equations in a distributed manner is referred to as distributed routing of flood waves. Routing by distributed system methods is called hydraulic routing and the flow is calculated as a function of both space and time. When no spatial variability is taken into account and when the channel reach or reservoir is considered as a black box, the corresponding routing procedure is referred to as lumped routing. Routing by lumped system methods is called hydrological routing. These methods calculate the flow as function of time alone.

3.4.1 Hydrologic Routing

The hydrologic routing involved the balancing of inflow, outflow, and storage-discharge relation through use of the continuity equation. This application of hydrologic routing was used for reservoir routing. The reservoir component initiated by receiving upstream inflows and routed these inflows through a reservoir using Storage Routing Methods. The shape of breach, IDF, average width of breach, side slope of breach, breaching time, and other dam breach parameters were identified and predefined for generating the breaching outflow hydrographs by HEC RAS computer software.

3.4.2 Hydraulic Routing

Dam-breach outflow hydrographs were used as inputs to the river routing through the immediate downstream reaches of dam site. By the very nature, dam-break outflow hydrographs are highly unsteady flows that require a full unsteady flow routing method. In order to fully define an unsteady hydrograph, St. Venant equations should be used to analyze the routing flood wave propagation. Thus, the HEC-RAS hydraulic routing subroutine was adopted to route the dam-break outflow hydrographs though the Gibe Channel.

In order to carry out the hydraulic routing through the Gibe, the downstream cross sections data were imported from the dam owner study that included all cross sections geometries, Manning coefficients, reach lengths and boundary conditions. The boundary conditions included all of the external boundaries of the system, as well as the internal locations and set the initial flow and storage area conditions at the beginning of the simulation. Since the Gibe downstream channel was modeled as an open-ended reach, the downstream boundary condition was set as a normal depth. As recommended in HEC-RAS user manual for this option of boundary condition, the last cross section was placed far enough such that any errors it produced would not affect the results at the study reach.

3.5 Hydraulic Model Development

A river hydraulics model is only going to be as good as the data and personnel used to develop it. Detailed terrain information for the main channel and overbank floodplain areas are the principal data required for creating a river hydraulics model. Land use data (used for estimating Manning‘s roughness coefficients) and hydraulic structure information (inline structures) are also essential to develop a complete river hydraulics model. For modeling dam failures, further information describing the failure mode, breach size, and breach timing are necessary. This section of the paper will discuss data considerations for developing a river hydraulics model to perform dam failure analysis.

3.5.1 HEC-RAS Development

The United States Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) developed the HEC-RAS Hydraulic channel flow model as part of their suite of hydrologic and hydraulic modeling tools (Brunner, 2010). While primarily used as a flow routing model, a dam breach module has been added to the model to simulate the breach process. HEC-RAS can simulate steady or unsteady one-dimensional flow by solving the full one dimensional Saint-Venant equations. Also subcritical, supercritical, or mixed flow regimes can be simulated.

Initial model development may be performed using HEC-GeoRAS and using an HEC-RAS option to import the GIS data. At a minimum, the data import should establish the river/reach schematic and the description of cross sections. The river hydraulics model will need additional cross section information, hydraulic structures data, flow data, and boundary conditions prior to simulation. This section will focus on just a few of the more important data considerations.

3.5.2 HEC-GeoRAS Development and Project Setup

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, 2005). It consists of a set of tools and procedures for extracting primarily topographic data within the GIS platform and importing directly into a HEC RAS geometric file. HEC-GeoRAS allows for a quick and seamless conversion of electronic contour data into a cross section format used by HEC-RAS to facilitate the calculation of water surface elevations. Cross sections, stream centerlines, and other geometric features of the stream were extracted from GIS using HEC-GeoRAS and ArcGIS. GeoRAS assists engineers in creating datasets (referred to collectively as RAS Layers) in ArcGIS to extract information essential for hydraulic modeling. The latest release of HEC-GeoRAS supports the extraction of elevation data from DEM in either the TIN or grid format. GeoRAS requires that the user have a DEM. The DEM must be projected into a coordinate system. The coordinate system of the DEM is used as the basis for developing each of the RAS Layers. GeoRAS also requires that the Stream Centerline layer and Cross-Sectional Cut Line layer be created. The development of all other RAS Layers is optional based on the data needs for the river hydraulics model.

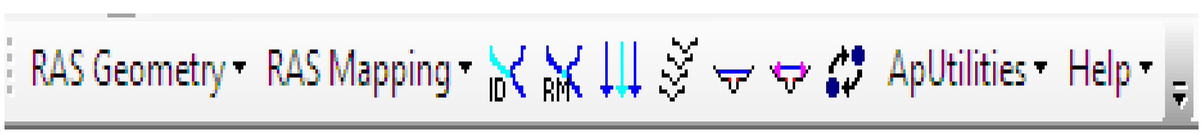

Figure 3-7: HEC-GeoRAS Tool Bar Used in ArGIS 10.1

Once the RAS Layers have been created, GeoRAS tools and menus are available to assign and populate attribute data. Lastly, the data are written out to the HEC-RAS geospatial data exchange format and can be imported into HEC-RAS.

The Stream Centreline 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, bridges, 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 centrelines were created before laying out cut line locations.

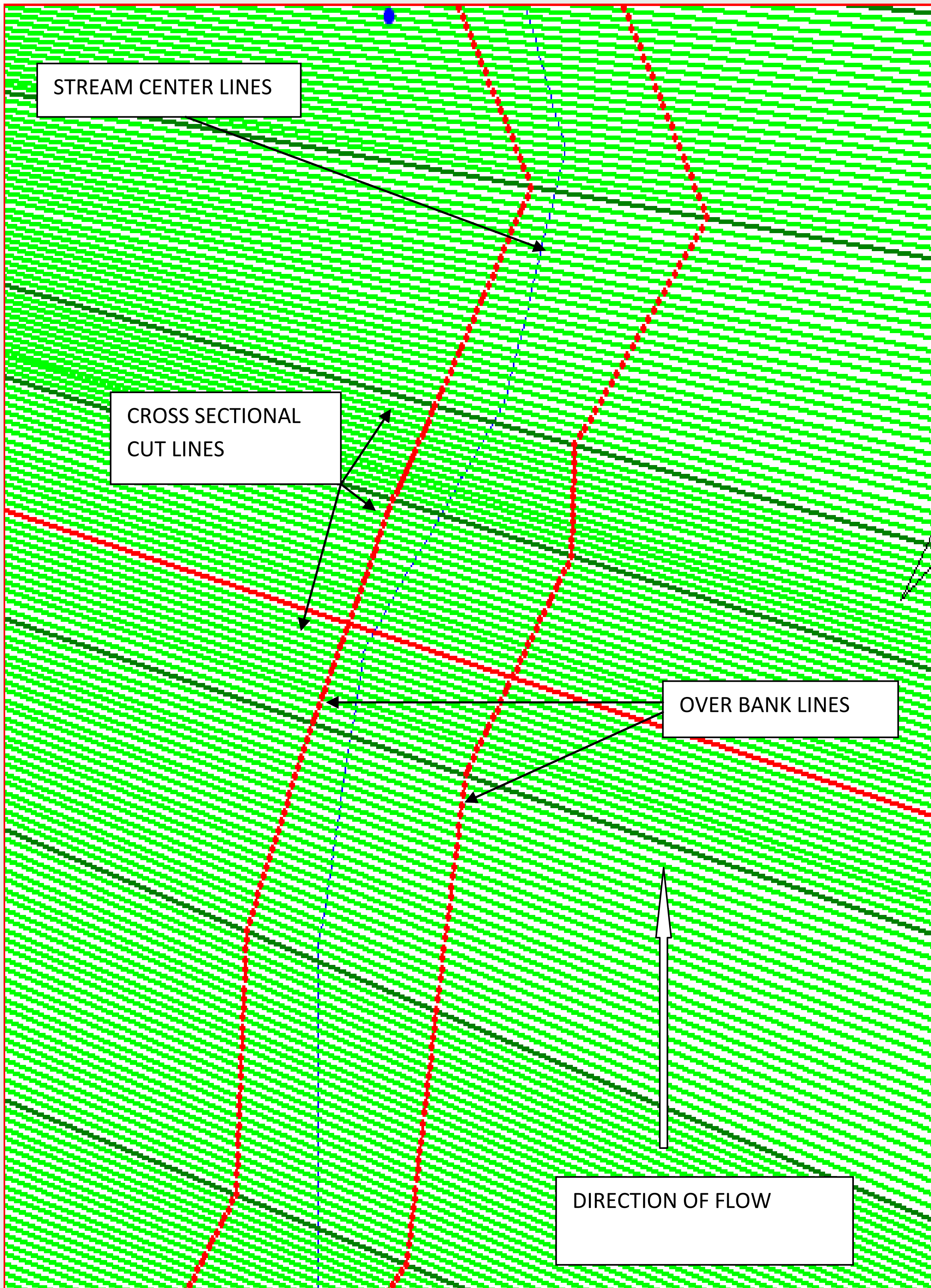
STREAM CENTER LINES
CROSS SECTIONAL CUT LINES
OVER BANK LINES
DIRECTION OF FLOW

Figure 3-8: Gilgel Gibe River Geometry developed by HEC-GeoRAS tool in ArcGIS 10. 1

HEC-GeoRAS extracted Cross sections, stream centerlines, and other geometric features of the stream were from GIS using HEC-GeoRAS and ArcGIS. In this study area the extract primarily topographic data and the catchment were delineated. The catchment is projected and treated by HEC-GeoRAS.

3.6 Floodplain Mapping

The aerial extent of flooding downstream area for Gilgel Gibe dam breach analysis was analyzed. For this study, two failure mode scenarios were used; i.e. overtopping and piping modes of failure. The reservoir conditions, maximum water level (MWL) and normal pool (spillway crest) for piping failure and maximum storage elevation (top of the dam crest) for overtopping failure were considered for downstream consequence of Gilgel Gibe Dam breach.

Once the HEC-RAS model was complete, output data was exported to GIS. HEC-GeoRAS was used to compile the data into useful graphical output such as floodplain polygon shape files.To generate floodplain shape files, the GeoRAS extension is used to first create a water surface TIN for each of the flood events. The water surface TIN is automatically clipped to fall within the bounds of the cross sections (i.e. it does not extend beyond the end points of any cross section), and is completely independent of the terrain TIN. After the water surface TIN is created, the rasterization of the water surface TIN and the terrain TIN takes place and the floodplain is delineated where the water surface exceeds the terrain elevations.

Because the resulting floodplain shape file is only as good as the quality of the TINs that are used to create it, some manual adjustment of the floodplain boundary is necessary for the final product. Also, there were areas where the floodplain extended beyond the extent of some of the cross sections. Because the water surface TIN is clipped at the end of the cross sections, manual extension of the floodplain was necessary. This process involved starting at a point within the water surface TIN bounds and tracing the floodplain boundary outside the TIN along a consistent contour elevation. This is continued until floodplain boundary returns within the bounds of the water surface TIN.

3.7 Emergency Action Plan

An emergency action plan is formal documents that identifies potential emergency conditions, safeguard lives and specifies preplanned actions to be followed to minimize property damage, environmental and social impact and loss of life at event when Gilgel Gibe dam fail. Emergency Action Plans (EAPs) must be developed for all projects that could pose a threat to life. In general, the EAP should describe procedures for responding to unusual or emergency situations and procedures for initiation of notification or warning of individuals who may be at risk in downstream areas. As a minimum, the EAP should include the following:

- Notification procedures (preferably in the form of a flow chart) and responsibilities for notifying downstream residents in the event of an impending dam failure.
- A notification list that includes the names and telephone numbers of all affected downstream residents, dam owner and operator, local emergency officials, and appropriate government agencies (including the Dam Safety Office).
- Specific instructions for responsible parties to be followed at the dam site in response to emergencies such as floods, equipment failures, or other unusual events where the situation is evolving slow enough that immediate remedial action can be effective to prevent failure.
- Procedures to follow for emergency situations which probably would not lead to dam failure, but still could represent a hazard for downstream residents.
- Inundation maps to identifying critical infrastructure and population-at-risk sites that may require protective measures and warning and evacuation planning.
- Clearly delineating the responsibilities of all those involved in managing the incident and how those responsibilities should be coordinated.

To carry out this mission, the EAP contains:

- Procedures to monitor Gilgel Gibe dam periodically and during flood warnings issued by the National Weather Service.
- Notify Emergency Operation Center of a potential dam failure.
- Warn and evacuate the isolated residences at risk. These procedures are to supplement and be used in conjunction with County‘s Emergency Operation Plan.

Failure of the Gibe dam could cause significant damage to (all roads and isolated residences downstream of the dam within the danger reach) located downstream of the dam.

3.7.1 Operating Procedure

The dam will be inspected periodically each year for maintenance and distress signals. The dam observer will inspect the dam when the National Weather Service issues a Flood Warning for the area and complete the following tasks.

- The dam observer will note & 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 ominous distress signs, the dam observer will call the Emergency Operation Center to send out police to roadblock downstream roads and warn any isolated residences in the danger reach. The dam observer may contact the Dam Safety Division or his designated engineer to provide assistance.
- If the pool level rises too within one meter of the dam crest, the dam observer will contact the County Emergency Operations Center to dispatch police to roadblock downstream roads and warn any isolated residences in the danger reach.

3.7.2 Preventative Actions

The following points are potential emergency actions which may prevent or delay the failure during occurrence of dam failure. They should be considered based on site-specific conditions, as well as the risk of failure (i.e. dam and d/s area) and risk to environment. If overtopping appears imminent, the following actions should be taken:

- Notify local authorities and state dam safety officials about possible failure.
- Be sure that the spillways are not plugged with debris and are functioning as efficiently as possible and Increasing the spillway capacity and open the drain gates,
- Open all drains or other gates to lower the pool level.
- Rising the dam crest(free board) by placing sand bag along the crest of dam ,
- Place riprap or sandbags in damaged areas of dam.
- Diverting the flood to less sensitive areas or emergency spillway,

- Implementing local flood mitigation measures in the watershed or
- Changing the reservoir regulation strategies.

If piping has developed through the Embankment, Foundation, or Abutments the following actions should be taken:

- Notify local authorities and state dam safety officials about possible failure.
- Plug the seepage with appropriate material such as (riprap, hay bales, bentonite, sandbags, soil, or plastic sheeting if the leak is on upstream face of dam).
- 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.

3.7.3 Supplies and Resources

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

3.8 Remedial Measure

Mostly it is difficult and expensive to mitigate dam breach after completion of construction work. But to reduce the risk of failure one must go to mechanism mitigation to reduce the failure mode. These failure modes are addressed with it remedy measure or controlling manner under this work. For both overtopping and piping failure modes remedial measures are considered as much as possible.

4 RESULTS AND DISCUSSION

4.1 Dam Breach Parameters Result

Simulation of dam breach analysis is essential to characterize and identify hazards happen on the downstream due to hypothetical dam failures. Different hydraulic models are used to analyze dam breach of those Hydraulic models HEC-RAS are often used for the analysis of downstream impacts resulting from potential dam failures of Gibe. The important point in dam break analysis is the estimation of breach location, width of breach, breach formation time and peak discharge pass through the beach are estimated accurate out hydrograph and the downstream inundations area. Estimations of the dam breach parameter, such as formation time, width and side slopes, have usually done external to the hydraulic model.

4.1.1 For Overtopping Failure Case

For overtopping case the failure location is assumed to be at main channel centerline (at half of the crest length of dam 852 meters). From the calculation of breach parameters using these four methods used in this study, the following results were obtained.

Using Macdonald and Langridge - Monopolis (1984), the breach bottom width and breach development time results were W_b = 362.43 m and tf = 2.85 hr. using Von Thun and Gillette (1990), these values were W_b = 110.4 m and t_f = 0.99 hr. Using Froehlich (1995), these values were W_b = 222.08 m and t_f = 2.86 hrs. Using Froehlich (2008), these values were W_b = 164.05 m and t_f = 2.46 hrs. According to the study undertaken by Wahl (2004 & 1998), the first equation (Maconald and langridge Monopolis-1984) was over-predicting the breach bottom width and the breach bottom width for the second equation (Von Thun and Gilette-1990) was less than the three and is under estimate. The result from the third equation (Froehlich-1995) was fall between the two equations (Macdonald 1984 and Froehlich 2008). The result from the four equations (Froehlich-2008) was less than the third equation but greater than second equation. The trend of these parametric equations to over-predict and under predict the breach size may be attributed to the fact that they are developed based on the assumption of breach forms. The review by Wahl (2004) found that the best methods of breach width and breach time prediction(Reclamation 1988; Von Thun & Gillete 1990;Freohlich 1995) had uncertainties of about 1/3(plus or minus) Order of magnitude and the best prediction of breach time (Froehlich 1995) had uncertainties of about 2/3 (plus or minus) order of magnitude. Froehlich (1995) method was selected based on the results obtained by (Wahl, 2004) which showed that this method is more accurate than other existing prediction methods. Having the above study in consideration the critical values of breach parameters estimated using Froehlich (1995) to be W_b = 222.08 m and t_f = 2.86 hrs was selected. Table 4-1 shows estimated dam breach parameters for overtopping failure.

Table 4-1: Estimated Dam breach parameters based on PMF event

Equation Name	MacDonald & LM (1984)	Froehlich (2008)	Froehlich (1995a)	Von Thun and Gillette (1990
Estimated breach parameters for Overtopping failure				
Bottom breach width (m)	362.43	164.05	222.089	110.4
Discharge (m3/s)	49,021.36	114,447.87	16,310.41	23,651.6
Failure time (hr)	2.85	2.46	2.867	0.99
Side slope (Z:1)	0.5	1	1.4	0.5

4.1.2 For Piping Failure Case

Using Macdonald and Langridge Monopolis (1984), the breach bottom width and breach development time results were (362.88 m, 2.86 hrs) and (461.53 m, 2.49 hrs) at MWL and Spillway crest, respectively. Using Froehlich (1995), these values were (142.90 m, 2.83 hrs) and (134.76 m, 3.58 hrs) at MWL and Spillway crest, respectively. Using Von Thun and Gillette (1990), these values were (105.4 m, 0.95 hrs) and (85.4 m, 0.79 hrs) at MWL and Spillway crest, respectively. Using Froehlich (2008), these values were (111.76 m, 2.46 hrs) and (110.23 m, 3.18 hrs) at MWL and Spillway crest, respectively. According to the Tony Wahl (1998) technical article, Froehlich‘s (1995) equation for average breach width is the best predictor for the cases with observed breach height greater than 50 meters (i.e. 164 feet). Therefore, Froehlich‘s (1995) equations for both average breach width and for breach failure time were used to calculate the ―sunny-day" breach parameters. Therefore, the critical values of breach parameters estimated using Froehlich (1995) to be 142.90 m and 2.83 hrs were selected for MWL. Table 4-2 shows the summary of estimated breach parameters for piping (at MWL and Spillway crest) through four mentioned regression equations.

Table 4-2: Estimated Dam breach parameters based sunny day case.

Equation Name	MacDonald & LM (1984)	Froehlich (2008)	Froehlich (1995a)	Von Thun and Gillette (1990
Estimated breach parameters for piping failure				
Bottom breach width at MWL (m)	348.88	111.76	142.90	105.4
Bottom breach width at Spillway crest (m)	461.53	110.23	134.76	85.4
Estimated Discharge at MWL (m3/s)	45,713.53	70,496.035	14,719.1	17,009.2
Estimated Discharge at Spillway (m3/s)	41,216.63	47,007.34	10,669.1	14,923.01
Failure time at MWL (hr)	2.71	2.46	2.83	0.95
Failure time at Spillway crest (hr)	2.49	3.18	3.58	0.79
Side slope (Z:1) for	0.5	0.7	0.9	0.5

From here, all two set of parameters (bottom breach width and time failure) should be entered into HEC-RAS software and run as separate breach plans. This will result in twelve different breach outflow hydrographs which consists three computers run for each four regression methods. However, once the hydrographs are routed downstream, they will begin to converge towards each other. For our case using Probable Maximum Flood (PMF) conditions analyzed overtopping and piping Failures mode.

