Addis Ababa University

School of Graduate Studies

Addis Ababa Institute of Technology

Dam Breach Analysis & Inundation Map for Melka Wakena 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

Yonatan Sisay Asfaw

SUPERVISED BY: - Dr.Ing. Asie Kemal

October, 2016

Certification

This to Certify That This Thesis Entitled **Dam Breach Analysis & Inundation Map for Melka Wakena Dam,** Done and Submitted

By

Yonatan Sisay Asfaw

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Advisor	Signature
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External Examiner	Signature
Chairman (Department of Graduate Committee)	Signature

ABSTRACT

This study presented dam breach analysis & inundation map for Melka Wakena Dam. Melka Wakena Dam is an earth-rock fill dam with 42m height and 2000m long. The dam was designed to produce Hydropower of 153 MW. In this study, the Melka Wakena Dam breaching outflow hydrographs and downstream flood propagations were simulated by applying computer programs to recognize the possible relationships among the peak flows, dam breach parameters and downstream river parameter.

Dam breach analysis & inundation map involved with reservoir routing and river routing techniques. The key inputs required in the flows routing processes include time to dam failure in hours (TFH), side slope of dam breach (SS), shape of opening, downstream channel geometries, Manning roughness coefficients and inflows to reservoir. The reservoir components of routing were performed applying HEC-RAS computer program. Similarly, the HEC-RAS computer program was applied to conduct unsteady flow routing through the river components of the testing waterways and produced various peak flows for given conditions at specified downstream reach stations.

The maximum breach discharge resulted from HEC-RAS model was 36,527.15m3/s which results in overtopping the dam by 18cm and the maximum breach discharge results for piping 32,627.70m3/s. The influences of dam breach and river parameters on maximum breaching outflow discharges were analyzed at the dam site and downstream stations. Dam breach analysis & inundation map analyses results showed that the maximum discharge and downstream routing. The breach discharge for overtopping maximum than piping.

Key Words: - Dam Break, Inundation Map, Dam Breach Parameters, HEC-RAS.

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CHAPTER ONE

INTRODUCTION

1.1. Background

Dams are an important part of this nation's infrastructure, providing flood control, water supply, irrigation, hydropower, navigation, and recreation benefits. Despite their many beneficial uses and value, dams also present risks to property and life due to their potential to fail and cause catastrophic flooding. Catastrophic flooding occurs when dam fails and the impounded water escapes through the breach into the downstream valley. When dams fail, property damage is certain, 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.

There may be many reasons for dam failures, among them the floods occurring in river basins near existing dams, triggered by intensive rain is the most responsible one. When the flood hydrograph entering a dam reservoir reaches a peak value of unusual magnitude, the amount of water exceeding the capacity of the dam reservoir should be diverted downstream of the dam. If a spillway built for just that function was not designed for that kind of magnitude, excess water may spill over the dam crest. In case this happens, a breach may form in the dam body in minutes or hours depending on the type of material used in the dam body. As this breach gets larger and larger in time, the enormous amount of water stored in the reservoir upstream of the dam may start its motion as an uncontrolled flood wave downstream of the dam. A flood caused by a dam failure may occur in a much bigger magnitude compared to those floods generated by rain or snow melt. In the downstream river bed, the fast moving flood wave with its great power having the potential to destroy whatever comes in its way, may provoke deadly consequences should there be residential areas on its course.

Flood induced by dam breach happens occasionally throughout the world. Floods can induce serious loss of life and significant economic losses. To recognize the possible effects of dam breaks, a detailed knowledge of dam breakage processes and flood propagation is required. Warning time is the most important parameter affecting potential loss of life due to dam failure. Numerical and hydraulic models can be used to predict flood wave propagation and provide the information about the wave front arrival time, area to be flooded and water depth. Therefore, models are useful tool for developing evacuation plans and warning system for areas having potential flood risk. When population centers are located close to dams, accurate prediction of breach parameters is crucial for development of effective emergency action plans. The development of a dam breach is a complex process involving numerous uncertainties.

The outflow flood hydrograph from a dam failure is dependent upon many factors. The primary factors are the physical characteristics of the dam, the volume of the reservoir and the mode of failure. The parameters which control the magnitude of the peak discharge and the shape of the outflow hydrograph include: the breach dimensions; the manner and length of time for the breach to develop; the depth and volume of water stored in the reservoir; and the inflow to the reservoir at the time of failure. The shape and size of the breach and the elapsed time of development of the breach are in turn dependent upon the geometry of the dam, construction materials, and the causal agent for failure.

Ethiopia can be considered as the water tower of Africa because of its high water resources availability. The country has 12 river basins. The total mean annual flow from all the 12 river basins is estimated to be 122 BMC (MoWR, 1999). Construction of dams has been commenced since the first dam was built in 1939 and the dam was constructed on Akaki River to generate hydro-electric power. The dams built so far are being used in order to alleviate the water related problems of the population. However, it should be pointed out at the outset that the development of dams is being threatened by (1) sedimentation problems arising from the degradation of catchment areas fueled by four pressure indicators namely agricultural production, rapid population growth, poverty and wood energy demands, (2) in appropriate runoff estimation methods resulting in over sizing or under-sizing of dams, (3) and unreliable spillway flood estimation methods.

Large dam projects are prone to delays. The dams built in Ethiopia are no exception to the rule and all have been delayed by at least one year. A complex geology has been one of the reasons for the delays, leading to landslides and tunnel collapses. The Gibe II dam has been affected by such problems even after its completion, when a tunnel collapsed and put the hydropower plant out of service for several months. The construction of large dams entails many tangible and intangible costs. The financial cost itself is already substantial. Resettlement adds to the social costs of the dams. Sedimentation from unchecked erosion in the upper watershed of rivers reduces the lifespan of reservoirs. Environmental costs are imposed on communities living downstream of the dams in Ethiopia (Wikipedia, 2015).