As Wahl(2004) evaluated , MacDonald-Langridge-Monopolis(1984) , Froehlich (1995) , Froehlich (2008) and Von Thun & Gillette(1990) methods are used in estimation of the maximum breach outflow but Froehlich (1995) equation was used to estimate the peak flows discharge of study because this method significantly estimate fair peak flows discharge than that produced by an instantaneous failure to the ultimate breach geometry. The discharges calculated through MacDonald & Langridge- Monopolis , Froehlich (1995) , Froehlich (2008) and Von Thun& Gillette(1990) equations were (49,173.53 m^3/s, 41,216.63 m^3/s) , (14,719.1 m^3/s, 10,669.1 m^3/s), (70,496.035 m^3/s, 47,007.34 m^3/s) and (17,009.2 m^3/s, 14,923.01 m^3/s) for

MWL and spillway crest of piping, respectively. The value obtained by using Froehlich (1995) as show above will be used as an upper peak breach outflow.

Breach bottom width and breach development time are governing parameters in determine the peak and shape of the outflow hydrograph happen in dam breach. The study of dam breach analysis and remedial measure of Gibe Dam mostly involves in examining a number of dam breach parameters which is used as input parameters in HEC RAS model and help in sensitivity analysis with outflow hydrograph routing. Those parameters examined in HEC RAS defined for the reservoir, river and inline component of the analysis were prepared based on existing data and manual empirical formulas. 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 dam breach analysis findings were discussed in the following sections.

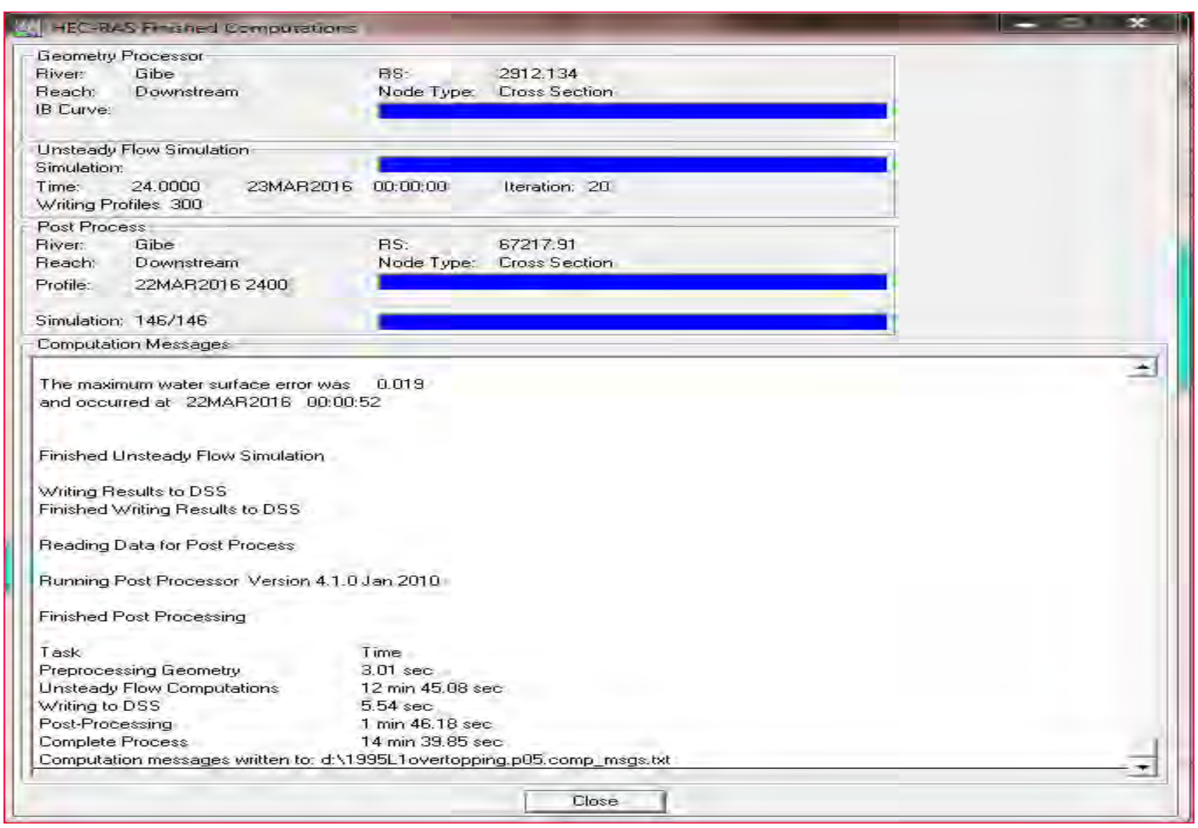

Figure 4-1: HEC-RAS Unsteady Flow Simulation Computer Run for overtopping mode

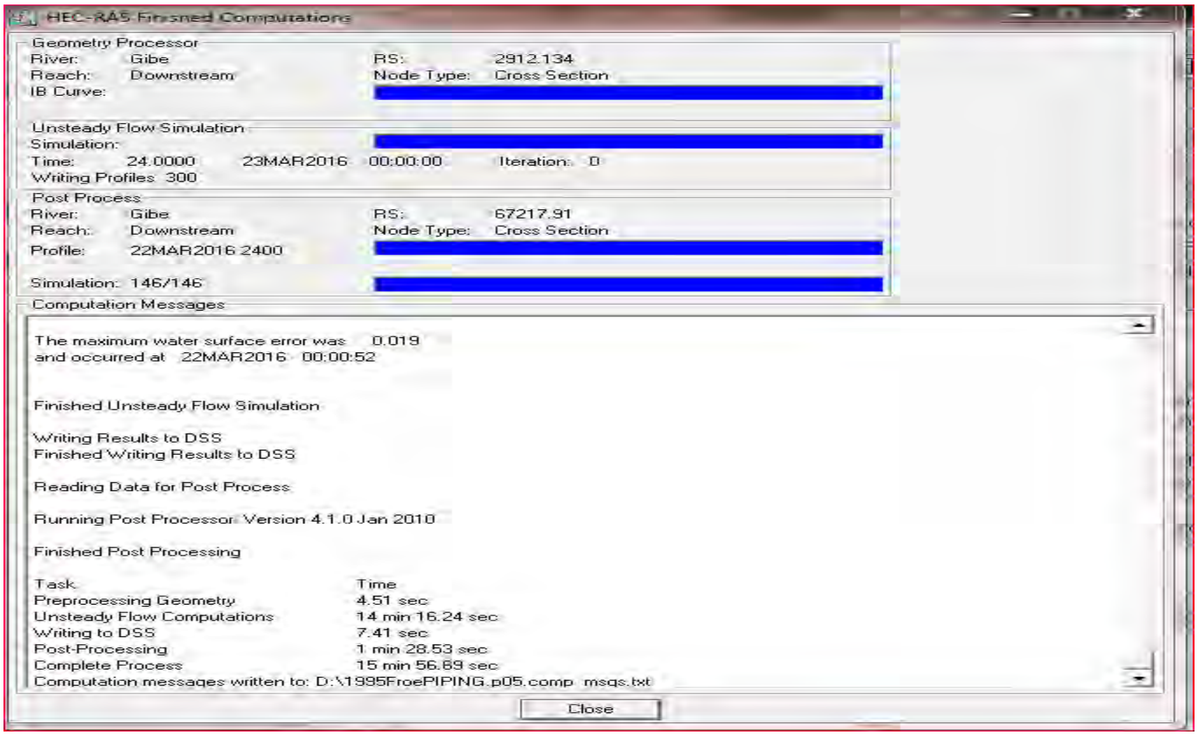

Figure 4-2: HEC-RAS Unsteady Flow Simulation Computer Run for piping mode

The Gilgel Gibe dam breach flood is routed downstream in order to carry out the inundation map analysis and is shown in the figure below.

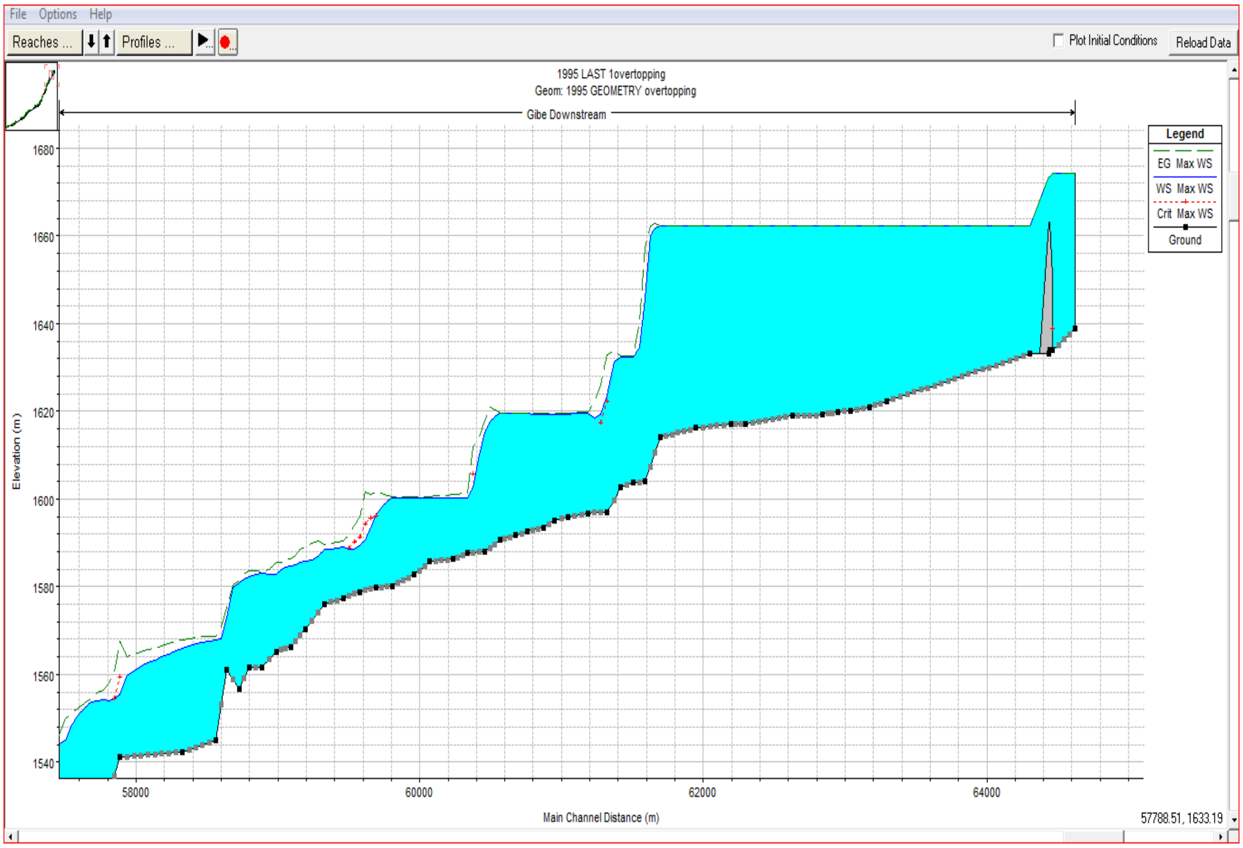

Figure 4-3: Gilgel Gibe Reach after the simulation of computer program runs for overtopping failure mode

4.2 Dam Breach Simulation Result for Overtopping Mode of Failure

In this study two scenarios are analysis in different hydrological events these are the overtopping and piping mode which is related to the PMF events and sunny day at normal pool levels respectively.

The sensitivity analyses are grouped into two. These are local sensitivity analysis which includes Maximum breach discharge, breach bottom width, breach development time and side slope of breach at inline structure with its reservoir component and where as global sensitivity analysis includes manning roughness and channel bed slope at downstream reach. The overtopping mode of hydrological event was analysis and resulted as follows.

The bottom breach width, breach formation time, breach side slope and peak out flow discharge of the study are calculated by Froehlich (1995) method are 222.089 m, 2.867 hrs, 1.4 and 16,310.41 m^3/s for overtopping failure mode. These calculated values are inserted in to HEC RAS as input to generate the flood hydrograph, peak out flow discharge and time to peak.

The dam breach outflow hydrograph for the Gilgel Gibe dam resulted from the HEC-RAS dam break modeling is shown in the figure below. Gilgel Gibe dam overtopping failure has been happened at an elevation of 1673.57 m above the crest of the dam with outflow 10,938.43 m^3/s at 18 hrs. Overtopping dam failure has been 57 cm above the top crest elevation of the dam (1673 m) this leads to the overflow of 10,938.43 m^3/sec discharge as HEC RAS model output result. The maximum breach outflow from the hydrograph is 10,938.43 m^3/sec which is less than the upper bound limit of breach outflow value calculated using Froehlich (1995) (16,310.41 m^3/sec). The stage and flow hydrographs of Gilgel Gibe more or less looks like the figure below which is mostly the description of the breach analysis at all which verifies the reasonableness of the outflow hydrograph obtained from the HEC RAS model.

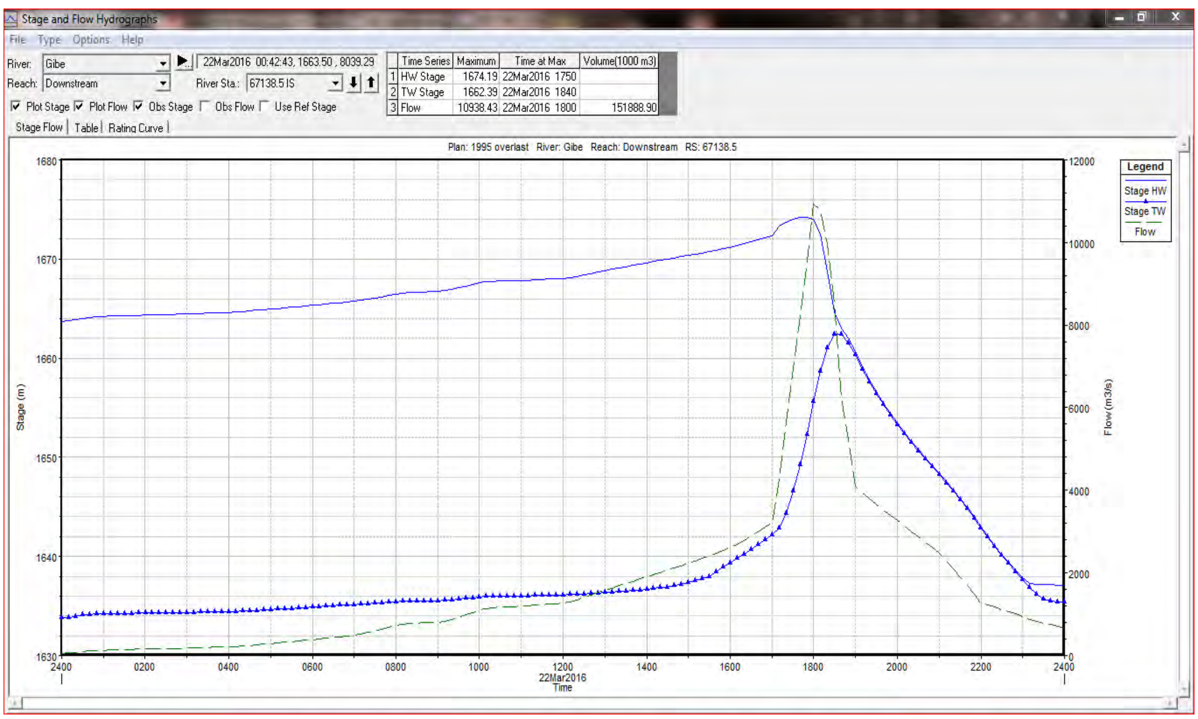

Figure 4-4: Breach Outflow Hydrograph at the Dam during overtopping

4.2.1 Routing of Flood Hydrograph at Different Chainage for Overtopping Failure.

For overtopping flood hydrograph routing has been analyze at five chainage points, at dam site, At 15 km downstream, at 25 km downstream , at 45 km downstream and at 64 km downstream to show the effect of flood under overtopping at downstream of dam. At the dam site,15 km,25 km ,45 km and 64 km the peak discharge and water surface elevation become (10,938.57 m3/s, 1673.57m) ,(6,923.22 m3/s, 1,326.16 m),(6,597.90 m3/s, 1,171.12 m),(5,511.19 m3/s, 989.75 m) respectively.

4.3 Dam Breach Simulation Result for Piping Mode of Failure

The second point in study was to analyze the piping failure mode of Gilgel Gibe and to inundate the downstream banks of the river as the impacts has been seen. For piping failure mode, the average breach width, breach formation time, breach side slope and peak out flow discharge calculated by Dr. Froehlich (1995) were 142.91 m, 2.84 hrs, 0.9 and 14,719.097 m^3/s at time of failure.

The dam breach outflow hydrograph for the Gibe dam resulted from the HEC-RAS dam break modeling is shown in the figure below. Piping dam failure has leads to the outflow of 8,700.57 m^3/sec discharge as HEC RAS model output result. The maximum breach outflow from the hydrograph is 8,700.57 m^3/sec which is less than the upper bound limit of breach outflow value calculated using Froehlich(1995) (14,719.097 m^3/sec). When we compare the peak out flow of piping with that of overtopping, overtopping peak out flow of Gibe exceeds the piping one by 2,237.87 m^3/s. This indicates that the Overtopping failure is catastrophic than piping as the study result of HEC RAS model show. The stage and flow hydrographs of Gilgel Gibe for piping failure more or less looks like the figure below which is mostly the description of the breach analysis at all which verifies the reasonableness of the outflow hydrograph obtained from the HEC RAS model.

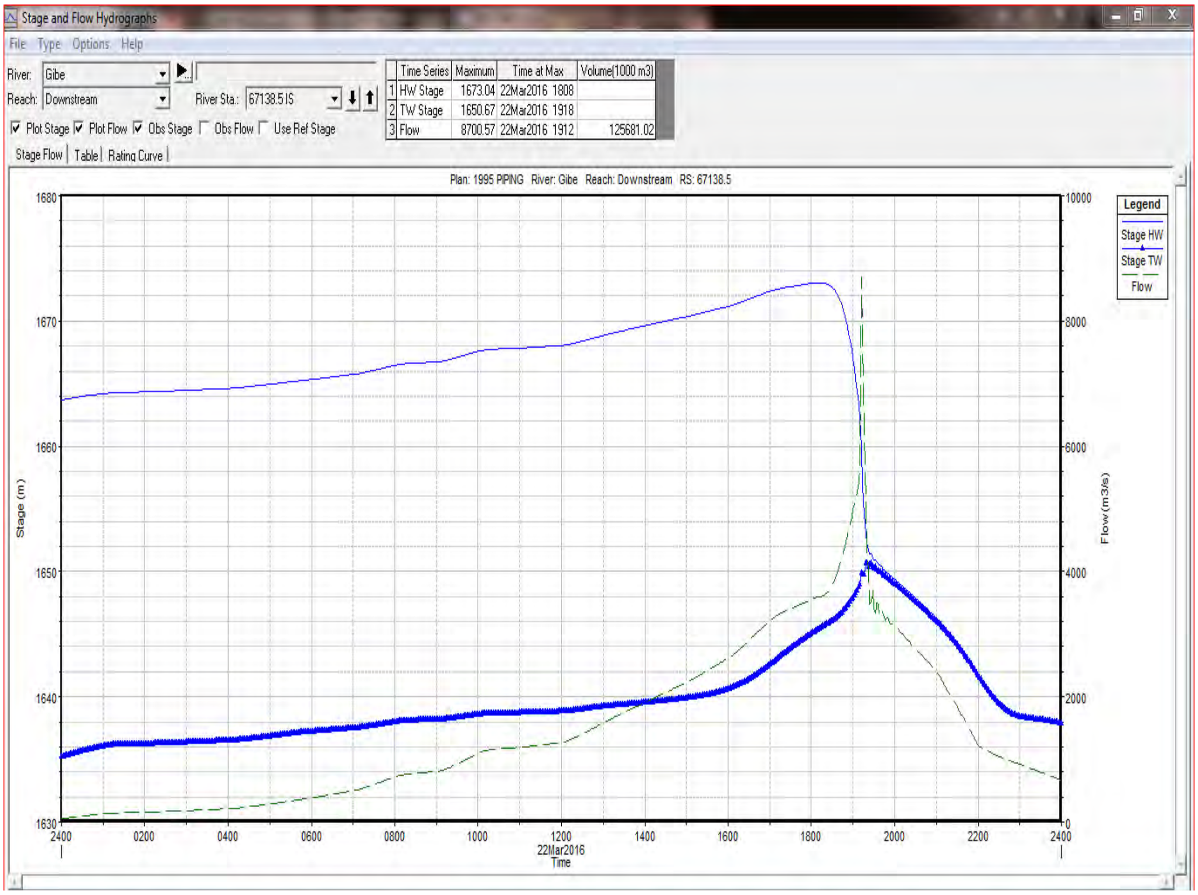

Figure 4-5: Breach Outflow Hydrograph at the Dam during piping failure mode

4.3.1 Routing of Flood Hydrograph at Different Chainage for Piping

In similar manner, like that of overtopping the flood routing has been under taken in HEC RAS model for the chainage : at dam site , 15 km from dam to downstream ,25 km from dam to downstream ,45km from dam to downstream and 64 km from dam to downstream. The peak discharge flow out At dam site, 15 km, 25 km,45 km and 64 km are (8700.57 m^3/s,1673.04 m), (5,984.63 m^3/s,1322.50 m), (4,810.813,879 m^3/s,1166.94 m) and (3,401.45 m^3/s,948.48 m) , respectively. This indicates that the water surface level and peak discharge decrease as distance from dam downstream increase. Which reduce the risk of loss of life, property and environment etc.