The purpose of the dam breach analyses has been to illustrate how the flood wave propagates and attenuates along the river valley from Melka Wakena dam. In the present analyses the HEC-RAS model is used for simulation of the flood wave caused by dam failure. This model is one of the most widely accepted model of its kind.

1.2. Objective

1.2.1. General Objective

The term dam break analysis usually relates to the process of studying a dam failure phenomenon and analyzing the resulting consequences at the downstream region. This generally deals with simulation of assumed failure for existing dams and analyzing the resulting consequences. The prime objective is prediction of the reservoir outflow hydrograph and the routing of that hydrograph through the downstream valley to determine dam failure consequences and to facilitate effective emergency action planning.

1.2.2. Specific Objective

- Prediction of breach parameter.
- Prediction of outflow hydrograph due to dam breach.
- Routing of the hydrograph through the downstream valley using a model.
- Produced inundation map for the downstream.
- Prepare Emergency Action Plan (EAP).

CHAPTER TWO

LITERATURE REVIEW ON DAM BREACH ANALYSIS

2.1.Background

Floods resulting from dam failures led to catastrophic and tragic consequences in the past. Researchers have been working to develop computer programs that would help to design new dams or evaluate existing dams.

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). Some investigators of dam-break flood waves assumed the breach encompasses the entire dam and that it occurs instantaneously. Others, such as Army Corps of Engineers (1960), have recognized the need to assume a partial failure rather than complete breaches.

Several researchers have devolved regression equations to estimate breach size, shape, and time to dam failure from historical dam breach information. A few researchers have tried to develop computer models to simulate the physical breaching process.

2.2.Types of Dames

Dams may be classified by purpose, type, size, and hazard potential, the latter of which varies greatly between States and Federal agencies. This section describes the most common types of dams.

There are numerous intended purposes for man-made dam structures, such as flood retarding, diversionary, irrigation and water supply, hydroelectric power generation, and recreational. Recreation, flood control, and fire protection are the three most common applications. Fire protection, as defined by the NID data dictionary, includes stock ponds and small farm ponds.

The NID classifies dams by the type of construction material used with the majority listed as either a concrete or embankment type dam. Concrete dams include arch, buttress, concrete, gravity, masonry, multi-arch, and roller-compacted concrete (RCC) and are typically constructed of concrete or other masonry components. Embankment dams are made of earthen materials and may be filled with rock, earth, or other materials resistant to erosion.

Concrete Dams

There are several types of concrete dams ranging from conventional design styles such as gravity, arch, multi-arch, and buttress dams to newer design approaches such as RCC dams.

Embankment Dams

Embankment dams are made from compacted earth. There are several types, as shown in Figure 2-1. The two most common types of embankment dams are rock-fill and earth-fill dams.

Earth-fill dams are composed of suitable soils obtained from borrow areas or required excavation that are spread and compacted in layers by mechanical means. Earth-fill dams may be constructed with homogenous layers (homogeneous dam) or zones of different materials of varying characteristics (zoned-earth dam). Earth-fill dams are typically trapezoidal in shape and rely on their weight to hold back the force of water, similarly to concrete gravity dams. Typical zones include a clay core and filter and drain zones.

A unique category of earth-fill embankment dams are tailings dams used by the mining industry. Tailings dams are often constructed of coarse tailings produced by the mine but may also consist of other soils obtained near the construction site. Tailings dams often rely on the stored tailings to control seepage, but otherwise include many of the same design features as conventional water storage dams.

Rock-fill dams are constructed from compacted earth fill that contains a high percentage of rocks and other larger particles. The fill typically drains easily and therefore no drainage layer is required. To prevent seepage, rock-fill dams have an impervious zone on the upstream side of the dam or within the embankment. The impervious zone can be made from a variety of materials including masonry, concrete, plastic, steel pile sheets, timber, or clay. If clay is used, it is often separated from the fill by a filter to prevent erosion of the clay into the fill material.

Earth-fill dams may include a water-tight core can also be made from asphalt concrete. Dams with this type of core are called concrete-asphalt core embankment dams. Most concrete-asphalt dams use rock and/or gravel as the main fill material. These types of dams are considered especially appropriate for areas susceptible to earthquakes due to the flexible nature of the asphalt core.

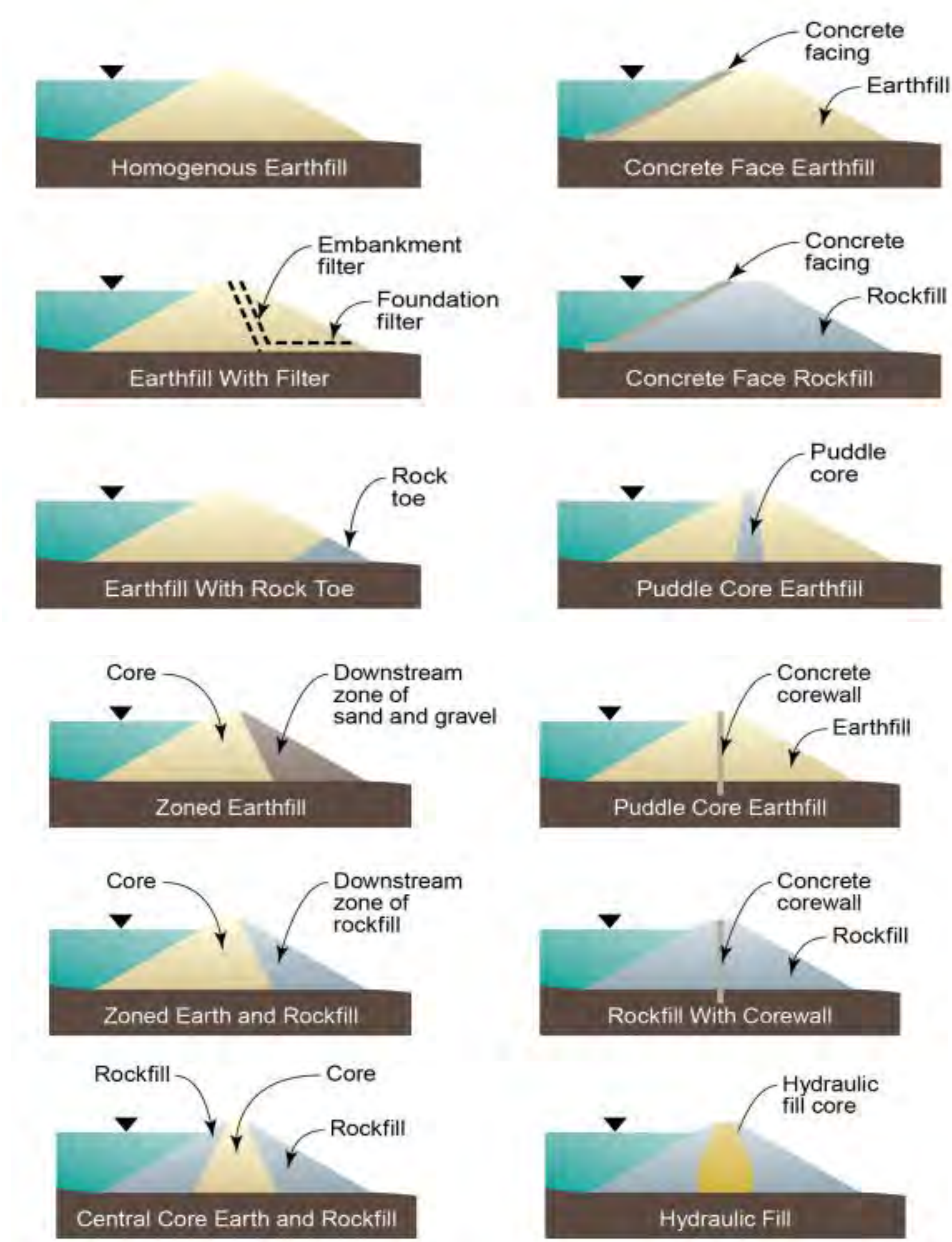


Figure 2.1: Types of embankment dams

2.3.Dam Breach Analysis Purpose

In the context of risk informed decision making, dam breach analyses are needed for determining the potential consequences of a failure mode's occurrence over a range of loading conditions. It can also be used as part of a dam's remedial design process in the selection of alternatives. The type of analysis as well as the level of accuracy required by the results must be scalable to the potential hazards and complexity of the downstream area being modeled. For risk informed decision making, the dam breach parameters are based on best estimates from similar case studies considering the range of possible values associated with the potential failure mode's specifics and the dam's characteristics.

The results of dam breach analyses are typically tabulated in spreadsheet form and plotted on inundation maps of sufficient detail to understand the potential consequences associated with life loss and economics. These can then be used to formulate estimates of the potential for loss of human life and the economic impacts of resulting damages; however, analysis of social and environmental impacts, damage to national security installations, and political and legal ramifications (which are not easily evaluated and are based on subjective or qualitative evaluation) may be required.

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 depending on the type of dam. For instance, concrete gravity dams tend to have a partial breach, as one or more monolith sections formed during dam construction fail, whereas concrete arch dams tend to fail suddenly and completely (Canadian Dam Association, 2007b). In contrast, embankment dams do not usually have a complete or sudden failure, but rather tend to breach to the point where the reservoir is depleted or to where the breached materials resist erosion, such as at the dam foundation.

The most common causes of dam failure between January 1975 and January 2011are summarized in Table 2-1. Flood or overtopping was the most common cause of dam failure, followed by piping or seepage.

Table 2.1: Causes of Dam Failure 1975-2011

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

2.5.Dam Failure Examples

The following examples of dam failures illustrate that dams fail for a number of causes. These examples illustrate the need for dam breach inundation modeling and mapping to identify the flood risk and the importance of developing EAPs to mitigate the potential loss of life and other losses that can result from dam failure incidences.

Flooding and overtopping failure: - The Kaloko Reservoir Dam in Kauai, HI, failed due to overtopping during an extreme rain event in March 2006. This earth dam was built in 1890 with a storage capacity of about 420 million gallons. The embankment had a maximum height of about 40 feet. The dam crest was about 770 feet long and 15 feet wide. There were seven deaths reported due to this dam failure.

The southern embankment of the Lake Delhi Dam in Delhi, IA, failed on July 24, 2010, due to heavy raining and flooding. The dam failed after receiving about 10 inches of rainfall in 12 hours. Before the breach, river levels upstream of the dam had reached 24.22 feet, 10 feet above flood stage, breaking the May 2004 record of 21.66 feet.

Piping and seepage failure: - In 1976, the failure of the Teton Dam in Idaho led to flooding in the cities of Sugar City and Rexburg (Figure 2-2). The dam failure killed 14 people and caused over $1 billion in property damage. Over 2,000,000 cubic feet per second of sediment-filled water emptied through the breach into the remaining 6 miles of the Teton River canyon, after which the flood spread out and swallowed on the Snake River Plain. Study of the dam's environment and structure placed blame for the collapse on cracks in the permeable soil (loess) used in the core and on cracks in the foundation bedrock that allowed water to seep under the dam. The combination of these flaws is believed to have allowed water to seep through the dam, which led to internal erosion, called piping, which eventually caused the dam's collapse.

Figure 2.2: Teton Dam failure, Rexburg, ID, June 1976

Structural failure: - The Kingston Plant coal waste dam failed in Harriman, TN, on December 22, 2008. This was a 40-acre pond used by the Tennessee Valley Authority to hold slurry generated by the coal-burning Kingston Steam Plant. The dam gave way just before 1 a.m., burying a road and railroad tracks leading to the plant. Although no one was seriously injured or hospitalized, 5.4 million cubic yards (> 1 billion gallons) of sludge damaged 12 homes and covered hundreds of acres.