4.4 Sensitivity Analysis for Gilgel Gibe Dam Breach

For breach bottom width, breach time, breach side slope, manning coefficient and channel bed slope the sensitivity analysis for overtopping using Gibe Dam as a testing basis had been undertaken. Two sensitivity analyses were involved: global and local analysis. The global analysis involved changing both selected parameters parallel getting the output where as local analysis involved changing one parameters and having the remaining parameters constant obtain the output. Under the local sensitivity analysis, Maximum breach discharge (Q_{max}), breach bottom width (B_o), breach development time in hours (TFH), side slope of breach (1: Z_0) were parameters analyzed at the dam site. Whereas for global sensitivity, analysis Manning coefficients and channel bed slopes were considered over the downstream. These parameters defined for the reservoir and river component of the analysis were prepared based on existing data and some empirical formulas developed by Froehlich (1995).The findings were discussed in the following sub-sections.

4.4.1 Sensitivity of Time to Dam Failure (TFH) for a given Side Slope of Breach

Under this section the sensitivity between TFH and Q_{max} was analysis having the output of HEC RAS dam break sub routines. For this part the constant parameter was side slope of breach and changing parameters was time of breach development. The results of testing were summarized in Table 4-3, table 4-4 and appendix 3 for the detail outputs of HEC RAS dam-break subroutines.

Table 4-3: Maximum discharge and time to dam failure at dam site

Time of breach development(hrs)	Side slope of breach(1: Z_o)	Maximum discharge (m3/s)
0.125 T_o	1: Z_o	12,863.45
0.25 T_o	1: Z_o	11,552.50
0.5 T_o	1: Z_o	10,980.72
T_o	1: Z_o	10,938.43
1.1T_o	1: Z_o	8,675.50
1.39 T_o	1: Z_o	6,570.26
Note: Z_o and T_o represents typical values &equal to 1.4 & 2.867 hrs respectively		

Table 4-4: Percent change in max discharge and time to dam failure at dam site

Time of breach development(hrs)	Side slope of breach(1: Z_o)	Change in Maximum discharge (%)
0.125 T_o	1: Z_o	19.33
0.25 T_o	1: Z_o	7.17
0.5 T_o	1: Z_o	1.86
T_o	1: Z_o	0
1.1T_o	1: Z_o	-19.52
1.39 T_o	1: Z_o	-39.05
Note: Z_o and T_o represents typical values &equal to 1.4 & 2.867 hrs respectively		

For this local sensitivity analysis, six different TFH were used for HEC RAS dam-break computer runs and resulted in six dam breaching outflow hydrographs. The above Table 4-3 and 4-4 showed that the maximum breach discharge varied with TFH with a given constant side slope Z. Based on the evaluation 87.5 % reduction in TFH (T_o to 0.125T_o) resulted in 19.33 % increase in peak discharge at the dam site. Whereas, a 39 % increase (T_o to 1.39 T_o) in TFH resulted in 39.05 % reduction in peak discharge at the same site. Based on above results the peak discharge decreased when TFH increased. The rate of increase in peak flow relative to change in TFH was very drastic compared to the rate of decrease which was slightly steady as shown in the above table. This trend showed in the following figures.

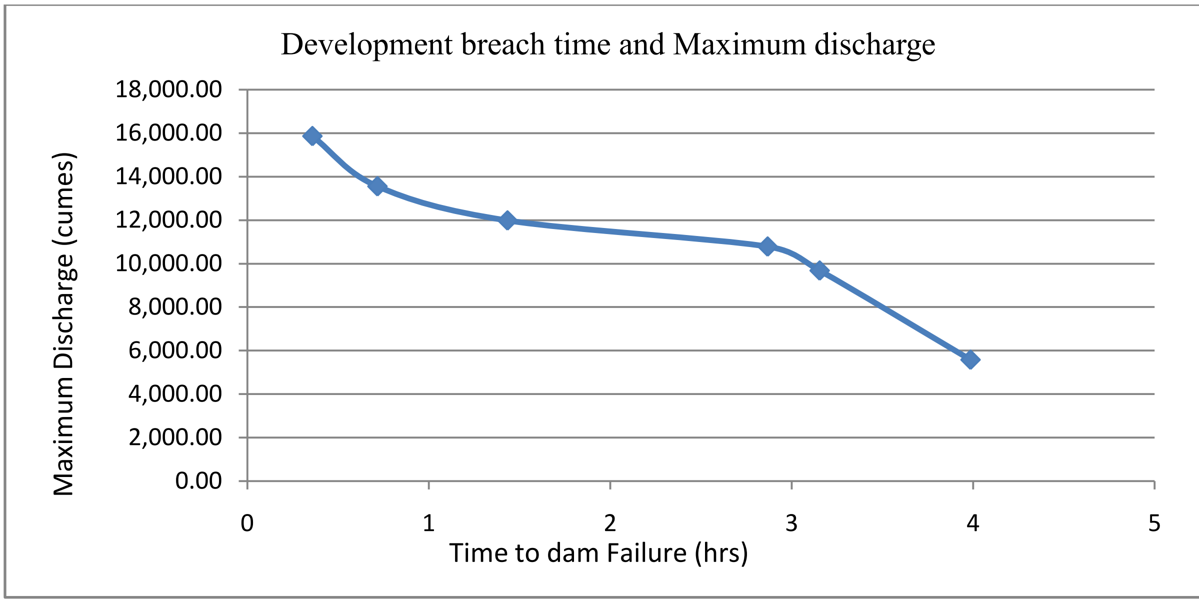

Figure 4-6: Time to dam failure and maximum discharge at dam site

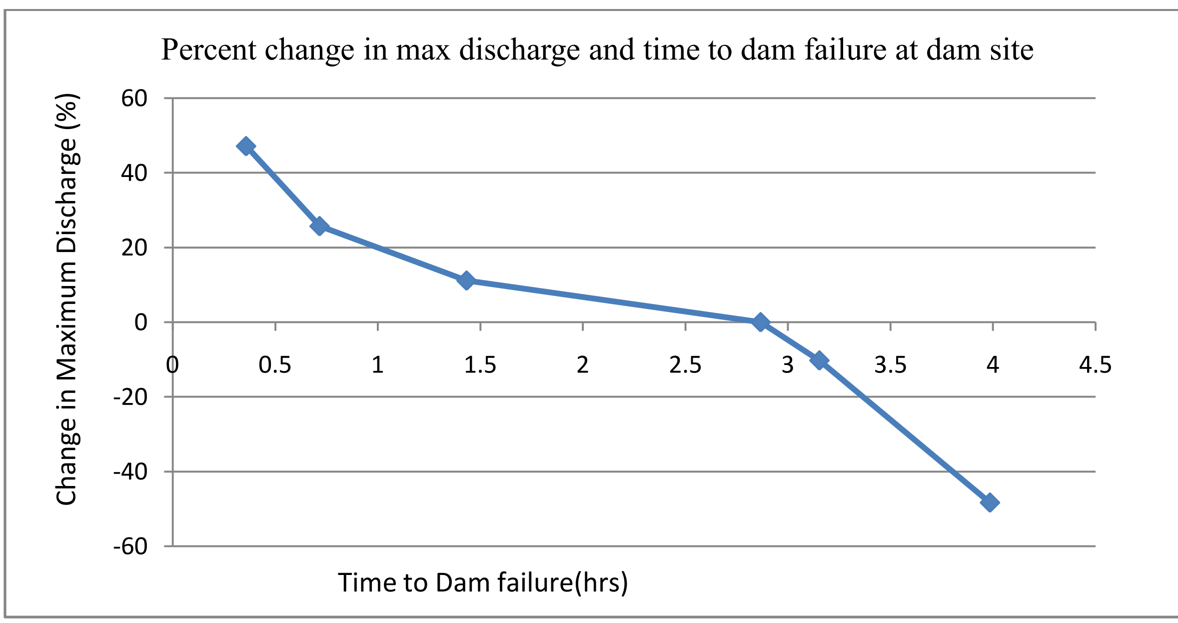

Figure 4-7: Percent change in max discharge and time to dam failure at dam site

In estimation of breaching hydrograph the empirical formula, historical data and a lot of personal experience were a multi-variable. Despite all of these, the above results confirmed that predicting dam breach outflow hydrographs was dependent on and sensitive to a minor change in dam breach development time.

4.4.2 Sensitivity of Side Slope of Breach (Zo) for a given Time to Dam Failure

For local sensitivity analysis of side slope of breaches vs. Maximum Breach Discharges, the constant parameter was time of breach development and changed parameter was side slope of breach. Table 4-5, table 4-6 & Appendix 3 presented the detail outputs of HEC-RAS reservoir routings.

Table 4-5: Side slope of breach and maximum discharge at dam site

Time of breach development(hrs)	Side slope of breach(1: Z_o)	Maximum discharge (m3/s)
T_o	0.125Z_o	14,742.45
T_o	0.5Z_o	12,758.21
T_o	Z_o	10,938.43
T_o	1.25Z_o	9,789.31
Note: Z_o and T_o represents typical values &equal to 1.4 & 2.867 hrs respectively		

Table 4-6: Percent change in max discharge and side slope of breach at dam site

Time of breach development(hrs)	Side slope of breach(1: Z_o)	Change in Maximum discharge (%)
T_o	0.125Z_o	36.76363768
T_o	0.5Z_o	18.35612194
T_o	Z_o	0
T_o	1.25Z_o	-10.11511537

Table 4-5 & 4-6 shown that as Z value increased, the maximum discharge at the dam decreased at a steady rate. From the above results, when side of slope of breach (Z_o to 0.125Z_o) decreased by 87.5% the maximum discharge at the dam increased by 36.76 % with relative to original data. Whereas, increased in SS (Z_o to 1.25Z_o) by 25% produced 10.11 % decrease in maximum discharge at the dam site. The rate of increase in peak flow for a given percent change in SS was higher compared to the rate of decrease as shown in the results. However, the increments of percent change in peak flow were very small compared to the corresponding percent changes in SS. Figure 4-8 and 4-9 were further illustrated the results at dam site.

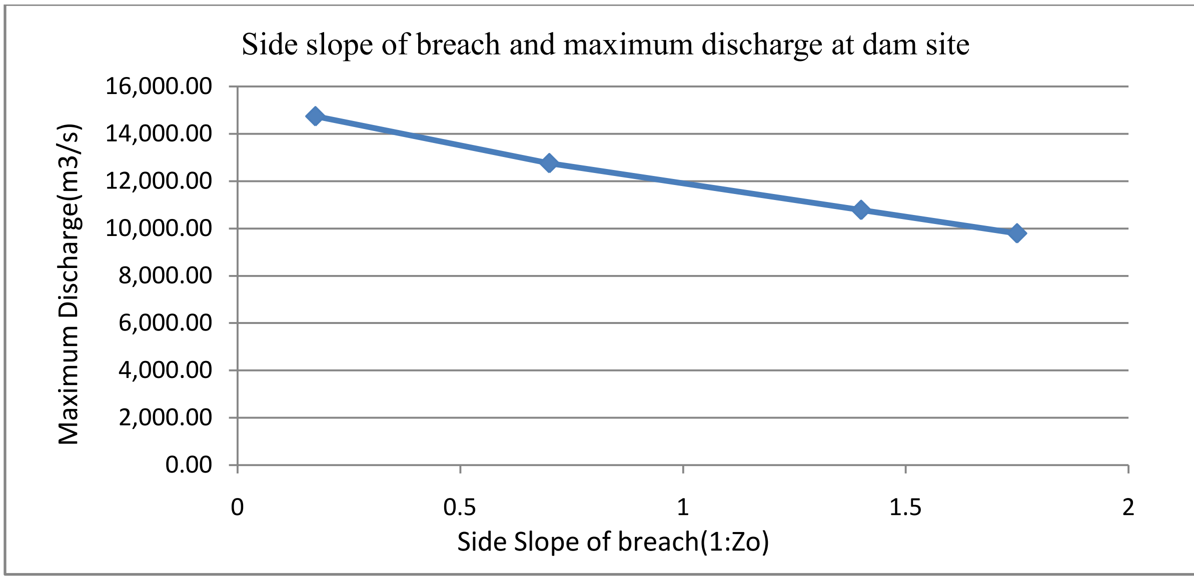

Figure 4-8: Side slope of breach and maximum discharge at dam site

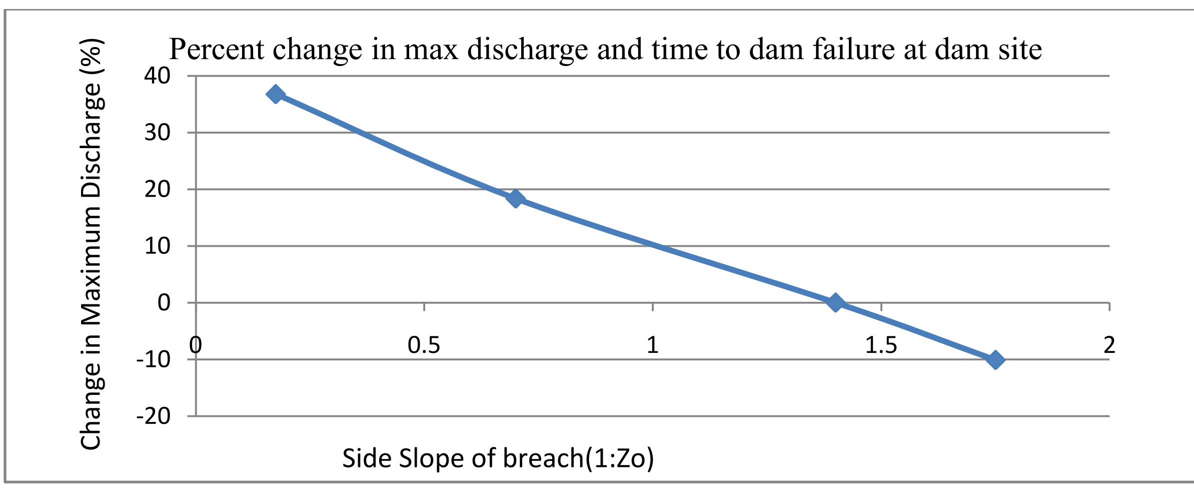

Figure 4-9: Percent change in max discharge and side slope of breach at dam site

When side slope of breach became steeper or decreased, the maximum discharge increased as it was shown on the above figures. Obviously when the Z values increases (breach slope becomes flatter), the area of breach increases. The wider conveyance area means decrease in velocity for a given flow. The rate of increase or decrease in conveyance area with respect to velocity determined the rate of outflow. However, the results of this sensitivity analysis indicated that the maximum discharge at the dam site decreased as the Z values increased under the condition, TFH was remained constant.

4.4.3 Sensitivity of Bottom Breach Width (Bb) for a given Time to Dam Failure

The bottom breach width vs. maximum breach discharges was the last local sensitivity analysis in this study. The analysis b/n Q_{max} and B_b were applied and result was summarized in the following tables (table 4-7 & 4-8). Appendix 3 presented the detail outputs of HEC-RAS reservoir routings.

Table 4-7: Bottom breach width and maximum discharge at dam site

Time of breach development(hrs)	Bottom breach width (m)	Maximum discharge (m3/s)
T_o	$0.125B_o$	10,572.94
T_o	$0.5B_o$	10,655.99
T_o	B_o	10,938.43
T_o	$1.25B_o$	10,951.81
T_o	$2B_o$	11,176.93
Note: B_o and T_o represents typical values &equal to 222.089 m & 2.867 hrs respectively		

Table 4-8: Percent change in max discharge and Bottom breach width at dam site

Time of breach development(hrs)	Bottom breach width (m)	Change in Maximum discharge (%)
T_o	$0.125B_o$	-1.917230772
T_o	$0.5B_o$	-1.146794736
T_o	B_o	0
T_o	$1.25B_o$	0.669783044
T_o	$2B_o$	3.685847633

From the above result, increase in B values lead to increase in maximum discharge at the dam. An 87.5% decrease in bottom breach width (B_o to $0.125B_o$) resulted in 1.92 % decrease in maximum discharge at the dam relative to original data. Whereas, a 100 % increase in B_o (B_o to $2B_o$) value produced 3.68 % increase in maximum discharge at the dam site. The rate of increase in peak flow for a given percent change in B_o was higher compared to the rate of decrease as shown in the above tables. The above observation was further illustrated in figure 4 -10 and 4 - 11 as follows.