Spillway gate failure: - A spillway gate of Folsom Dam in California failed in 1995, increasing flows into the American River significantly. The spillway was repaired and the USBR carried out an investigation of the water flow patterns around the spillway using numerical modeling,

Earthquake failure: - The Lower San Fernando Dam in California failed during an earthquake in 1971, causing the fill in the dam wall to liquefy which resulted in the collapse of the upstream part of the dam. A disastrous flood was only prevented because the reservoir level happened to be low at the time of the earthquake and no water escaped downstream.

Poor design/construction failure:- In August 2008, the Redlands Ranch Dam located in Havasu, AZ, failed due to neglect and poor design and construction. No loss of life was reported, but 426 people were evacuated by helicopter and there was significant damage to the landscape.

2.6.Dam Breach Analysis Study Approaches

The two primary dam breach study approaches used by State governments and Federal agencies are an event-based approach and a risk-based approach. The event-based approach has been traditionally the most widely used for dam breach analysis. The event-based approach is a deterministic method based on specific precipitation and non-precipitation events for the dam breach analysis 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).

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

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.

2.6.1. Event-Based Approach

An event-based approach is a deterministic method that requires the use of a specific or series of specific precipitation and non-precipitation events for the evaluation of dam failure and downstream inundation mapping. These events include extreme rainfall and runoff events that can lead to natural floods of variable magnitude. The maximum flood for which a dam is to be designed or evaluated is often dependent on its existing hazard potential classification or size classification (FEMA, 2013).

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. Typically, the hazard potential classification of the dam is used to select the extreme hydrologic failure event. The PMF is the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the drainage basin under study. The Probable Maximum Precipitation (PMP) is an estimate of the maximum possible precipitation depth over a given size catchment for a given length of time (Stedinger et al. 1996).

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

2.6.1.1. Fair Weather (Non-Hydrologic) Failure

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

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

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

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

Invert of Auxiliary Spillway (lowest uncontrolled spillway)

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

Top of Dam / Maximum High Pool

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

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

Table 2-2. provides the recommended water surface elevation of a reservoir for used in dam breach modeling based on published documents from Federal agencies and dam safety resource groups. The normal pool elevation is recommended as the default volume for the fair weather failure. States should consider a larger storage volume for dams where the primary and emergency spillway systems are considered susceptible to blockage resulting in a higher water surface elevation and volume during a non-hydrologic event.

Table 2.2: Range of Initial Reservoir Pool Levels for a Fair Weather (Non-Hydrologic) Analysis (FEMA, 2013)

Initial Reservoir WSEL	Referenced Name in Publication	Initial Inflow to Reservoir	Failure Mode	Supporting Federal Organization	Supporting Documentation
Normal pool	Normal full reservoir(1)	Normal stream flow	None specified	Federal Emergency Management Agency (FEMA)	Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams. pp. 17. 2004b.
	Normal full reservoir	Normal stream flow	None specified	Federal Energy Regulatory Commission (FERC)	Engineering Guidelines for the Evaluation of Hydropower Plants. Ch 2. pp. 2-7. 1993.
	Normal pool elevation	Normal stream flow	Piping	Mine Safety and Health Administratio n (MSHA)	Engineering and Design Manual: Coal Refuse Disposal Facilities. 2009.
	Normal pool elevation(2)	Normal stream flow	Piping	National Dam Safety Review Board (NDSRB)	Simplified Inundation Maps for Emergency Action Plans. 2009.
	Top of active conservation	Normal stream flow	Piping	U.S. Department of the Interior (USDOI)	Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report. 2011.
	Seismic	Normal stream flow	Piping/inte rnal erosion	Natural Resources Conservation	National Engineering Manual: 210-V. 1982.

Invert of auxiliary spillway				Service (NRCS)	
	Top of joint use(3) (auxiliary spillway)	Normal stream flow	Piping	USDOI	Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report. 2011.
Between normal pool and top of dam	Hydrologically induced static failure	Hydrologic event	Below top of dam (piping); above top of dam (overflow breach)	USDOI	Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report. 2011.
Top of dam	Static	Normal stream flow	Not specified	NRCS	National Engineering Manual: 210-V. 1982.
Other	Seismic	Not specified	Catastrophic failure or overtopping (caused by liquefaction) and seismic-induced piping	USDOI	Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report. 2011.
	Normal Low Pool: 90% exceedance duration pool elevation	Constant inflow as required to produce the 90% exceedance duration pool	Piping	U.S. Army Corps of Engineers (USACE)	Modeling, mapping, and Consequences Production Center Standard Operating Procedures (Final-Draft). Unpublished draft, 2011.
	Normal High Pool: 10% exceedance	Constant inflow as required to produce the	Piping	USACE	Modeling, mapping, and Consequences Production Center Standard Operating

	duration pool elevation	10% exceedance duration pool			Procedures (Final-Draft). 2011.

WSEL = Water Surface Elevation

(1) Normal reservoir level: “For a reservoir with a fixed overflow sill the lowest crest level of that sill. For a reservoir whose outflow is controlled wholly or partly by moveable gates, siphons or other means, it is the maximum level to which water may rise under normal operating conditions, exclusive of any provision for flood surcharge”.

(2) For small and intermediate-sized dams, it may be appropriate to use a single fair weather failure with the initial elevation set to the top of the dam instead of the rainy and fair weather situations. This “eliminates the need for expensive watershed and spillway studies and provides a reasonable upper limit estimate for warning and evacuation”

(3) Joint use is a designation for dams with gated spillways. In these cases, the top of joint use is not the invert of the spillway but rather some elevation that places water up on the gates.

2.6.1.2. Hydrologic Failure

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

2.6.2. Risk-Based (Consequences-Based) Approach

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

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

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

2.6.2.1. Inflow Design Flood and the Incremental Hazard Evaluation

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" (FEMA, 2004). Therefore, incremental hazard evaluation and the establishment of the IDF is, in essence, a risk-based approach.