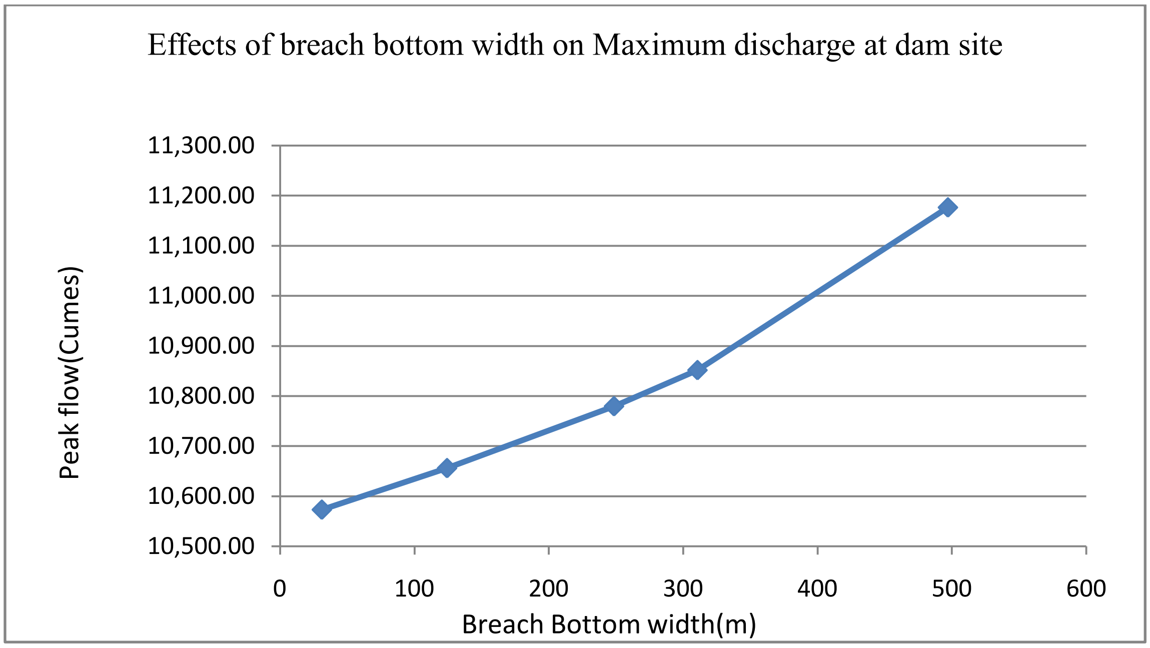

Figure 4-10: Bottom breach width and maximum discharge at dam site

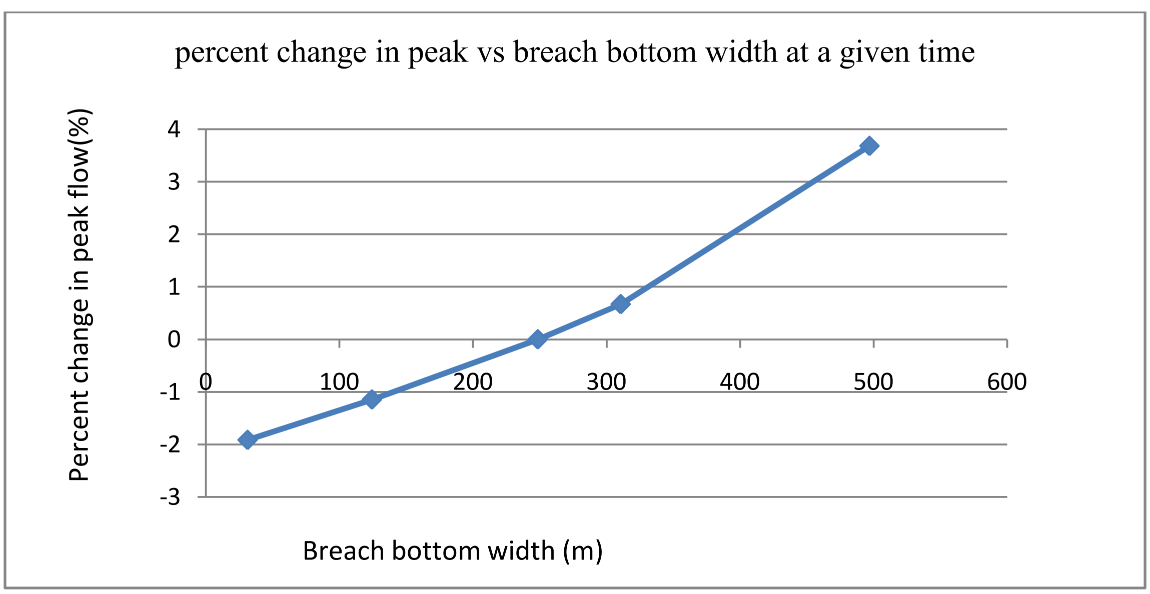

Figure 4-11: Percent change in Max discharge and bottom breach width at dam site

As the B value decreased or the Bottom breach width became narrow, the maximum discharge at the dam decreased. It was obvious that when the B_o value increases or breach bottom width becomes larger (width), the area of breach increases this leads release of huge water. The breach widens and deepens and results in increasing flood flow through the breach. However, the results of this sensitivity analysis indicated that the maximum discharge at the dam site decreased/increased as the Bo values decreased/increased under the condition, TFH was remained constant.

4.4.4 Sensitivity of Time to Dam Failure (TFH) and Side Slope of Breach (SS) Relatively

The global sensitivity analysis b/n TFH and SS over the Q_{max} were applied at the dam site. The summary results were tabulated in the following tables (i.e. table 4-9 & table 4-10). Appendix 3 presented detail outputs of HEC-RAS dam-break subroutines.

Table 4-9: Effects of time to dam failure and side slope on peak flows at dam

Time of breach development(hrs)	side slope of breach(1:Zo)			
	$0.125Z_o$	$0.5Z_o$	Z_o	$1.25Z_o$
	Peak flow(m3/s)			
$0.125\ T_o$	18,950.71	18,320.67	17,963.45	17,897.12
$0.5\ T_o$	17,389.33	17,218.00	17,080.72	16,914.83
T_o	10,742.45	10,758.21	10,938.43	10,789.31
$1.1T_o$	7,658.54	7,621.55	7,615.50	7,210.29
$1.39\ T_o$	2,562.37	2,465.61	2,270.26	1,837.03
Note: Z_o and T_o represents typical values &equal to 1.4 & 2.867 hrs respectively				

Table 4-10: Percent change in max discharge for a given time to dam failure and side slope of breach at dam site

Time of breach development(hrs)	side slope of breach(1:Zo)			
	$0.125Z_o$	$0.5Z_o$	Z_o	$1.25Z_o$
	Peak flow (%)			
$0.125\ T_o$	75.80307454	69.95828	66.64	66.02906811
$0.5\ T_o$	61.31837161	59.72897	58.46	56.91650177
T_o	-0.343800414	-0.1976	0	0.090913223
$1.1T_o$	-28.95280027	-29.296	-29.35	-33.11115255
$1.39\ T_o$	-76.22925346	-77.1269	-78.94	-82.95813075
Note: Z_o and T_o represents typical values &equal to 1.4 & 2.867 hrs respectively				

The above table 4-9 and 4-10 shows how the peak discharge at the dam site reacted to change in time to dam failure and side slope of breach. The tables show four conditions of side slope of breaches was combined with five times to dam failure and produced five maximum discharges at the dam site. This analysis was done four times for the four Z values as shown in the above tables.

According to the Results in above table that the peak discharge increased by 75.05 % when the time to dam failure reduced by 87.5 % whereas side slope of breach decreased by 87.5%. On the other hand, time to dam failure (T_o to $1.39T_o$) increased by 39 % where as 25% increase in side slope of breach (Z_o to $1.25Z_o$) resulted in 82.96 % reduction in maximum discharge at the dam site. Similarly, decrease in time to dam failure by 87.5 % and 25% increase in side slope of breach resulted increase in maximum discharge by 66.029 %.

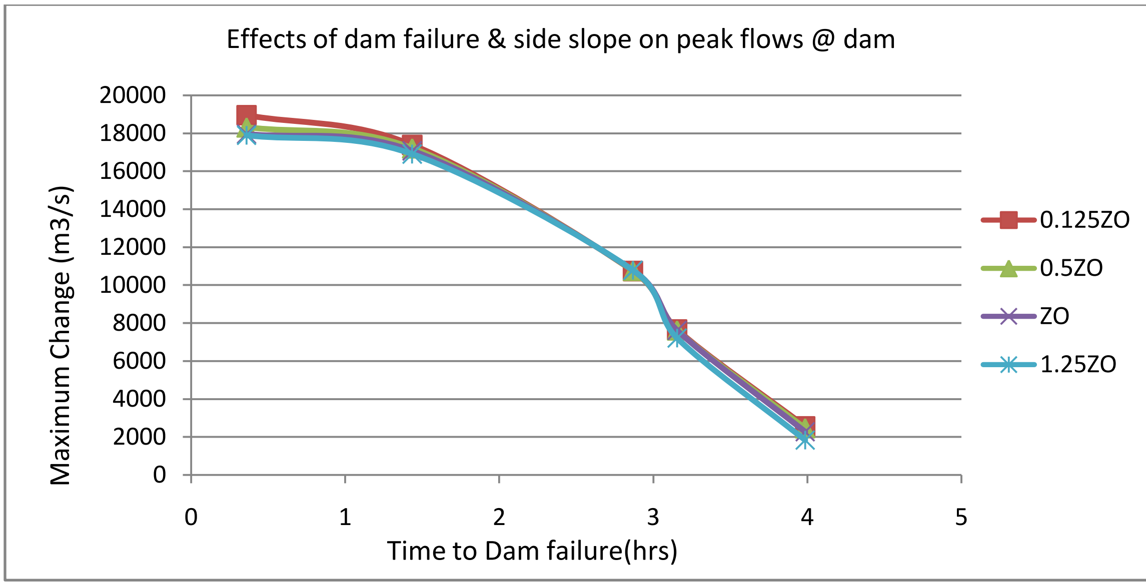

Figure 4-12: Effects of time to dam failure and side slope on peak flows at dam site

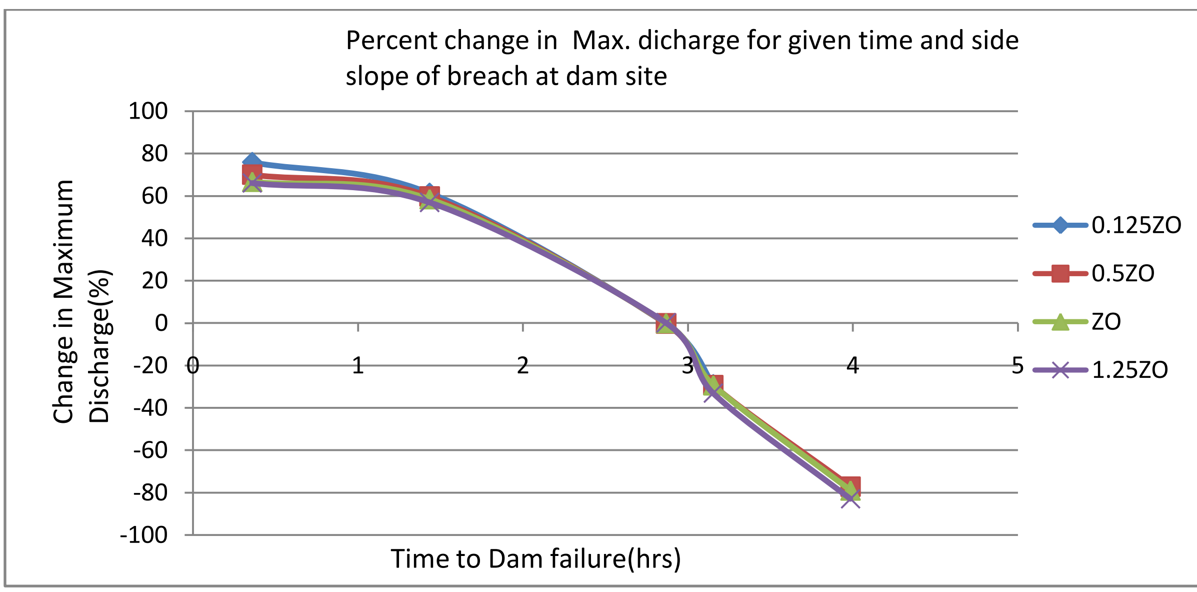

Figure 4-13: Percent change in max discharge versus time to dam failure for specified side slopes of breach at dam site.

From the above global sensitivity analysis graphical representation was done to determine a controlling parameter that influenced the maximum discharge at the dam site the most. Figure 4-12 and figure 4-13 illustrate the trend between the parameters at the dam site. The two figures depicted that the change in side slope of breach resulted in closely spaced graphs for a wide range of change in time to dam failure. This implied that the maximum discharge at the dam site was highly sensitive to the TFH than Z and B_o.

4.4.5 Sensitivity of Manning Coefficient for specified Time to Dam Failure

The downstream river routings were carried out by HEC-RAS computer program for five dam-break breaching outflow hydrographs under five TFH scenarios with side slope factor Z equal to 1.4. 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. Appendix 3 presented the detail inputs and outputs for downstream channel routing. This analysis was executed based on existing Manning coefficients and channel geometries data provided by the dam owner.

Table 4-11: Downstream peak flows for five TFH breaching outflow hydrographs

River station(Km)	Assumed time to dam failure for inflow hydrographs					Mannining Coeff.
	0.125 T_o	0.5 T_o	T_o	1.1 T_o	1.39 T_o	Nlb,Nc,Nrb
	peak flow(Q_{max})					
0	11891.53	11567.81	10,938.43	10675.5	10,570.26	0.035,0.03,0.035
15	7103.45	6753.9	6,117.59	6140.57	6,182.26	0.035,0.03,0.035
25	6708.34	6231.01	6,040.73	6298.24	6,144.67	0.035,0.03,0.035
45	6234.87	6123.21	5,408.22	5333.74	5,370.43	0.035,0.03,0.035
64	5821.89	5524.73	5,103.92	5270.43	5,114.32	0.035,0.03,0.035
Note: T_o represents original time to dam failure value equal to 2.867 hrs						

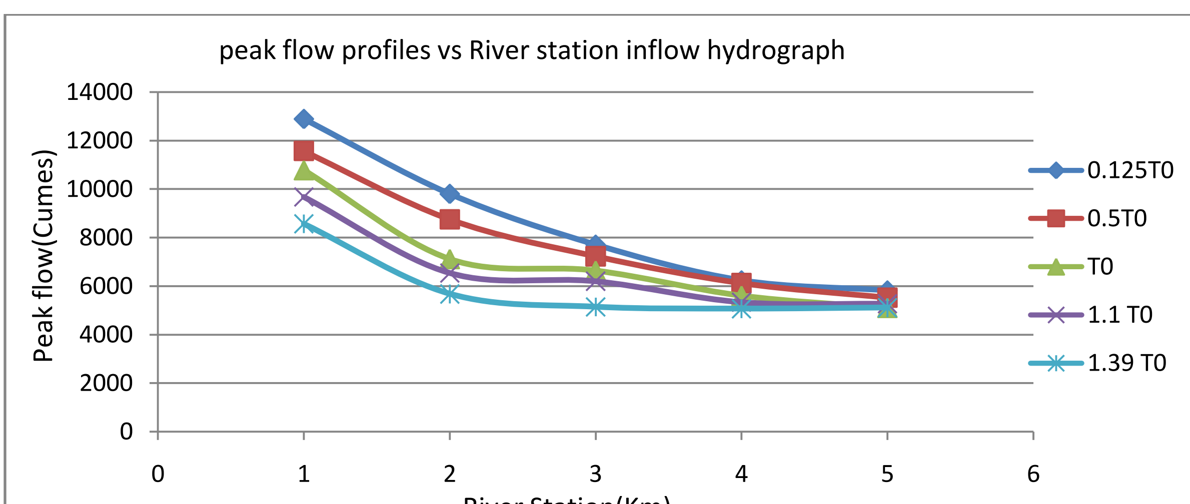

Figure 4-14: Peak flow profiles for five TFH breaching outflow hydrographs

As shown in table 4-11, five different unsteady flow routings were done to produce peak flows at respective river stations. Similarly, figure 4-14 shows the five peak flow profiles generated from five breaching outflow hydrographs that had five different dam breach development times at the dam site. It was obvious that smaller time to dam failure resulted in a higher dam breach hydrograph as discussed above, and the maximum discharge at the dam sustained its peak throughout the river routing.

The global sensitivity analysis was done to evaluate the relationship between Manning coefficients and peak flows for a given TFH breaching outflow hydrograph. According to the dam owner data, the downstream left, main and right channel sections of the Gibe whole reach had the manning coefficients (0.035, 0.03, 0.035), respectively. This analysis assumed four additional manning coefficients based on reasonable assumptions and literature reviews. Appendix 3 presented HEC-RAS inputs data and outputs for different Manning values. Tables 4-12 and figures 4-15 showed the influence of Manning coefficient change on the peak flows along the river reach. The peak flow profiles starting from the dam site down to the downstream end of the channel showed smooth transitions that implied the gradual effects of change in Manning value. The global sensitivity analysis results indicated that the smaller Manning coefficient resulted in higher peak flows for the entire reach length.

Table 4-12: Peak flows at different river stations for five Manning values

River station(Km)	Assumed Manning Coeff.				
	0.875No	0.937No	No	1.125No	1.25No
	peak flow(Q_{max})				
0	10,938.43	10,938.43	10,938.43	10,938.43	10,938.43
15	8,762.45	8,382.11	8,117.59	8,098.31	7,898.67
25	7,841.79	7,465.80	7,440.73	7,088.21	7,093.82
45	6,626.50	6,462.38	6,160.22	6,074.90	5,903.20
64	5,652.03	5,442.65	5,353.92	5,100.11	4,991.07
Note: N_o represents original time to dam failure value equal to 0.03					

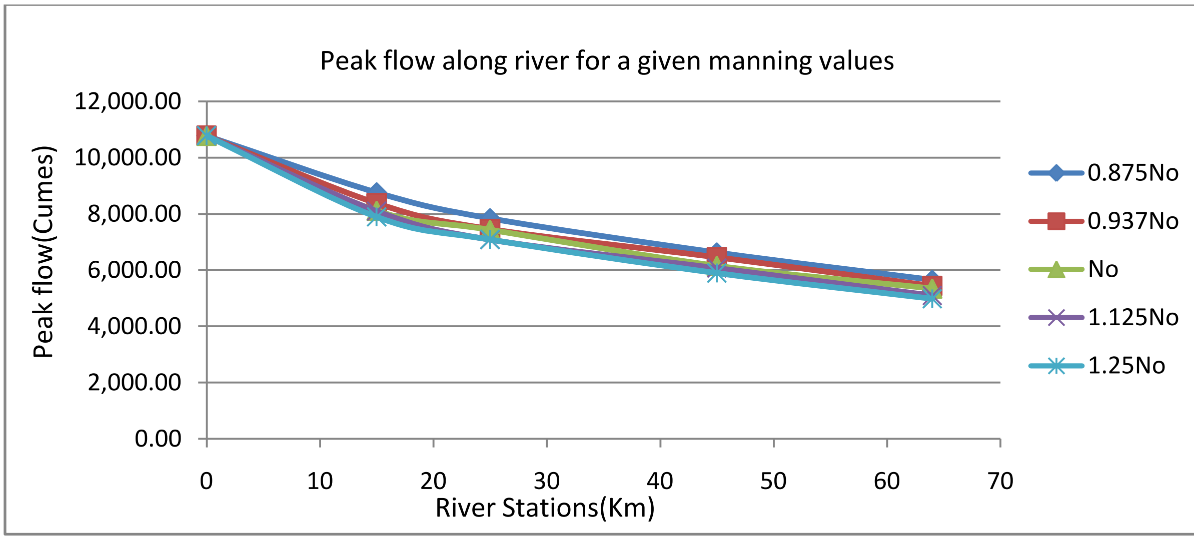

Figure 4-15: Peak flow profiles based on five different Manning values

To investigate the relationship b/n percent change in peak flows for a given change in Manning coefficient relative to baseline flow generated from existing Manning coefficients was further analysis. As it has been evaluated decreasing in percent change in peak flow from dam to downstream end of the channel. As it has been shown in table 4-13, a decrease in Manning coefficients by 12.5 % resulted a change in peak flows that ranged from the Dam by 0 % and decrease at the downstream end of the channel by 5.57 %. An increase in Manning coefficients by 25% resulted a change in peak flows that ranged from the dam by 0% to decrease at the downstream by 6.78 %. The results suggested that the peak flows in the channel were highly sensitive to changes in Manning coefficients.