The selection of the IDF is based on the evaluation of the magnitude of several flood events (FEMA, 2013). The incremental hazard evaluation begins with simulation of a dam failure during a hydrologic flooding condition, typically beginning with the PMF or percentage of the PMF as specified by the State hazard potential classification requirements. The same hydrologic event is then run for non-failure conditions. The water surface elevations for both the breach and non-breach events are compared to determine the increase in the water surface elevation resulting from the dam breach. If the incremental increase in downstream water surface elevation between the failure and non-failure scenarios results in an acceptable increase in consequences, (as defined by State requirements) a smaller percentage of the PMF flood inflow or other magnitude flood is then used to repeat the process. The process is repeated until the incremental increase in consequences due to failure falls within acceptable requirements.

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

New guidance in Selecting and Accommodating Inflow Design Floods for Dams includes the recommendations shown in Table 2-3.

Table 2.3: Recommended IDF Requirements for Dams Using Prescriptive App. (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.

2.6.2.2. Loss of Life / Population at Risk

It is important that consistent approaches for consequence estimation be adopted across the dam-safety sector. FEMA's Estimating Loss of Life for Dam Failure Scenarios discusses the strengths and limitations of several methods for estimating loss of life. This section further describes the procedure described in (USBR 1986, 1999) as it is the most currently and widely used procedure for estimating loss of life resulting from dam failure.

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 factors: 1) the number of people occupying the dam failure floodplain, 2) the amount of warning provided to the people exposed to dangerous flooding, and 3) the severity of the flooding (FEMA, 2013). Without exception, dam failures that have caused high fatality rates were those in which residences were destroyed and timely dam failure warnings were not issued. Estimating when dam failure warnings would be initiated is probably the most important part of estimating the loss of life that would result from dam failure**.**

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

2.6.3. 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 tiered dam breach analysis approach presented in this document adapts the USDOI approach and provides additional detail.

The NDRSB EAP Workgroup (2009) noted that the cost of detailed dam breach studies is consistently cited as the primary impediment to EAP development and, therefore, many States have adopted a form of simplified and conservative inundation maps for use in EAPs. 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. 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 2-2. Provides guidance to determine the tier level for analysis for dam failure inundation modeling and mapping. 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 2.4: 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	• Low-hazard potential / small size • First level screening for significant- or high-hazard dams	Empirical Equations	Simplified Models (SMPDBK, GeoDam-BREACH, or Technical Release [TR]-66) or HEC-HMS	GeoDam-BREACH, SMPDBK, DSAT,1D HEC-RAS Steady State, or HEC-HMS Hydrologic Routing
Tier 2 – Intermediate	• Significant-hazard potential / intermediate size • High-hazard dams with limited population at risk	Empirical Equations	HEC-HMS or HEC-RAS Unsteady Model	HEC-RAS (Steady or Unsteady Modeling) 1-D or 2-D models
Tier 3 – Advanced	• High-hazard potential / large size dams with sufficient population at risk to justify advanced analyses	Empirical Equations, NWS BREACH, or WinDAM	HEC-RAS Unsteady Model	HEC-RAS Unsteady Model or 2-D models

Tier 1 and 2 analyses are most appropriate for low-hazard potential / small sized and significant-hazard potential / intermediate-sized dams with a limited number of structures. More detailed surveying or modeling may be warranted for Tier 3 analyses for high-hazard potential / large-sized dams, those with a large population in the evacuation area, or those with significant downstream hydraulic complexities

2.7. Dam Breach Parameter

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

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

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

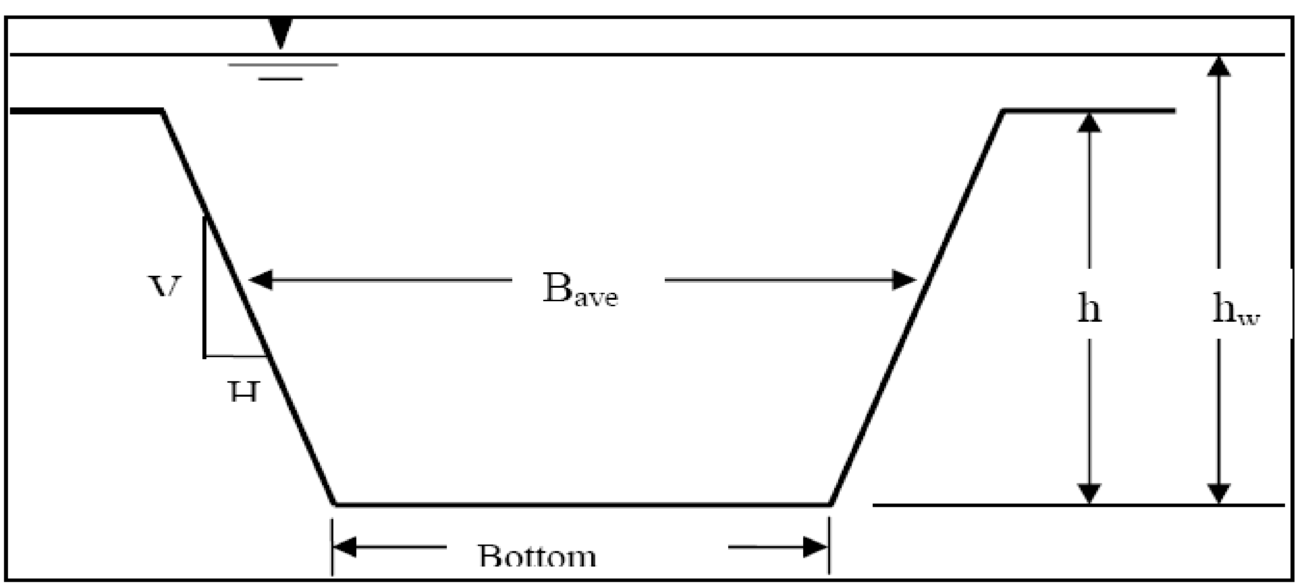
2.7.1. Dam Breach Parameter Definitions

For the purposes of this discussion, the term breach parameters will include the parameters needed to physically describe the breach (breach depth, breach width, and side slope angles) as well as parameters that define the time required for breach initiation and development. The physical parameters are shown graphically in Figure 1 and are briefly summarized below.