Table 4-13: Percent change in peak flows for given Manning values

River station(Km)	Assumed Manning Coeff.				
	0.875No	0.937No	No	1.125No	1.25No
	Percent change in peak flow (%)				
0	0.00	0.00	0.00	0.00	0.00
15	7.94	3.26	0.00	-0.24	-2.70
25	5.39	0.34	0.00	-4.74	-4.66
45	7.57	4.91	0.00	-1.38	-4.17
64	5.57	1.66	0.00	-4.74	-6.78
Note: N_o represents original time to dam failure value equal to 0.03					

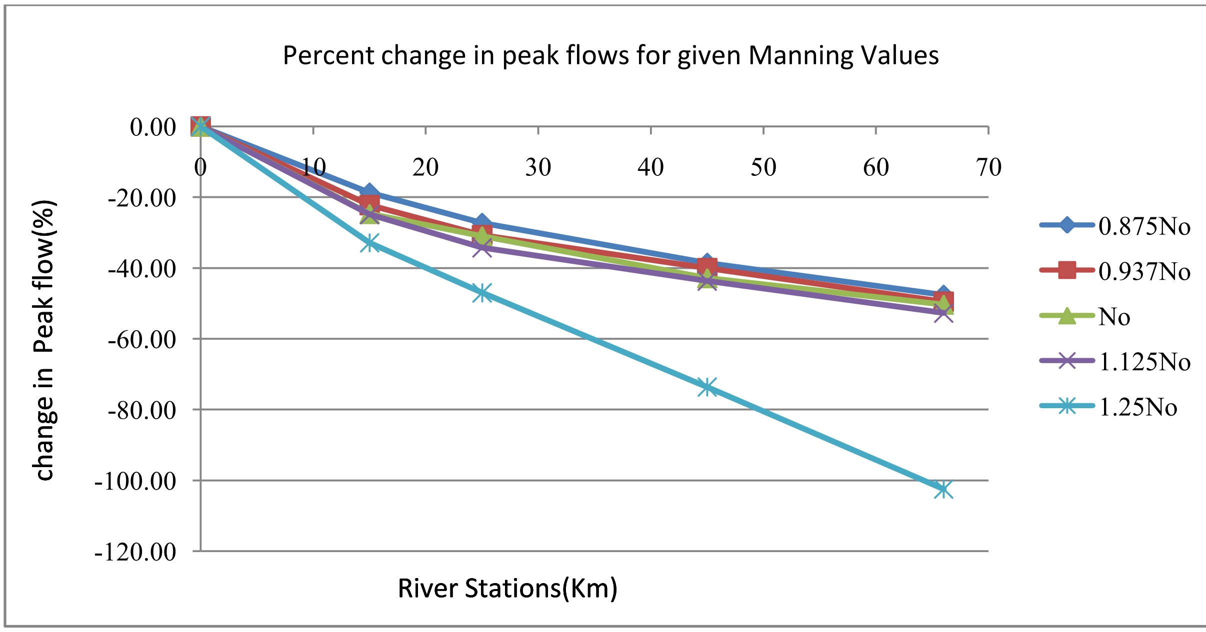

Figure 4-16: Change in peak flow profiles based on five different Manning values

The profiles of percent change in peak flow for specified Manning coefficients along the river relative to the baseline has been shown on figure 4-16. The relationship between the peak discharge and manning coefficients have been discussed and studied by researchers such as Chow for years. Based on his finding, the peak discharge and manning coefficients had inverse relationship as indicated in Manning's equation. According to the above figure and table, the Manning coefficient value decreases with increases in peak flow which confirmed with literatures reviewed by chow.

4.4.6 Sensitivity of Channel Bed Slope (So) under a specified Time to Dam Failure

The last part of global sensitivity analysis involved in studying the reactions of peak flow to change in channel bottom slopes that changed the corresponding downstream channel elevations at river stations. Five channel bed slopes were considered in the analysis as shown in table 4-14 above. Appendix 3 presented the detail of HEC-RAS inputs and outputs data.

Table 4-14: Existing and assumed channel bed slopes

River station(Km)	Assumed channel bed slope				
	S_o	$S1 = 0.950S_o$	$S2 = 0.975\ S_0$	$S3 = 1.025\ S_0$	$S4 = 1.05\ S_0$
0	0.0034	0.00323	0.003315	0.003485	0.00357
15	0.0032	0.00304	0.00312	0.00328	0.00336
25	0.0032	0.00304	0.00312	0.00328	0.00336
45	0.00289	0.0027455	0.00281775	0.00296225	0.0030345
64	0.0034	0.00323	0.003315	0.003485	0.00357
Note: S_o represents existing or baseline channel bed slope provided by dam owner					

Similarly, Figure 4-17 shows the profiles of the four channel bed slopes considered in the analysis relative to the existing channel bed.

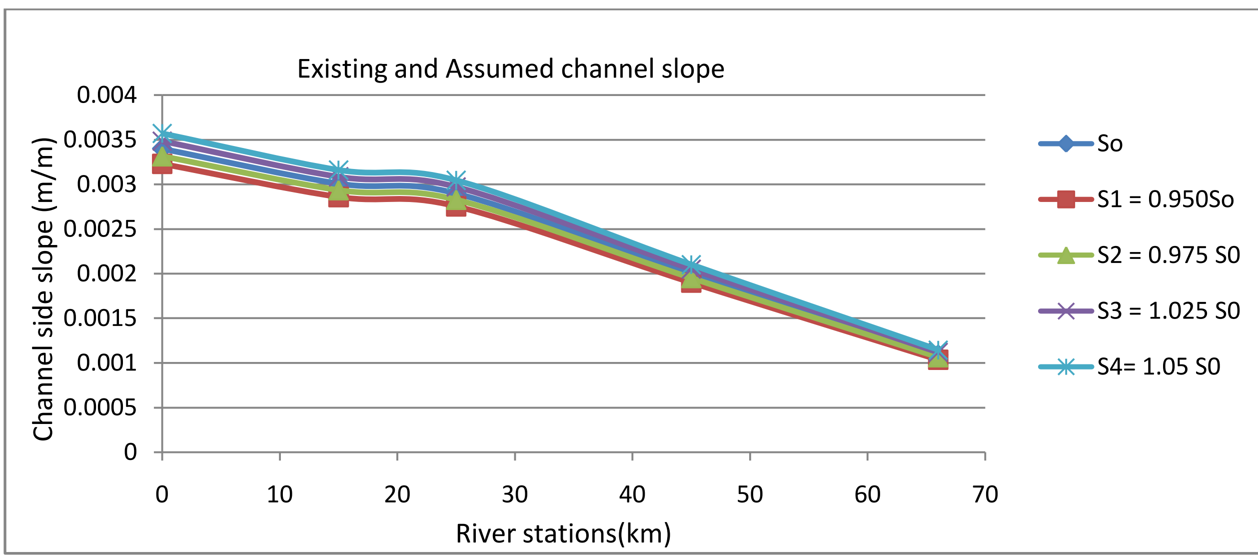

Figure 4-17: Profiles of existing and assumed channel bed slopes

The existing channel bed slope was used as a baseline to develop four additional bed slopes that helped to investigate the relationship between channel bed slopes and peak flows along the channel. In this analysis, the breaching outflow hydrograph generated by 2.867 hr of TFH was used to perform the unsteady flow routing through the downstream reaches. Appendix 3 showed the detail of inputs and outputs data for this analysis. The summary of computer runs for the assumed channel bed slopes are shown in the following tables.

Table 4-15: Peak flows for specified channel bed slopes

River station(Km)	peak flow(m3/s)				
	S_o	$S1 = 0.950S_o$	$S2 = 0.975\ S_0$	$S3 = 1.025\ S_0$	$S4 = 1.05\ S_0$
0	10,938.43	10,938.43	10,938.43	10,938.43	10,938.43
15	9523.09	9485.09	9499.12	9702.31	9791.39
25	7342.01	7221.07	7289.45	7714.01	7793.9
45	5342.41	5017.09	5120.97	5491.03	5580.13
64	3350.18	3160.05	3191.78	3467.17	3495.92
Note: S_o represents existing or baseline channel bed slope provided by dam owner					

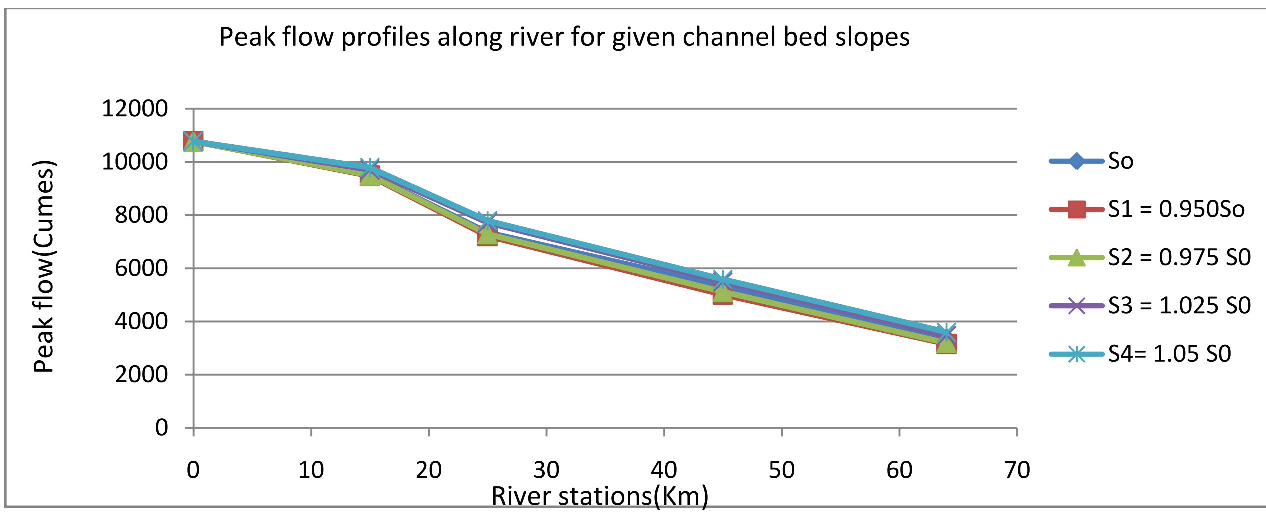

Figure 4-18: Peak flow profiles along the river for specified channel bed slopes

Table 4-16: Change in peak flows for specified channel bed slopes

River station(Km)	Change in peak flow along river (%)				
	S_o	$S1 = 0.950S_o$	$S2 = 0.975\ S_0$	$S3 = 1.025\ S_0$	$S4 = 1.05\ S_0$
0	0	0	0	0	0
15	0	-0.3990301	-0.2517	1.881952	2.81736285
25	0	-1.6472328	-0.71588	5.066732	6.15485405
45	0	-6.0893866	-4.14495	2.781891	4.44967721
64	0	-5.6752175	-4.7281	3.492051	4.35021402
Note: S_o represents existing or baseline channel bed slope provided by dam owner					

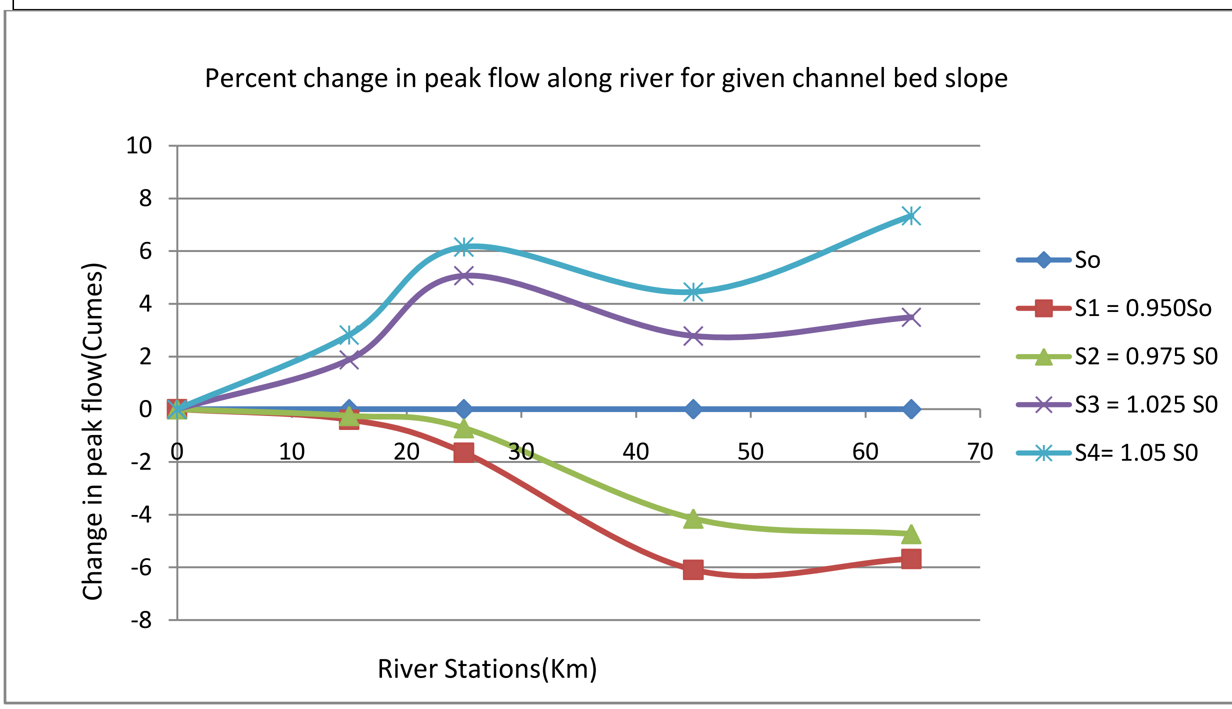

Figure 4-19: Change in peak flows along the river for given channel bed slopes

From the above analyses in table 4-15 and 4-16 shown that decrease in channel bed slopes by 5% resulted in changes of peak flows that ranged from 0% at the dam to 5.67 % decrease at the downstream. Similarly, 25% increase in channel bed slopes resulted in changes in peak flows that ranged from 0% at the dam to 6.15 % at the downstream.

From figure 4-18 and 4-19 shown the graphical representations that indicated the variation in peak flows was not noticeable at the dam site, but effects of change in channel bed slopes became more prominent at the reach stations distant from the dam site. The relative significances of change in channel bed slopes were consistent from Dam to downstream end as shown in above figures.

The computation results shown in the above tables and figures revealed that the peak flow along the channel was very sensitivity to a minor change in channel bed slopes, there was a significant variation in peak flow for a small degree change of S_o. As indicated in the above tables, the peak flow and channel bed slope have a direct non-linear relation. This implied that as the channel bed slopes increased by certain coefficients, the peak flow increased by certain multiplier which was consistent with the finding of this study which confirmed with literature from different researcher.

4.5 Flood Inundation Mapping and Its Preparation

The model results gave a flood depth 0.0247 m as the minimum to a critical height of 54.01 m for the catchment(overtopping) whereas flood depth 0.031 m as the minimum to a critical height of 42.23 m for the catchment(piping) (see Figure 4-20 & 4-21). In general, high water depth occurred along the main channel and spreads gradually to the floodplains. This can be attributed to the fact that the river has a lot of tributaries which contributes to high inflow into the main channel. Also the river flows in between slightly hilly terrain and, as such, rain water uphill flows rapidly into the river channel.

The inundations map for both overtopping and piping failure scenarios have been done and shown as follows. The inundation map Prepared due to PMF (hydrologic failure) represents overtopping failure scenarios where as non hydrologic failure represent piping failure scenarios of Gibe. The significant failure scenario which leads large flood plain according to the study result is the overtopping one. The inundation map for both failure has been shown as follows in figure and which is mostly representative for the study case (Gilgel Gibe I).

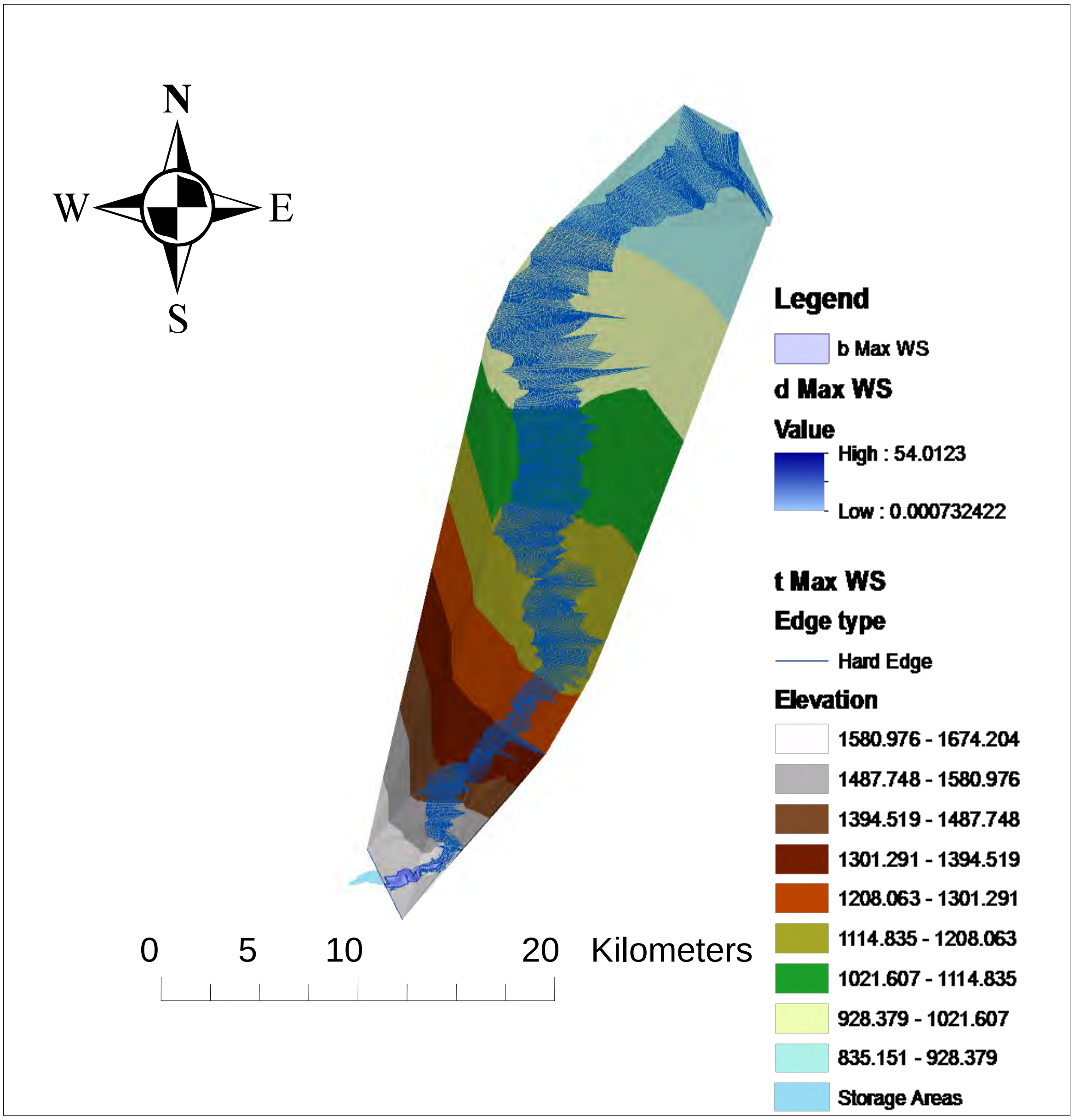

Figure 4-20: Inundation Map for overtopping

According to study result the inundation map for piping failure looks like the following. The piping failure has low significances when we compare to overtopping failure.

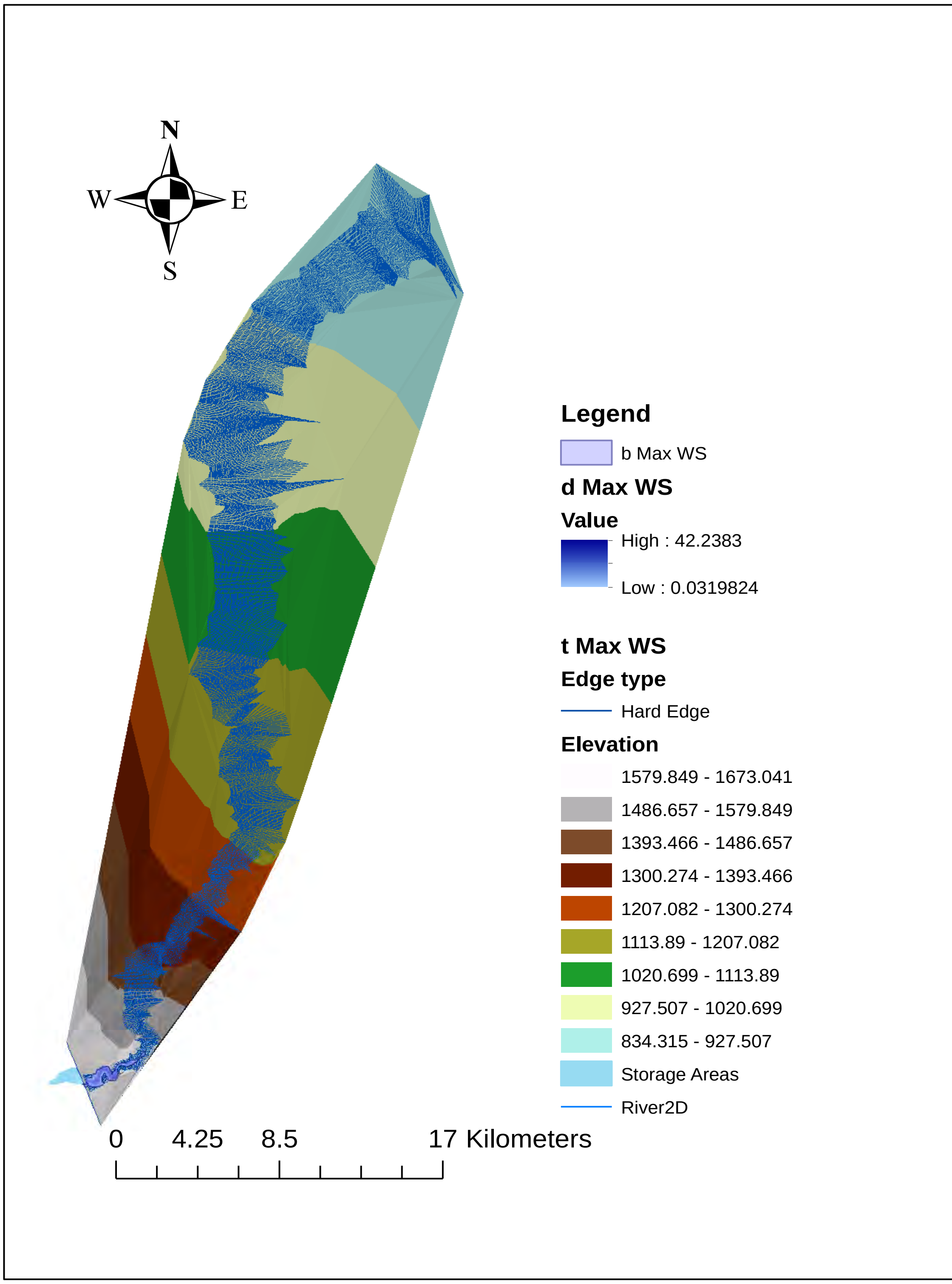

Figure 4-21: Inundation Map for Piping Failure

As it has been known major settlements are located downstream of the dam, and scattered rural dwellings and farm buildings are likely to be affected due to fails of dam and the downstream area is inundated. Other potential environmental and economic damages arising from a dam break event include the following: destruction of agricultural land of the community nearer to dam, economic loss to the dam owner including loss of asset and operating revenue, loss of livestock and topsoil, damage to Gibe I, deposition of silt and flood debris downstream and floodplain areas due to the size of the flood wave and expected inundation areas whether the failure mode is overtopping or piping. The inundation map and failure scenarios give direction for emergency action plan to see the perspective through which mitigation measure undertaken to reduce the risk if dam fails or take appropriate remedial measure before the failure happen.

5 REMEDIAL MEASURES FOR DAM BREACH

5.1 Introductions

Remedial measures are mostly undertaken in order to restore and strength the dam to meet the desired requirements and benefits beside that of improving the safety of dam over its design period. The remedial measure depends up on economical, environmental, social and technical factors etc. These factors affect the monitoring manner directly and indirectly. In this paper Overtopping and piping failure modes are studied. The hydraulic failures (overtopping) and seepage failure remedial measures are addressed. As it is know that Remedial measures are important in Strengthening, repairs, maintenance and restoring the damaged part to meet the desired function dam. In order to Strengthening, repair, maintance and remediate, dams should be carefully selected depending upon the following points.

- The risk that happen during the failure over the d/s as well as on dam,
- The economic and purpose of impounded water,
- Considering nature of foundation stratum and dam site,
- Materials and methods used in construction of the dam.

Unless remedial measures have be properly undertaken, the flood which comes from piping and overtopping dam failures results in flooding the downstream as a rampaging flood wave. Piping control measure may form a complex which occurs through embankment and its foundation. This can make detection methods and remedial measure difficult than any failure else. Whereas for overtopping it is not like that of piping but it‘s difficult is as it is. In most case remedial action may varies from permanent control to day to day monitoring through inspecting. Factors affecting the type of remedial measure needed includes: geological/geotechnical environment, risk, extent of correction required, feasibility of correction, failure of prior remedies etc.

Before the remedial measure selection criteria‘s has been taken in to account dam should be monitored and examined for cracks, leakages, deficiency of spillway to pass flood, loss of free board, saturated areas or wet spots, sinkholes, erosion, excessive growth of vegetation, frost, animal burrows, and deterioration of rip-rap or other slope protection materials. When determining the appropriate remedial action, consider how the action might affect other aspects of the project. The major remedial measures needed for this study are explained for overtopping and piping as follows:

5.2 Treatment for Controlling Piping Failure

The primary purpose of remedial measure for piping failure is to reduce the amount of water seeps through the body of dam and foundation. Monitoring seepage / piping control measures in foundation can lead to a rational conclusion with a minimum of expenditure. Visual observations and inspections are the most common and easiest monitoring manner at various intervals and reservoir elevations. The method of treatment may differ for foundation and body of embankment dam. The remedial measures for piping failure are categorized in A) foundation treatment and B) treatment in the body embankment.

5.2.1 Foundation Treatment

While foundation treatment undertaken it must minimize leakage/seepage, prevent internal erosion, prevent settlement that will cause loss of free board, sufficient friction development between the embankment and its abutments and foundation to ensure base sliding stability. The water seeps through impervious foundation mostly tackled by providing impervious cut off, relief wells and drain trenches and grouting the affected zone.

A) **Impervious Cutoff**

Vertical impervious Cutoff walls are provided at the u/s end (i.e heel) or far from u/s end (i.e. to reduce uplift pressure) to remedy seepage through foundation. Impervious cutoff can be extended till it reach the impervious foundation and if the previous layer is large the cutoff must be provided at lesser depth called partial cutoff. This cutoff reduces the seepage discharge in smaller amount. However, cutoff walls are extremely costly and thus are usually considered only as a last resort. The following Figure 5-1 shows the vertical impervious cutoff.

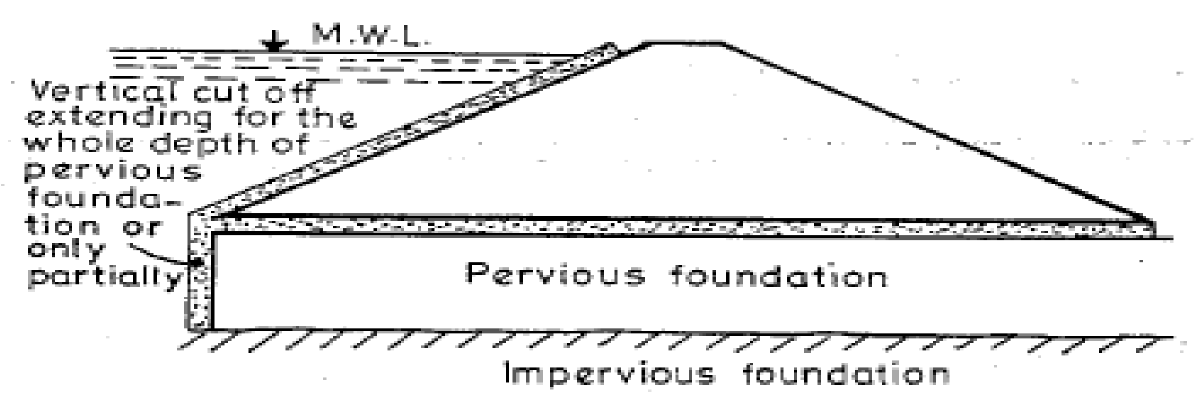

Figure 5-1: vertical impervious cutoff

B) **Relief wells and Drain Trenches**

When excess hydrostatic pressure / seepage through deep pervious strata underlying the dam occur there is a possibility that water may boils at near toe of dam. The severity of under-seepage both in respect of excessive hydrostatic pressure and seepage-flow is dependent upon head of water, permeability of substratum and characteristics of the upper strata of downstream portion. This problem may be tackled by properly constructed relief wells. Figure 5-2 shows the row of relief wall at toe of dam.

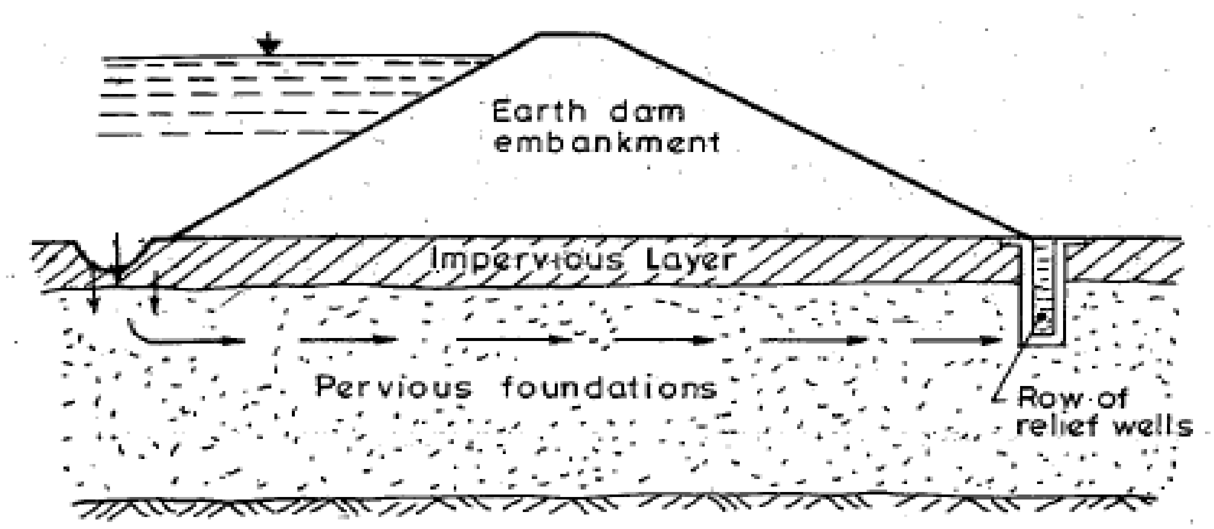

Figure 5-2: Row of relief wall at toe

The possibility of water boil at toe can be tackling by providing d/s berms beyond the toe of dam. In this case the provisions of berms protect the downstream toe from sloughing due to seepage. Figure 5-3 shows the place where berms are provided.

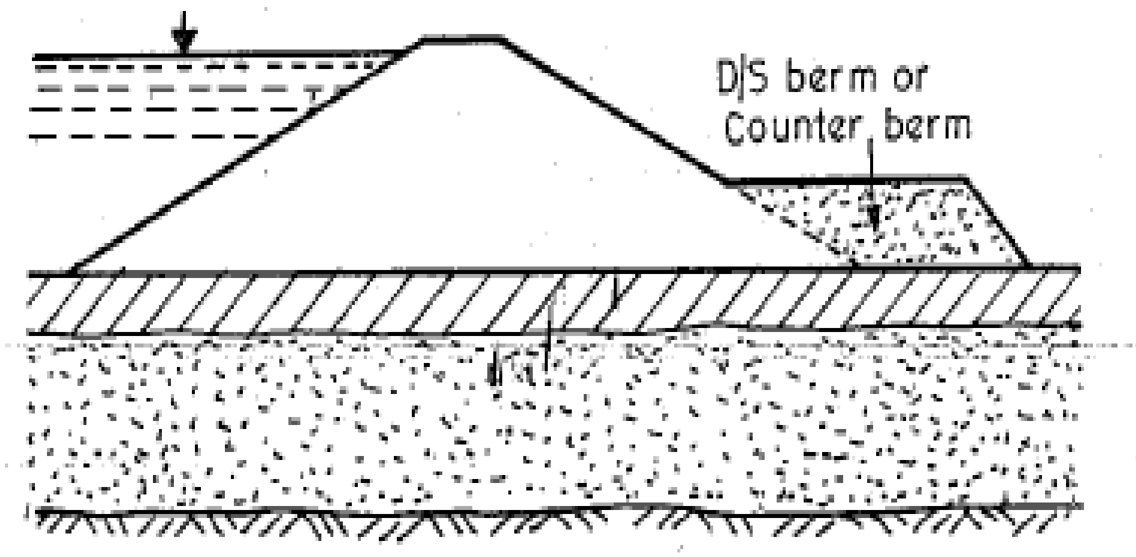

Figure 5-3: Downstream Berm provision above the toe of dam

C) **Grouting the Affected Zone**

Mechanism of injecting mixture of water, cement, and other chemical compounds by pressure into openings (voids, cracks, or joints) in a foundation by pressure is called grouting. Grouting may be done from the u/s or top of dam depending up on the nature of foundation. Grouting is the long term remedy seepage in foundation**.** The efficiency of grouting is very important in weak zone or affected area. Mostly it is applicable in area where seepage affects frequently. Removing the affected area and grouting it is more appropriate. It is impractical to provide a positive cut off or open partial cut off throughout the length of dam at service time of reservoir. If grouting results in sealing of the foundation just downstream of on beneath the downstream portion of the dam, uplift pressures may increase beneath the embankment or seepage may be forced up into the downstream portion of the embankment. Before grouting has been adopted pore pressure instrumentation should be in place to monitor the change. This is important in designing the remedial measures. Due to different variables, during grouting experienced engineers and contractors are needed.

5.2.2 Treatment in the Body of Embankment

The seepage /piping through embankment mostly affect the nature of the dam as whole. This means that as water leaks through the dam it erodes the internal part gradually and leads the small porosity of soil granular to a bigger hole which cause dam failure. In order to prevent /

reduce such failure the drainage measure are important. These includes: A) rock toe/filter, B) horizontal blanket, C) chimney drain and, D) u/s face impervious blankets.

A) Rock toe/filter

Rock toe consist of stones of varying size ranging from 15 to 20 cm. a toe filter is provided as transition zone b/n embankment fill and rock toe. Sand, coarse sand and gravels are the three layer of toe filter which are used to resist the erosion of soil from the toe of dam. The height of rock toe must range from 25 to 35 % of reservoirs head. The toe rock must be higher than the tail water depth to prevent wave action of the tail water (i.e. not to erode the soil from the dam toe). Sometimes other seepage controlling measure are used with toe (or trench) drain. In conjunction with toe drains perforated collector pipes are used in a trench and backfilled with filter material surrounding the toe drain pipe. A toe drain may only attract a small portion of under seepage, with other detrimental under seepage bypassing the drain when pervious foundation is deep or stratified. In such cases, relief wells are used to relieve uplift pressure and collect water at greater depths. The following figure 5-4 shows the toe drain used in conjunction with relief wells.

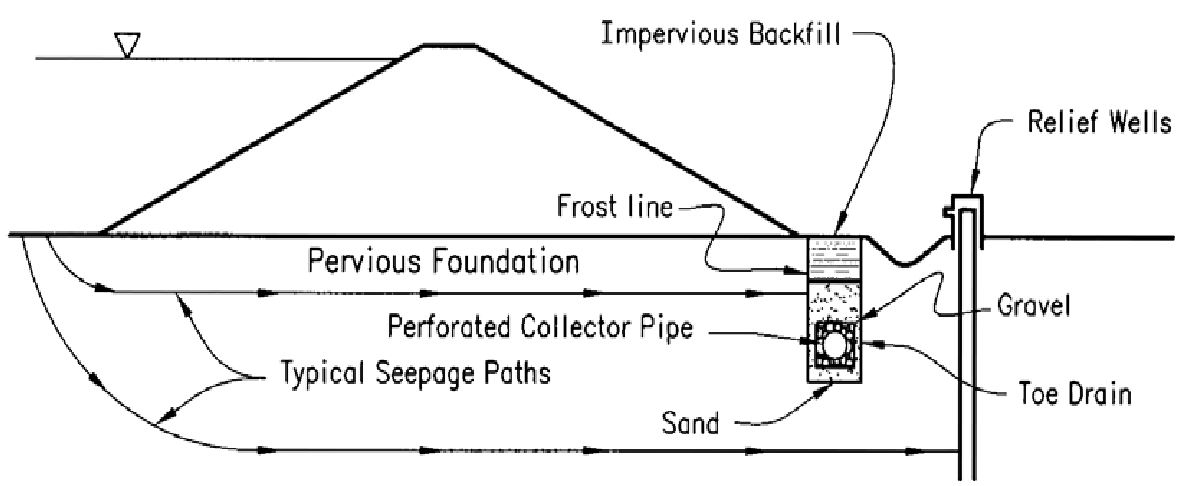

Figure 5-4: Toe drain used in conjunction with relief wells.

B) Horizontal Blanket/ filter

Horizontal blanket can be used to prevent seepage and excessive uplift pressures which occurs through an embankment or foundation. Horizontal filter mostly extends from d/s end (toe) of the dam up to a distance vary from 25 to 100% of the distance of the toe from the center line of the dam. It is sufficient if the length of Horizontal blanket is equal to three times the height of the dam. Horizontal blanket drains are effective only where seepage occurs through cracks, joints or fractured bedrock under uniform pervious soil foundation. Horizontal blanket drains are not completely effective in stratified embankments to draw down the phreatic line to the desired portion. Hence, the evaluations of horizontal drains should be seldom of the purpose on which the drains are required on. The use of horizontal drains significantly reduces the uplift pressure in the foundation under the downstream portion of the dam; however, it also increases the quantity of seepage under the dam.

Horizontal drains should infrequently be the only method for controlling embankment seepage. The reason for this is that in a stratified embankment the horizontal drain mostly at the base may not affect seepage in the upper portion of the embankment. A chimney drains are more effective than horizontal blanket drains. Figure 5-5 shows the horizontal drain arrangement.

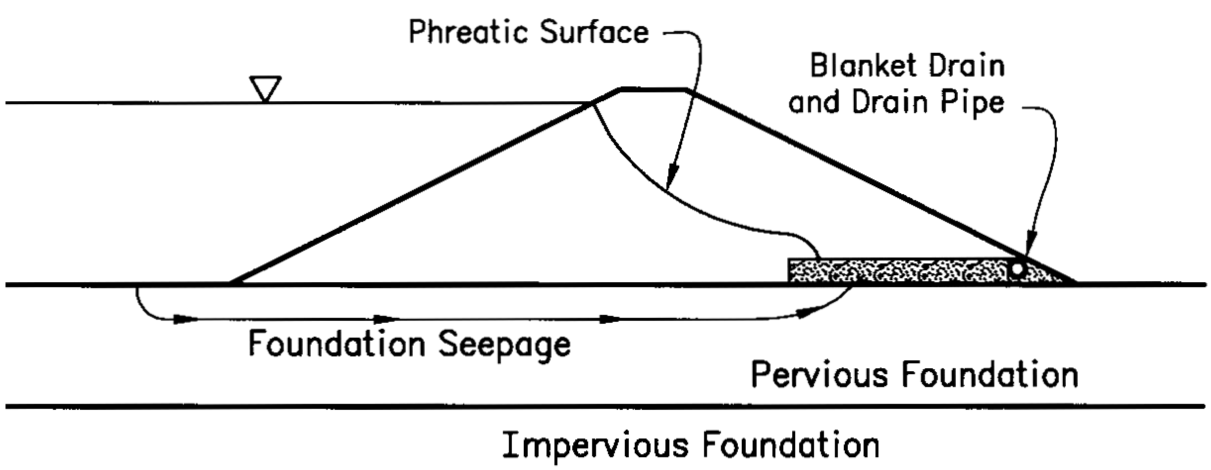

Figure 5-5: Horizontal (blanket) Drain

C) Chimney Drain

Depending up on the arrangement Chimney drains (inclined drains) are either inclined at u/s or d/s (45 degrees from horizontal to a vertical orientation) or vertically oriented drains of granular materials. In addition to Chimney drains horizontal filter must be provided to convey any seepage or leakage collected safely out of the interior of the embankment. Horizontal filter is important in bringing the phreatic line down in the body of dam, providing drainage of foundation and also helps in rapid consolidation. The horizontal filter arrangement makes the soil particles more previous in horizontal direction which causes stratification (i.e. horizontal permeability is greater than the vertical permeability). When large scale of soil particles becomes stratified such filter is inefficient. In such condition, vertical filter either inclined in u/s or d/s (chimney drains) is placed with horizontal filter to control seepage water efficiently. Chimney drains are commonly constructed with a filter zone at the upstream and downstream sides of the drain. Sometime the drains are used as filter.