- **Breach depth** - 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. Some publications cite the reservoir head on the breach, measured from the reservoir water surface to the breach invert.

- **Breach width** - 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. Case studies typically report either the average breach width or the breach width at the top and bottom of the breach opening.

- **Breach side slope factor** - The breach side slope factor along with the breach width and depth fully specifies the shape of the breach opening. Accurately predicting the breach side slope angles is generally of secondary importance to predicting the breach width and depth.



The breach width is described as the average breach width (B_{ave}) in several of the empirical equations. The breach height (h_b) is the vertical extent from the top of the dam to the invert elevation of the breach. Many publications and equations also use the height of the water (h_w), which is the vertical extent from the maximum water surface to the invert elevation of the breach

When breach formation times are reported in case studies, there is often some question as to whether the reported times are only for the breach formation phase, or if they might also include some portion of the breach initiation phase. Distinguishing between the two during (or after) a failure is a difficult task, even for a trained observer. In the interest of promoting more accurate reporting of breach initiation and breach formation times, the following definitions are offered:

Breach initiation time - The breach initiation time begins with the first flow over or through a dam that will initiate warning, evacuation, or heightened awareness of the potential for dam failure. The breach initiation time ends at the start of the breach formation phase (see next item).

Breach formation time - The duration of time between the first breaching of the upstream face of the dam until the breach is fully formed. For overtopping failures the beginning of breach formation is after the downstream face of the dam has eroded away and the resulting crevasse has progressed back across the width of the dam crest to reach the upstream face.

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

2.8. Breach Mechanisms for Embankment Dams

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.

Overtopping Failures

Overtopping failures can occur very differently depending on the composition of the dam. Perhaps the simplest overtopping failure to discuss is failure of a cohesive soil embankment. According to a study by Ralston (1987), a small head cut typically forms on the downstream face of a cohesive soil embankment and progresses upstream as shown in Figure 2.3.

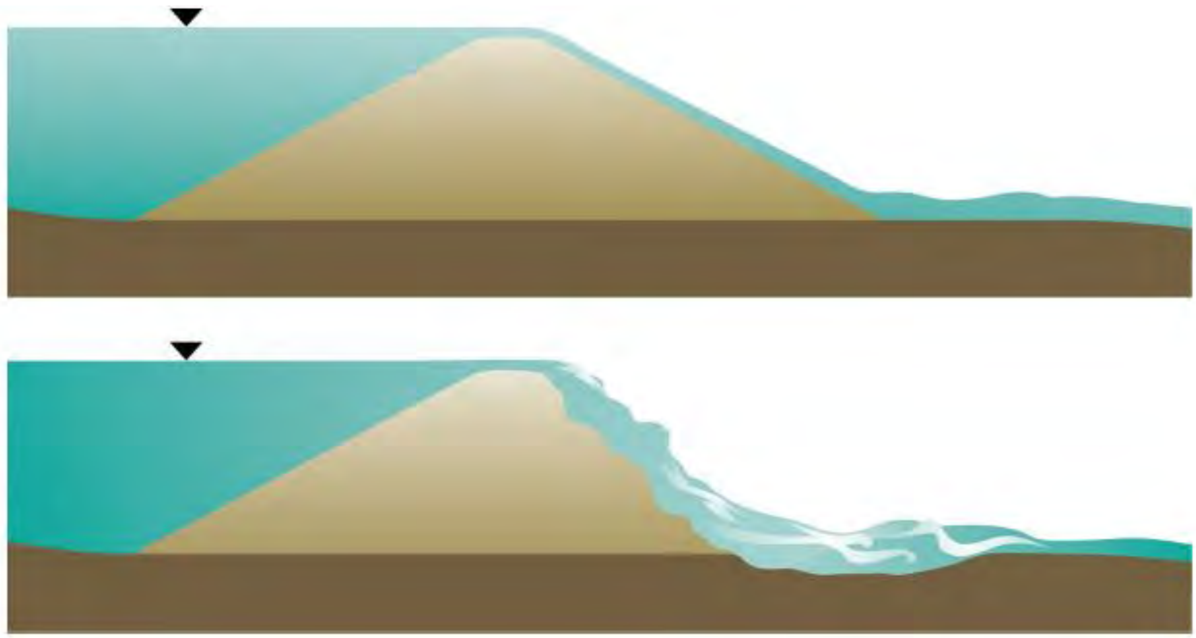


Figure 2.3: Erosion on the downstream face of a cohesive soil embankment dam

The breach is considered to begin when erosion occurs across the width of the dam crest. After the breach initiates at the top of the dam crest, it enlarges to its ultimate extent. If there is no physical reason to believe the embankment would fail at a certain location, the breach should be modeled as initiating at the maximum section typically located at the centerline of the downstream main channel. A generalized trapezoidal breach progression is illustrated in Figure 2.4.

Figure 2.4: Overtopping trapezoidal breach progression

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

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

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

Piping / Internal Erosion Failures

Piping and internal erosion occurs when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam, leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid. Water flow through the embankment will appear muddy as erosion increases. Once the erosion reaches the reservoir, the piping hole can enlarge and cause the dam crest to collapse. Figure 2.5 shows a schematic of a fully formed piping hole.

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

There are several possible options to identify the breach initiation time. For breaches associated with a hydrologic event, the initiation can be considered to begin when the reservoir water level reaches a certain elevation or after the water level has exceeded a certain elevation for a specified Duration. For fair weather breach analysis, an initiation time should be specified regardless of pool elevation (Gee, 2010).