Therefore providing a chimney drain with a horizontal drain, trench type drain, or relief wells are recommended to control seepage or internal erosion of embankment soils and foundation seepage. The chimney filter and drain are the best defense against transverse cracking and dispersive soils through the core, which may result from differential settlement or seismic shaking. Figure 5-6 shows an example of an embankment dam constructed with a chimney and horizontal drain combination.

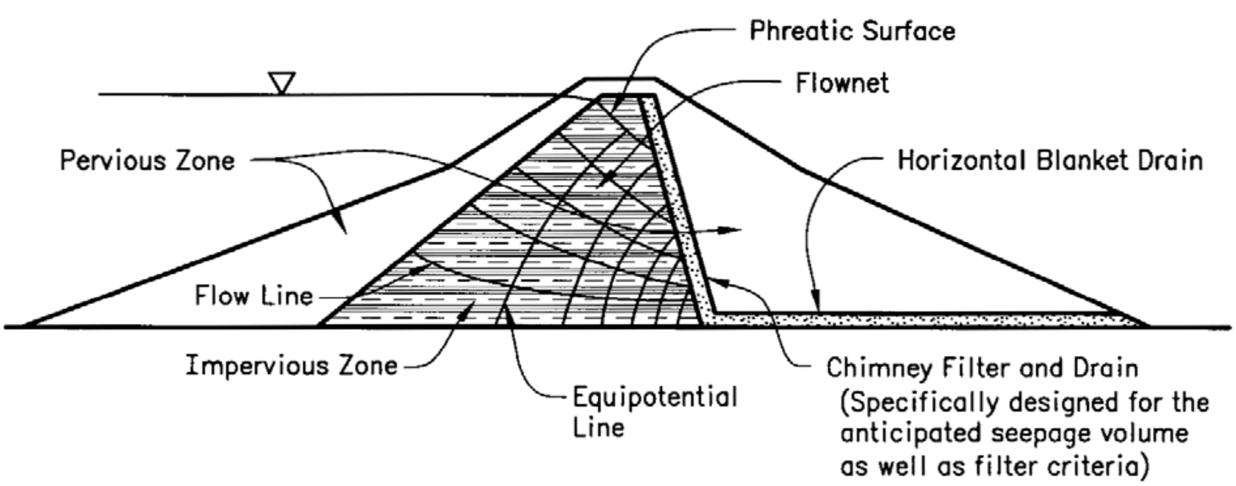

Figure 5-6: Combination of Chimney and Horizontal Drain

D) U/s face Impervious Blankets

The main aim of u/s face impervious blanket of low permeability is used to control seepage through the dam and foundation stratum. In most case it is constructed on upstream of embankment to increase the length of seepage paths in the dam body and reduce exit gradient of the d/s of the dam which prevent the piping and blowout. Upstream impervious blanket are applicable to reduce seepage quantities and pressure under the dam when sealing of the reservoir bottom or sides immediately upstream exist. It is economically feasible, efficient and successful, when the u/s source of seepage water is controlled through upstream blanket. When u/s face of embankments slope is Fine-grained materials the wave actions (i.e. drawdown) which generate from water in reservoir erode or remove the materials because of low saturated strength and high saturated weight. In such case the stone pitching must be provided to prevent the erosion of this soil. Upstream blanket will be less effective when upstream portion of embankment allows seepage water into the foundation. In such occasion additional remedial measure must be provided.

In order to prevent under seepage/ piping and/or excessive uplift pressures through embankment and foundation the downstream controlling measure like relief wells or toe trench drains are required with upstream impervious blankets. The effectiveness of the upstream blanket mostly depends up on length, thickness, vertical permeability, stratification on which it is placed. To safely pass under seepage water b/n blanket and foundations the Filter drains material are required. The upstream impervious facing with the total surface area of about 105,000 m² has been constructed with connection of sandwich-type asphalt concrete membrane to prevent the seepage dam and foundation for Gilgel Gibe dam. Therefore, the provisions of seepage controlling mechanisms are quite expensive and difficult after completion of construction whether it is through embankment or foundation. The above discussions coincide with study of Dunbar and Sheahan (1999).

Generally, another seepage controlling features must be combined with u/s blanket to prevent seepage through dam body and foundation.

5.3 Treatment for Controlling Overtopping Failure

An inability of spillway to safely pass flood flows can resulted failures of numerous dam in the world. Most of the time failures caused by hydrologic conditions can range from sudden (i.e. complete breaching or collapse) to gradual (i.e. with progressive erosion and partial breaching). Overtopping failure is the most common hydrologic conditions. The reasons that resulted overtopping of dam failure includes: insufficient capacity of spillway design, partly or fully blockage of spillway, losses of storage volume capacity. Due to these reasons , when the water level in the reservoir exceeds the height of the dam crest the water start to over flow on crest of dam then overtopping failures occurs. Embankment dams are very susceptible to failure when overtopped because of potential erosion over the d/s ace of the dam. If the erosion is severe, it can lead to a breach and failure of the dam.

For the failure happens in embankment due to insufficient capacity of spillway design, partly or fully blockage of spillway, upstream dam failure , losses of storage volume capacity there are many traditional alternatives for remedy the problems. Providing additional reservoir storage and enlarging the spillway capacity are few remedy alternatives which have been practiced in different countries. In economical wise it is often too costly. For this purpose some researcher designs different remedial measure to reduce costs and increase safety for both new and existing dams. Based on recent and ongoing overtopping protection research, the Federal Energy Regulatory Commission FERC has decided that embankment protection has sufficient merit to allow consideration on a project-specific basis (Frizell, et al 1991). The use of traditional in situ reinforced concrete has been enhanced as a major choice to achieve reliable stability and performance due to improvements in design methods and placements. Roller-compacted concrete, RCC, and cellular concrete mats, CCM, or precast concrete blocks have recently been used on several embankment dams in the US at a significant cost savings.

Among measure that has been practiced to reduce failures occurred due to overtopping over embankment are as follows.

A) Increasing crest width and free board
B) Increase the capacity of spillway and flood diversion
C) Covering the crest of dam and downstream face through concrete materials
D) Vegetation and placing riprap or stone pitching
E) U/s watershed management to reduce Sedimentation.

These are the remedy that has to be undertaken to prevent overtopping dam failure. The above remedial measures are discussed as follows.

A) Increasing crest width and free board

Freeboard mostly gives the margin of safety for overtopping failure of dams. As a general it is difficult to prevent splashing or overflow occur during wave action under extreme conditions. However, the occurrence of such threaten to structural integrity of the dam can be controlled. Depending up on their resistance occur due to wave action, the provision of freeboard for concrete dams can be less conservative than for embankment dams. As studies indicate when concrete dams overtopped without significant erosion of foundation or abutment material no need for freeboard is required under IDF conditions. This is because the overtopping of concrete dam cannot erode the d/s face as well as cannot lead significant failure of dam as it is expect on embankment dam. Under such circumstance special consideration must be required if power plant is located near the toe of the dam.

Crest width of the dams are most useful part which is determined based up on the properties of construction materials and minimum seepage allowable ,minimum roadway requirements, applicability and practicability of construction, designs and operation manner of dam in high seismic areas, crest raises and any planned future and potential security-related vulnerabilities. A minimum crest width should provide a reasonably low seepage gradient through the embankment at the level of the maximum reservoir. A wider crest provides greater safety against a breach of the dam in highly sensitive area of earthquake. Due to this increasing crest width of dam reduces the failure of dam.

Generally, raising and width crest of dam reduce the failures due to overtopping. In another word, increasing freeboard and crest width ensure that the IDF cannot easily overtop the embankment dams. In such circumstance dam failure cannot be entertained and no loss of life as well as property damage exists. To safeguard and reduce the failure of dam sufficient crest width and freeboard must be provided.

B) Increase the capacity of spillway and flood diversion

Increasing the spillway capacity is a way of securing that the requirements for dam safety against failure through overtopping as well as piping. Enlarging existing spillways and open up existing spillways that have been closed to regulate flow or by constructing new ones are way through which spillway capacity increased. One solution is to install auxiliary spillways in the form of self-regulating fuse plugs. The spillway which is large causes negative downstream consequences like erosion, flooding and increased load on downstream dams. Due to this it is vital to consider the whole river system and analyses the possible effects of an increased spillway capacity at a certain dam. In such condition increasing spillway capacity is complicated and expensive due to technical failure causes.

Another way of safeguarding embankment dam from failure due to overtopping is diverting the flood from dam to other way. Diverting flood away from the watershed in order not to reach the dam is one way of reducing the excessive incoming flood from the failure of the dam. In such case the entire water flow it can be diverted. The main benefit of flood diversion is to reduce the total hydrological load and stress that causes dam failure. Diversion of flood is a displacement of the water to space needed to reduce overtopping failure.

Generally, diversion of flood and increasing spillway capacity are important in reducing the effects of failure through overtopping flood. Increasing spill way capacity is expensive in economic wise.

C) Covering the crest of dam and downstream face through concrete materials

Among the failure modes overtopping is one that causes the dam to fail as well as allows the downstream face of the dam to erode comprehensively. In particular overtopping failure occurs when the reservoirs water level exceeds the crest elevation of dam due to PMF. The condition of mitigation method and potential impact must be evaluated for already constructed dam while trying to modify it for overtopping failure. This means that when excavation undertaken during construction it causes slope stability due to reduction in embankment cross section. Beside to this, when downstream stilling basin is constructed at downstream face to protect the dam failure the erosive potential increased. The geotechnical engineers and geologists should be involved in the evaluation of estimated risks of dam failure during construction as design of protection for overtopping failure. Different types of construction materials are used to construct Overtopping protection for embankment dams. The most commonly used overtopping protection for embankment dams includes: Roller-Compacted Concrete, Conventional Concrete and Precast Concrete Blocks.

The provision of Roller-Compacted Concrete has be successful erosion protection method for embankment dams. It is cost effective. The provision of RCC is very good in rapid construction with minimal project disruption. While RCC is provided no need of restricting reservoir but its limitation is for dam crest and downstream slope. RCC has been suitable for wide application because the material is suitable for wide range of flow depth and velocity.

Conventional Concrete is other types of overtopping protection for embankment dams. The use of conventional concrete for embankment dams relies on a continuous layer of concrete to serve as the flow surface for reservoir releases and to protect the underlying embankment from high velocity flows and surface erosion. In such case the guide walls are provided to protect the abutment and contain the overtopping flows. These guide walls are provided at sides of the overtopping protection. In order for conventional concrete to be effective as overtopping protection, the concrete layer must remain intact and be free of significant defects during a flood event.

Precast concrete blocks sometime called as articulated concrete blocks (ACBs). When the underlying materials are earthen materials, precast concrete blocks are provided to give a harder surface for overtopped flow to pass safely without causing erosion of underlying materials.

Based up on geometry, useful application and erosion prevention, installation procedures, maintenance requirements, aesthetic value, and cost there are many types of precast concrete blocks used for overtopping protection. Primary importance for overtopping protection is to select a commercial product that has been tested under the flow conditions expected during overtopping, and to ensure that an adequate filtered drainage layer is provided beneath the block system. Typical applications may experience high flow velocities, moderate flow depths, hydraulically steep slopes, and possibly tail water and energy dissipation on the flow surface.

Generally, Roller-Compacted Concrete, Conventional Concrete and Precast Concrete Blocks are overtopping protection mechanisms through which d/s dam face erosion are prevented to safeguard the dam from failure.

D) Vegetation and Placing Riprap or Stone Pitching

When heavy rains falls directly on d/s face of the embankment it erodes the surface by forming burrows, small rill and gullies. The formation of small rill, gullies and burrows on d/s face of embankment cause dam failure. The weathering effects and rill development on d/s face of dam due to heavy rainfall can be prevented by grass cover. In case if high rooted plants are grown on the downstream face it creates seepage paths due to extensive root systems. In addition to this, grassing the slope and by providing proper berms protect against the initiation of concentrated erosion which causes head cut of embankment dam. Nowadays, the provisions of riprap or stone pitching become important in preventing of wave action as well as d/s face erosion. Therefore, in preventing the overtopping problem on dam provide riprap to reduce the erosion. To get significant protective benefits good maintenance coverage is essential. For very steep embankment face grass vegetation is not suitable. Grass vegetation is an inexpensive cost wise and effective way to prevent erosion of embankment surfaces than other forms of mitigation measure, but maintenance costs can be higher. For lower flow rates and flatter slopes, riprap is most cost effective for overtopping case.

E) U/s Watershed Management

The watershed management is important in reducing the effect of reservoirs sedimentation and controlling water quality of impounded water. Reservoirs sedimentation cause decrease storage capacity of the reservoirs which leads reduces power generation, water supply and water delivery for irrigation land etc. Reservoir sedimentation depends up on reservoir geometry, river regime, flood frequencies, operation manner, sediment deposition way and possible land use changes over return period of the reservoir. The main sources of sedimentation arise from various geological formations, cutting and burning of brush land and forest, over-grazed grasslands from tribute, natural hazards including landslides, volcanoes, and changes in land use. The consequences of reservoirs sedimentation are loss of reservoirs capacity and subsequent effects. These effects are decreasing in irrigation and hydropower water demand, shortage of water supply for human consumptions, increasing hydropower equipment maintance and repair, decline in water quality and increasing cost for removal of sediment etc.

Unless sources of sediments are mitigated properly reservoirs capacity, water quality and subsequent effect will be disturbed. If the watershed over the catchment is properly managed or mitigated, the loss of reservoirs capacity due to sedimentation becomes dream. The effect of sedimentation can be mitigated through watershed management in sustainable pattern.

To increase reservoirs service life span practicing watershed management techniques are significant activities. This can be done either by implementing land management techniques (i.e. integrated watershed management) or by implementing reservoirs design (i.e. reduce the sediment intake).

Generally increasing vegetation cover over the catchment area and at tributary through plantations, tillage, crop management and constructing engineering measure such as terrace are recommended as watershed management measure to reduce sediment inflow in to reservoirs. Thus, to increase the life span of Gilgel Gibe it is must to use conservation measure (i.e. plantations u/s catchment, tillage, crop management and constructing terrace) to enhance the loss of reservoir capacity and subsequent effects for the desired return period.

5.4 Execution of Remedial Works

For any enlargement, alteration, rehabilitation, repair or abandonment of existing structures or facilities, the original design documents and all available construction and operation records should be carefully studied. If the rehabilitation would require substantial structural modification or if basic assumptions and environmental conditions which form the basis of the original design have considerably changed, the whole structure should undergo a new stability analysis. In most case mitigation measures such as relief wells, rock toe, horizontal drains, or chimney drains which prevent seepage forces must be practiced to reduce the piping failure. For overtopping failure prevention measures such as u/s watershed management, d/s grass cover and covering crest through concrete materials, increasing spill way capacity, flood diversion and increasing crest width and free board must be practiced. Slope instability is caused by inadequate erosion protection. The slope and toe protection of all embankment dams should be reviewed to determine if the dam is adequately protected against erosive forces.

6 CONCLUSION AND RECOMMENDATION

6.1 Conclusions

The significant dam breach analysis and remedial measure of Gilgel Gibe rock fill dam is simulated through HEC RAS as model for the hazards classification as thesis work. The impact of dam break over inters area both by piping and overtopping mode are observed in terms of flood hydrograph and flood prone area. A slight change in dam breach characteristics and downstream river parameters resulted in significant changes in peak flows at the dam site and specified reach stations in downstream channel at different kilometers. The rates of changes in peak flows were not uniform in the specified analyses as it is mention in result and discussion on the above chapter. In HEC RAS model, the peak flows were sensitive to changes in time to dam failure, side slope of breach, Manning coefficient at various degrees and breach parameters.

According to the result obtained through regression methods using Macdonald and Langridge-1984, Von Thun and Gillette-1990, Froehlich-1995 and Froehlich-2008 the breach bottom widths for overtopping event were become 362.42 m, 110.4 m, 222.089 m and 164.052m, respectively. Whereas the formation times for overtopping were become 2.85 hrs (Macdonald and Langridge-1984), 0.99hrs (Von Thun and Gillette-1990), 2.867 hrs (Froehlich-1995) and 2.46 hrs (Froehlich-2008). In similar manner the bottom breach widths for piping were obtained using Macdonald and Langridge-1984, Von Thun and Gillette-1990, Froehlich-1995 and Froehlich-2008 for MWL and spillway crest were (348.88m ,461.53m), (105.4 m ,85.4 m),(142.9 m,164.052 m) & (111.76 m ,110.23 m),respectively.

As results of HEC-RAS Model simulation shown, the peak discharge of 10,938.43 m3/s (overtopping) and 8,700.57 m3/s (piping) obtained from the analysis. Gilgel Gibe Dam breach overtopped in the case of maximum probable flood (PMF) by 43 cm, 72 cm, 57 cm and 33 cm for Macdonald and Langridge-1984, Von Thun, Froehlich-1995 and Froehlich-2008 respectively. In order to mitigate the failure mode and put the emergency action plan for maximum incoming flood which is overtopped by 57 cm the Freohlich - 1995 is selected based on study undertaken by Wahl 1998.

The local sensitivity analysis was undertaken between TFH, SS, B vs. peak discharge. According to the analysis, as 87.5 % reduction in TFH (T_o to $0.125T_o$) resulted in 19.33 %

increase in peak discharge at the dam site. Whereas, a 39 % increase (T_o to 1.39 T_o) in TFH resulted in 39.05 % reduction in peak discharge at the dam site. The peak discharge decreased when TFH increased as the evaluation result shown. In similar manner, when side of slope of breach was evaluated against peak discharge. The side slope of breach (Z_o to 0.125Z_o) decreased by 87.5% as maximum discharge at the dam increased by 36.76 % with relative to original data. Whereas, when SS (Z_o to 1.25Z_o) increased in 25% the maximum discharge at the dam site decreased in 10.11 %. This shown that, the rate of increase in peak flow against SS for a given percent change was higher compared to the rate of decrease as shown in the analysis. From the analysis peak discharge increased when bottom breach width increased at the dam. As bottom breach width decreased by 87.5% the peak discharge decreased by 1.92 % at the dam relative to original data. Whereas, a 100 % increased in bottom breach width value produced 3.68 % increased in maximum discharge at the dam site. From the analysis the rate of increase in peak flow for a given percent change in bottom breach width was higher compared to the rate of decreased.

The global sensitivity analysis was undertaken between roughness coefficients, side slope of channel against peak discharge. The global sensitivity analysis was shown the overall effect of side slope of channel and roughness coefficient from dam to downstream of the channel. As the study result shown decrease in Manning coefficients by 12.5 % resulted a change in peak flows that ranged from the Dam by 0 % and decrease at the downstream end of the channel by 5.57 %. An increase in manning coefficients by 25% resulted a change in peak flows that ranged from the dam by 0% to decrease at the downstream by 6.78 %. The results suggested that the peak flows in the channel were highly sensitive to changes in manning coefficients. Another global sensitivity analysis was shown that, as channel bed slope decreased by 5% the peak discharge resulted at dam 0% to 5.67 % decreased at the downstream. Similarly, 25% increase in channel bed slopes resulted in changes in peak flows that ranged from 0% at the dam to 6.15 % at the downstream.