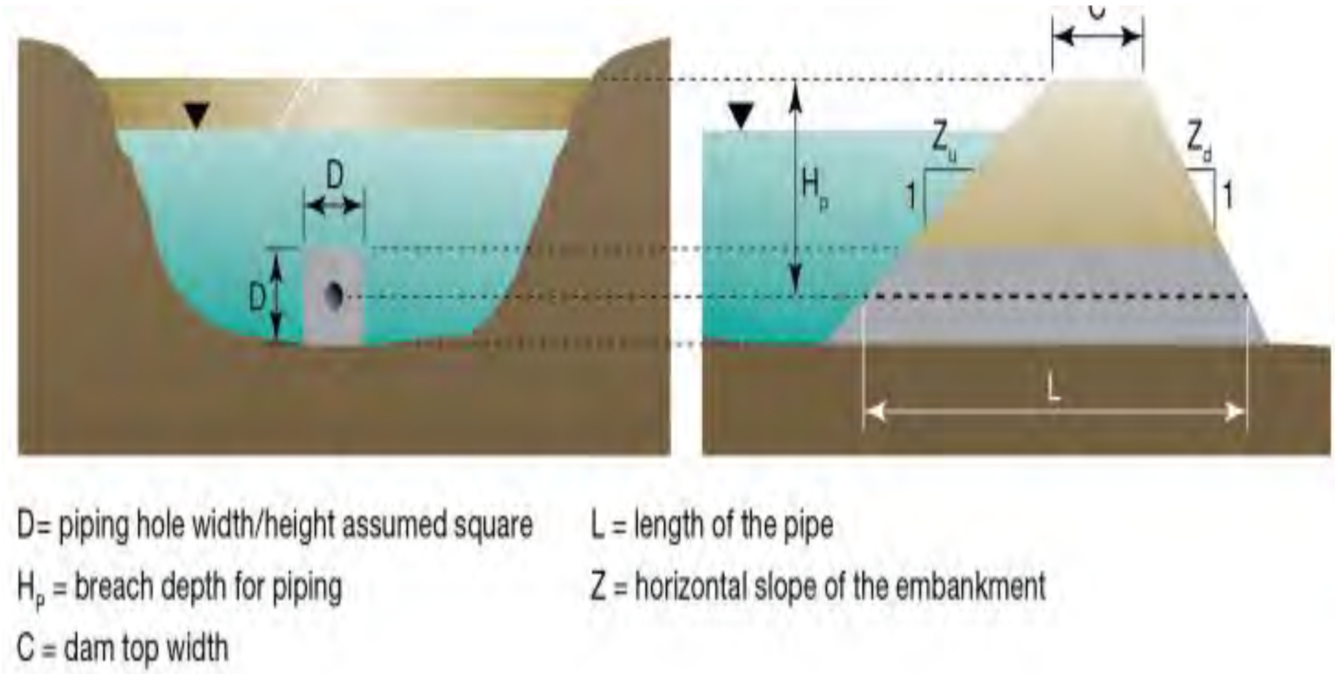


Figure 2.5: Schematic of piping hole

2.9. Available Approaches

Most methods are either based on

I. case study data from past failures or on
II. Physical models that do not account for the actual true erosion mechanism and flow regimes that a dam breach may face.

Here is a discussion about these two methods:

I. Case study methods are not very accurate because they are mostly based on small database of failed dams, especially of small dams. Case study data are not good in predicting the initiation time of a breach, breach's rate formation, and the total time required for failure. But under case studies, there are 3 methods:
   - parametric models: they first predict time of failure and ultimate breach geometry and compute breach outflows using hydraulics principles, second they simulate breach growth as being time dependent
   - predictor equations: these equations are most of the time empirical and they estimate peak discharge based on case study data
   - analysis by comparison: if the dam under study has characteristics similar to that of another actual failed dam with a well-documented failure, the breach characteristics and hydrograph can be determined by comparison. In other words, this method neglects the

process of breaching and is only based on comparison with a similar breached dam(comparative methods)

II. Physically based models such as BREACH give more extensive information but suffer from their limited accuracy ([16], p.5). The current models are mostly based on geotechnical concepts and sediment transport relations that are not applicable or are not well tested on dam's breach

Other physical models like DAMBRK simulate the breach of the dam and the resulting reservoir outflow. The geometry and time of formation of the breach should be given to this program as an input, and the output will give the breach enlargement as function of time (e.g., linear increase of breach dimensions). The required input parameters should be found from either comparative methods or from prediction equations or other physical models.

Problems of These Approaches

The problems of these three approaches are:

- Comparative analysis: this analysis is only appropriate to small dams, because most case studies in this approach are based on small dams.
- Predictor equations: the same restriction of the comparative analysis applies for the predictor equation method. Therefore the regression relations based on the available data have high uncertainty.
- Physical models: the main flaws from which this method suffers are due to insufficient understanding of breach development; breach and high erosion dominating dam breach.

Breach characteristics

When a small variation in one of the breach parameters (width, depth, failure time and overtopping head) occurs, large changes in peak flows will take place especially for reservoirs with relatively small storage. In 1984, Singh and Snorrason used some models such as DAMBRK and HEC-1 on 8 hypothetical breached dams to assess which breach parameter affects mostly the peak outflow.

Failure Time

They found that if failure time were reduced by half its initial value, the peak outflow for a PMF hydrograph would increase by 13 to 83 %. But for large reservoirs, the change in peak outflow was much smaller showing a variation of only 1 to 5 %.

Breach Width

It seems that the changes in breach width is more effective for large dams because it produced larger changes (35-87%) in peak outflow and smaller changes (6-50%) for small reservoirs.

Breach Depth

If breach depth is changed, little change in peak outflow has been identified, leading to the conclusion that the change in peak flow is not really dependent on the reservoir size.

Other studies conducted by Petrascheck and Sydler (1984) also proved that change in the breach width and breach formation time would significantly affect the outflow peak discharge, inundation levels, and flood arrival time. For locations not far from the dam, both breach width and breach formation time will have a great influence.

Some critical results have been found by Wurbs(1987). In large reservoirs, the peak outflow takes place at the moment when the maximum depth and width of the breach are attained. Changes in reservoir head are relatively slight during the breach formation period. In small reservoirs, a huge change in the level of the reservoir takes place during the formation of the breach; consequently the peak outflow occurs sometime before reaching the final breach. Here, the formation rate of breach is crucial.