The Breach outflow flood from HEC RAS model showed that the inundated areas ranges from a depth of 0.0247 to 54.01m at the river channel to flood plain areas out of the left and right banks for the case of overtopping failure. Flood depth for piping was 0.031 m to 42.23 m for the flood plain area. Due to this high water depth occurred along the main channel and spreads gradually to the floodplains. As the study result showed that overtopping failure mode was more risky that piping failure more.

According to the result obtained from the analysis major settlements were located downstream of the dam and scattered rural dwellings and farm buildings were likely to be affected due to fails of dam and the downstream area was inundated. Destruction of agricultural land of the community nearer to dam, economic loss to the dam owner including loss of asset and operating revenue, loss of livestock and topsoil, damage to Gibe I, deposition of silt and flood debris downstream and floodplain areas due to the size of the flood wave and expected inundation areas were the significant effect that occur due to the failures of Gilgel Gibe dam.

Risk reduction management measures to address potential failure modes for overtopping and piping of Gilgel Gibe embankment dam were developed. These risk reduction management measure were selected based up on effectiveness in reducing risk, cost effectiveness, damage on environment and hazard impacts after failure. The treatment for controlling piping failure was grouped as foundation treatment (i.e. impervious cutoff, relief walls and drain trenches, grouting affected areas) and embankment the body treatment (i.e. rock toe, chimney drain, u/s face impervious blanket). Whereas the overtopping failure treatment included u/s watershed management , vegetation and providing stone pitching , increasing the capacity of spillway and flood diversion , increasing crest width and freeboard , and covering the crest and d/s through concrete cover). These were the treatment measure that has to be under taken to reduce the risk of failure. Generally the economic and effectiveness of these measure were evaluated and examined as per the desire and failure type occur.

Generally, the above summaries were obtained through application of HEC-RAS model, the following conclusions were made.

- Gilgel Gibe dam fail both through overtopping and piping failure modes.
- The maximum breach outflow discharge for overtopping is 10,938.43 m^3/sec which largely small than calculated peak discharge by Froehlich (1995) method which results 57 cm overtopping.
- The peak breach out flow discharge in piping 8,700.57 m^3/sec is again small than calculated one.

- When we compare the peak out flow discharge of overtopping with piping, the overtopping peak discharge becomes 1.26 times the piping peak discharge or in other word 2,237.86 m^3/sec different happens b/n the two peak flow at dam breach time. This indicates that the overtopping failure mode is more risky than the piping according to the result of HEC RAS.
- The population settled at downstream of Gibe are largely affected. The effect may differ as the kilometers increase downstream from the dam site.
- The peak flows in the downstream reaches were directly related to maximum dam-break breaching outflow hydrographs. The smaller time to dam failure resulted in a higher dam breaching hydrograph, and the maximum discharge at the dam site sustained its peak throughout the downstream river. This indicates that the peak discharge will be high when the dam fails abruptly.
- Due to large pounding area (reservoir) the hydrograph attenuation of the study becomes small.
- The hydropower plant (184MW) of Gibe situated at downstream of the dam is highly affected by the breach of dam which leads economical crisis of the country.
- As the peak out flow hydrographs differs for both piping and overtopping failure mode in HEC RAS model result ,the crucial breach parameters, Manning roughness coefficient, channel bed slope, and PMF of the study area has its own contribution in making the output different even thought the principle of study in both case has difference.
- The maximum discharge at the dam site was very sensitive to change in TFH.
- As the Bottom breach width value decreased or the Bottom breach width became narrow, the maximum discharge at the dam decreased. It was obvious that when the bottom width becomes larger (width), the area of breach increases this leads release of huge water.
- The maximum discharge at the dam site was slightly sensitive to change in SS.
- Based on the test results, evaluation of the relative influences of the two parameters on the maximum discharge demonstrated that the peak discharge was highly sensitive to a smaller change in TFH than SS at the dam site.
- The peak flows in the channel were highly sensitive to changes in Manning coefficients.

- The peak flows in the channel were highly sensitive to a minor change in channel bed slopes, i.e. as the channel bed slopes became flatter and the peak flows became higher.
- Technical analyses to produce inundation maps and estimate downstream hazards can range from relatively simple to extremely complex depends on the degree and extent of the hazards, the accuracy and resolution of data available, and the financial and technical resources of the dam owner. Therefore, dam owner must look at the appropriate methodology to take the remedial measure accordingly following Inundation map analyses.
- Impervious cutoff, relief walls and drain trenches, grouting affected areas, rock toe, and chimney drain and u/s face impervious blanket were the treatment measure that has to be practiced to control piping failure.
- U/s watershed management , vegetation and providing stone pitching , increasing the capacity of spillway and flood diversion , increasing crest width and freeboard , and covering the crest and d/s through concrete cover were controlling measures to tackle overtopping failure.
- In addition to remedial measures the national strategy should be developed to tackle appropriate dam hazard mitigation, preparedness, and risk management actions.

Generally the above points has been seen and concluded while the study analysis is undertaken. The dam break modeling of Gilgel Gibe results can be used by different concerned body as it is desired.

6.2 Recommendations

The study of this paper mainly focus on dam break analysis and remedial measure which consist of the estimation of breach parameters through different regression methods, identify failure modes, estimate sensitivity analysis, indicate flood prone area and flood routing on the downstream of Gibe reach.

Based up on literature reviews and historic data that have been done so far, Dam breach analysis and breach parameters involve different assumptions using Gibe Dam. In estimation and prediction of breach parameters and breaching outflow large degree of uncertainty exist in real world. It would be helpful to minimize the ambiguities associated with estimation of breach parameters and flood routing using HEC RAS model. In analyzing dam breach of Gibe, only one dimensional unsteady flow routing technique was used to carry out the analysis in the downstream river. It would be helpful to utilize a different version of the software and enhance the findings of this study.

The results of this study showed that the dam will be overtopped and piped when an inflow flood of 48 hours 0.5 PMF (2 days 0.5 PMF) occurs in the watershed of the Gibe dam. In order to prevent the dam from failure the above mentioned remedial measure must be taken properly as it is need with it necessity. In addition to this, the communities living around dam should be encouraged to practice land use planning to failure zones to prevent low hazard dams from becoming high-hazard-potential dams. The concerned communities should aware about the dam failure and Emergency Action Plan (EAP) what to do when a dam failure is underway.

Since Gibe dam is under operation the above recommended points can be executed properly and improve the service return period as desired. In addition to these points before the dam breach happens if the downstream community is resettled it is good to the entire stakeholder to minimize the risks that happen in time of breach.

Finally, I would like to recommend the dam owner (Ministry of water, irrigation and Energy), the government and the concerned body to give special attention to the Dam break analysis and remedial measures make a detail investigation by using the latest version dam break software‘s and check whether the dam breach or not through overtopping and piping failure mode if the dam fails in both or one of the two mode take the remedial measure accordingly to save the economy , life loss and environmental damage happen in failure time.

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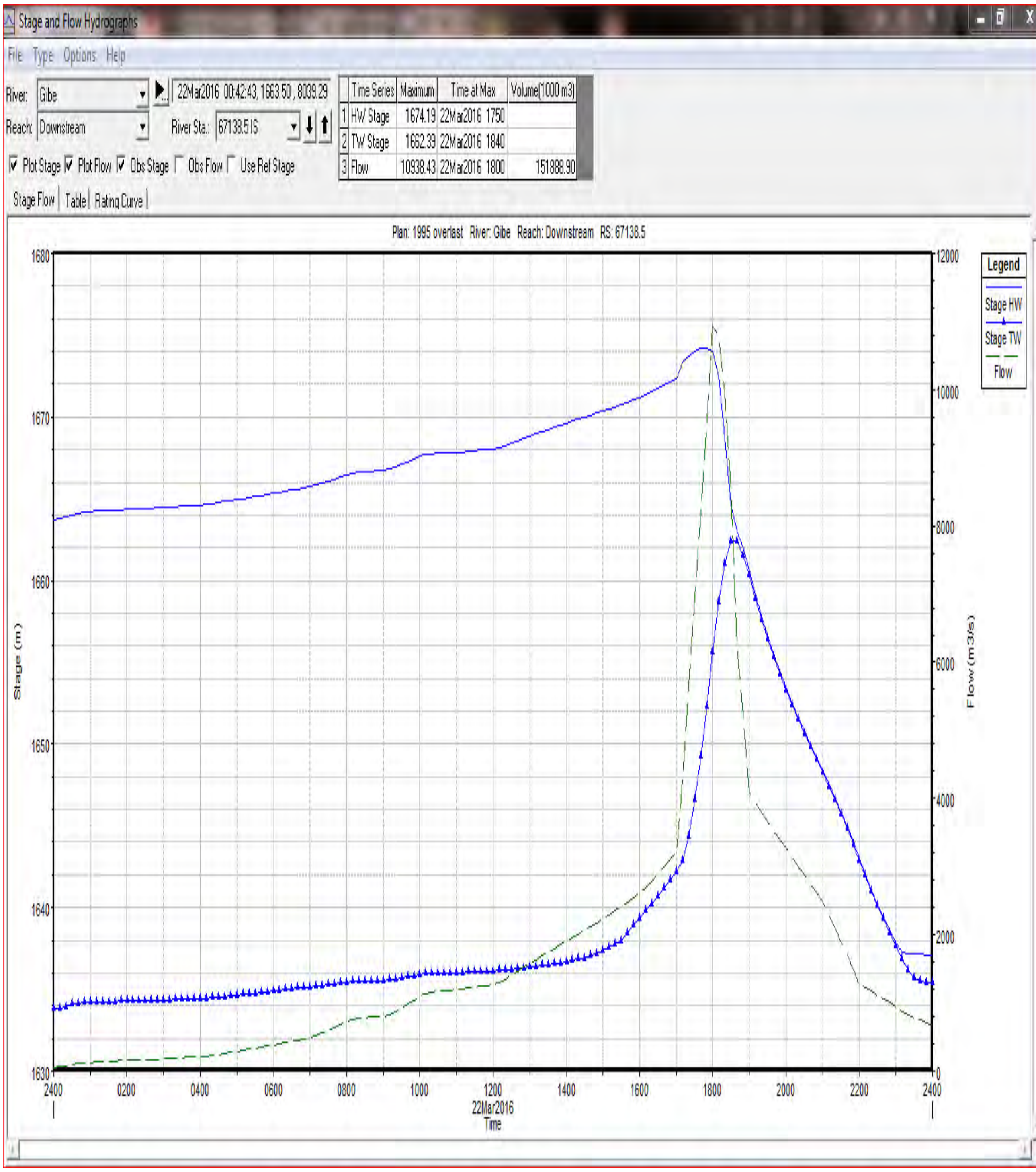
APPENDICES

HEC-RAS DAM BREAK ANALYSIS OUTFLOW DATAS AND HYDROGRAPH

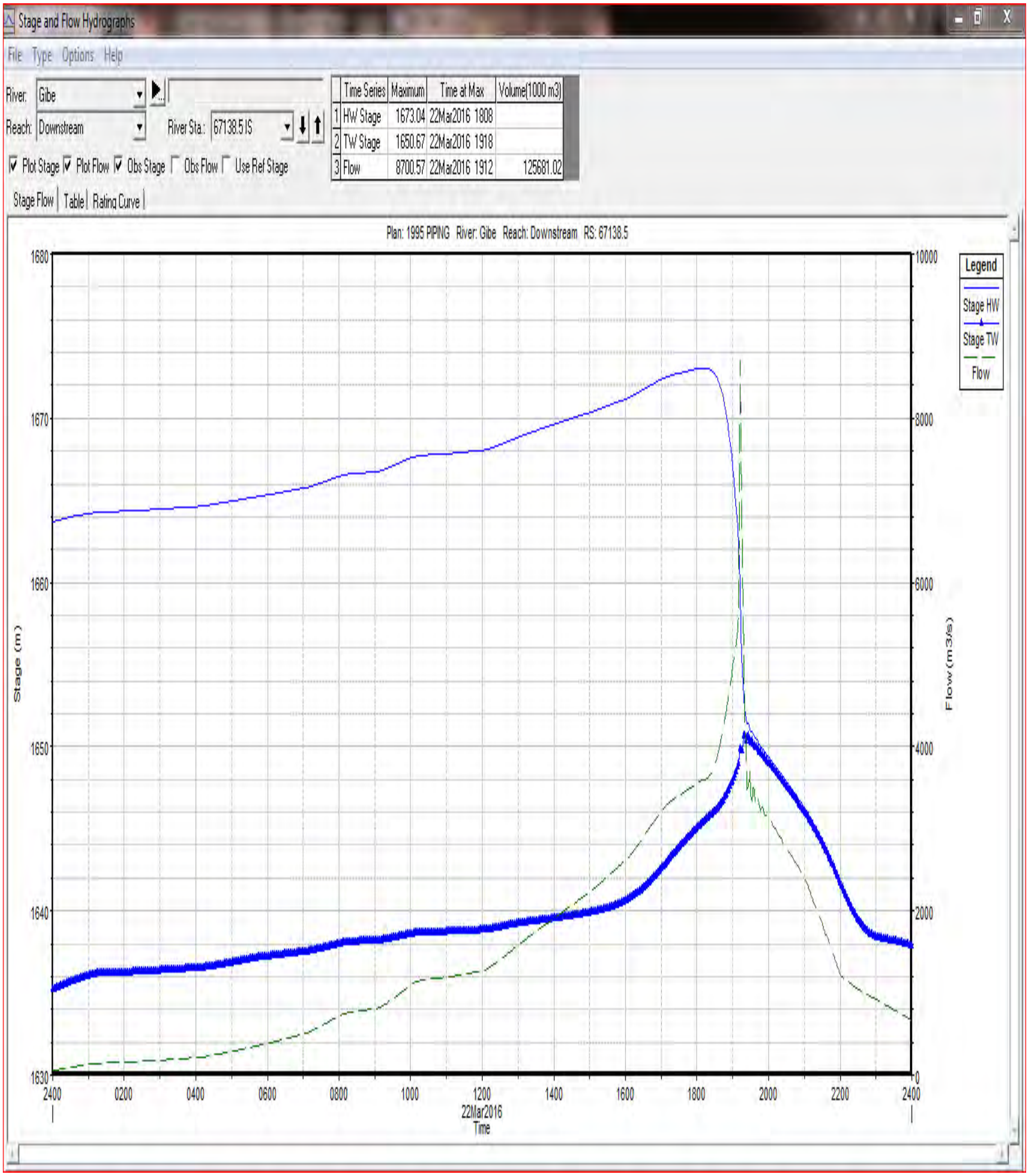
APPENDIX – 1

BREACH OUTFLOW HYDROGRAPH FOR OVERTOPPING


Stage and Flow Hydrographs
File Type Options Help
River: Gibe
Reach: Downstream
22Mar2016 00:42:43, 1663.50 , 8039.29
River Sta.: 67138.5 IS
Plot Stage Plot Flow Obs Stage Obs Flow Use Ref Stage
Stage Flow Table Rating Curve
Time Series | Maximum | Time at Max | Volume(1000 m3)
1 HW Stage 1674.19 22Mar2016 1750
2 TW Stage 1662.39 22Mar2016 1840
3 Flow 10938.43 22Mar2016 1800 151888.90
Plan: 1995 overlast River: Gibe Reach: Downstream RS: 67138.5
Legend
Stage HW
Stage TW
Flow
Stage (m)
Flow (m3/s)
22Mar2016
Time

APPENDIX – 2

BREACH OUTFLOW HYDROGRAPH FOR PIPING


Stage and Flow Hydrographs
File Type Options Help
River: Gibe
Reach: Downstream
River Sta.: 67138.5 IS
Plot Stage Plot Flow Obs Stage Obs Flow Use Ref Stage
Stage Flow Table Rating Curve
Time Series Maximum Time at Max Volume(1000 m3)
1 HW Stage 1673.04 22Mar2016 1808
2 TW Stage 1650.67 22Mar2016 1918
3 Flow 8700.57 22Mar2016 1912 125681.02
Plan: 1995 PIPING River: Gibe Reach: Downstream RS: 67138.5
Legend
Stage HW
Stage TW
Flow
Stage (m)
Flow (m3/s)
22Mar2016
Time

APPENDIX- 3

SENSITIVITY ANALYSIS OUTPUTS AT DAM SITE AND DOWNSTREAM REACHES

Maximum discharge and time to dam failure at dam site

Time of breach development(hrs)	Side slope of breach(1: Z_o)	Maximum discharge (m3/s)
0.358	1: 1.4	12,863.45
0.717	1: 1.4	11,552.50
1.434	1: 1.4	10,980.72
2.867	1: 1.4	10,938.43
3.154	1: 1.4	8,675.50
4	1: Z_o	6,570.26
Note: Z_o and T_o represents typical values &equal to 1.4 & 2.867 hrs respectively		

Side slope of breach and maximum discharge at dam site

Time of breach development(hrs)	Side slope of breach(1: Z_o)	Maximum discharge (m3/s)
2.867	0.175	14,742.45
2.867	0.7	12,758.21
2.867	2.867	10,938.43
2.867	1.75	9789.31

Bottom breach width and maximum discharge at dam site

Time of breach development(hrs)	Bottom breach width of breach(m)	Maximum discharge (m3/s)
2.867	31.06	10,572.94
2.867	124.3	10,655.99
2.867	248.51	10,938.43
2.867	310.64	10,951.81
2.867	497.02	11,176.93
Note: B_b and T_o represents typical values & equal to 248.51& 2.867 hrs respectively		

Effects of time to dam failure and side slope on peak flows at dam

Time of breach development(hrs)	side slope of breach(1:Zo)			
	0.175	0.7	1.4	1.75
	Peak flow(m3/s)			
0.358	18,950.71	18,320.67	17,963.45	17,897.12
1.434	17,389.33	17,218.00	17,080.72	16,914.83
2.867	10,742.45	10,758.21	10,938.43	10,989.31
3.154	7,658.54	7,621.55	7,615.50	7,210.29
4	2,562.37	2,465.61	2,270.26	1,837.03
Note: Z_o and T_o represents typical values &equal to 1.4 & 2.867 hrs respectively				

Downstream peak flows for five TFH breaching outflow hydrographs

River station(Km)	Assumed time to dam failure for inflow hydrographs					Mannining Coeff.
	0.358	1.434	2.867	3.154	4	Nlb,Nc,Nrb
	peak flow(Q_{max})					
0	11891.53	11567.81	10,938.43	10,675.5	10,570.26	0.035,0.03,0.035
15	7103.45	6753.9	6,117.59	6140.57	6,182.26	0.035,0.03,0.035
25	6708.34	6231.01	6,040.73	6298.24	6,144.67	0.035,0.03,0.035
45	6234.87	6123.21	5,408.22	5333.74	5,370.43	0.035,0.03,0.035
64	5821.89	5524.73	5,103.92	5270.43	5,114.32	0.035,0.03,0.035
Note: T_o represents original time to dam failure value equal to 2.867 hrs						

Peak flows at different river stations for five Manning values

River station(Km)	Assumed Manning Coeff.				
	0.02625	0.02811	0.03	0.03375	0.0375
	peak flow(Q_{max})				
0	10,938.43	10,938.43	10,938.43	10,938.43	10,938.43
15	8,762.45	8,382.11	8,117.59	8,098.31	7,898.67
25	7,841.79	7,465.80	7,440.73	7,088.21	7,093.82
45	6,626.50	6,462.38	6,160.22	6,074.90	5,903.20
64	5,652.03	5,442.65	5,353.92	5,100.11	4,991.07
Note: N_o represents original time to dam failure value equal to 0.03					

Peak flows for specified channel bed slopes

River station(Km)	peak flow(m3/s)				
	0.0034	0.00323	0.003315	0.003485	0.00357
0	10,938.43	10,938.43	10,938.43	10,938.43	10,938.43
15	9523.09	9485.09	9499.12	9702.31	9791.39
25	7342.01	7221.07	7289.45	7714.01	7793.9
45	5342.41	5017.09	5120.97	5491.03	5580.13
64	3350.18	3160.05	3191.78	3467.17	3495.92
Note: S_o represents existing or baseline channel bed slope provided by dam owner					