2.9.1. Empirical Models for Predicting Breach Parameters

Using case study data, many researchers developed formulas that enabled them to predict breach parameters like time of breach formation and breach geometry. In the following, a discussion concerning each method is given

Johnson and Illes (1976)

They were the first to predict failure shapes for earth, gravity, and arch concrete dams. For earth dams, their proposition was that the breach shape begins as a triangle and ends as a trapezoid. They also realized that failure width (general) B is given by:

$$0.5h_d < B < 3h_d \text{ for earth fill dams}$$

Where:

h_d=dam height (m)

Most other studies assume that the breach shape of earthen dam is trapezoidal.

Singh and Snorrason (1982, 84)

Their study was conducted on 20 case studies and they came up with the following. The breach width is constrained by:

$$2h_d < B < 5h_d$$

Where:

B=breach width (m)

h_d=dam height (m)

$$0.15 \text{ m} < d_{ovtopp} < 0.61 \text{ m}$$

Where:

$d_{overtopp}$ = the maximum overtopping height above the crest of the dam before failure

$$0.25\text{hr} < t_f < 1.0 \text{ hr}$$

Where:

t_f =failure time (hr)

MacDonald and Langridge-Monopolis (1984)

Based on 42 case studies, they suggested that most of the breach side slope are approximately 1h: 2v and that the breach shape could be trapezoidal or triangular and this depends on whether the breach has reached the bottom of the dam or not. They also estimated the quantity of eroded embankment materials V_{er} (m^3) for earth dams based on time of failure t_f.

$$V_{er} = 0.0261(V_{ou}t*h_w)^{0.769}$$

Where:

V_{out}=volume of water discharged through breach (m^3)

h_w = hydraulic depth of water at dam at failure above breach bottom (m)

$$t_f = 0.0179(V_{er})^{0.364}$$

Where:

t_f =failure time (hr)

V_{er} =volume of water discharged through breach (m^3)

On the other hand, for non-earth fill dams they came up only with estimation for volume of eroded embankment material V_{er}.

$$V_{er} = 0.00348(V_{out}*h_w)^{0.852}$$

Where:

V_{er} =volume of water discharged through breach (m^3)

h_w = hydraulic depth of water at dam at failure above breach bottom (m)

They could not predict the failure time for non-earth fill dams because sometimes the Failure of such dams may be caused by structural problems instead of erosion. They also found it crucial that the estimation of breach parameters and outflows should be conducted using several iterations.

FERC (1987)

FERC proposed usually

$$2h_d < B < 4h_d$$

But B can range

$$h_d < B < 5h_d$$

Where:

B=is the breach width (m)

h_d=dam height (m)

$$0.25 < Z < 1 \text{ (engineered, compacted dams)}$$

$$1 < Z < 2 \text{ (non-engineered, slag or refuse dams)}$$

Where:

Z =horizontal side slope factor (Z horizontal: 1vertical) for breach opening

$$0.1 < t_f < 1 \text{ hours (engineered, compacted earth dam)}$$

$$0.1 < t_f < 0.5 \text{ hours (non-engineered, poorly compacted)}$$

Where:

t_f=failure time (hr)

Froehlich (1987, 1995)

In his research, he used 43 case studies. He used no dimensional analysis in order to create equations that estimate the average breach width, side slope and the time of failure. These equations are:

$$B_{avg}= 0.47K_0(S)^{0.25}$$

Where:

B_{avg} is the no dimensional average width= $(B_{top}+B_{bottom})/(2h_b)$

h_b=height of breach (m) and

S=dimensionless storage=(S/h_b^3)

K_o = constant=1.4 if there is overtopping, else 1

$$Z=0.075\ K_c(h_w)^{1.57}(W_{avg})^{0.73}$$

Where:

Z = is the side slope factor,

h_w=dimensionless height of water above breach bottom (h_w/h_b)

W_{avg}=average dimensionless embankment width= $(W_{crest}+W_{bottom})/\ (2/h_b)$

K_c = constant=0.6 if there is a core or 1.0 if no core is present

$$t_f = 0.79(S)^{0.47}$$

Where:

t_f =dimensionless breach formation time=$t_f/(gh_b)^{0.5}$

These equations were based on very specific dam characteristics like the presence of core, height of water above breach bottom, the extent of overtopping and so on. He also realized that overtopping causes the most breach extension and erode at a higher rate than any other failure mode.

In 1995, 8 years after his first study, he published new and revised equations based now on 63 case studies. This time, the new equations are not non dimensional. These equations have better estimated coefficients. These new equations are:

$$B_{avg}\ (m) = 0.1803 K_0 V_w^{0.32}\ h_b^{0.19}$$

Where:

K_0=constant=1.4 if there is overtopping and 1 if else.

$$t_f = 0.000254 V_w^{0.53}\ h_b^{(-0.9)}$$

Where:

t_f =failure time (hr)

Z=1.4 if there is overtopping, if not Z=0.9

Reclamation (1988)

They develop these equations for earthen dams where:

$$B = 3h_w$$

t_f (hours) = 0.011B and B is in meters

Where:

h_w =height measured from the initial reservoir water level to the breach bottom elevation which is assumed to be the streambed elevation at the toe of the dam.

t_f =failure time (hr)

B=is the breach width (m)

Reclamation uses these formulas in the SMPDBK model. The suggested formulas are conservative, and thus they represent a factor of safety for the hazard classification procedure.

Singh and Scarlatos (1988)

Their study is based on 52 case studies. They found that the top widths 106% to 174% larger than the bottom width with an average of 129% and an acceptable standard deviation of 18 %. Whereas, they found that the ratio of the top breach width to dam height was widely distributed. The breach side slopes were inclined40o to 80o with the horizontal. Moreover, most failure times were less than 3 hours.

Von Thun and Gillette (1990)

They have used the data of Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) in order to develop some breach parameters ([32]; [16], p.15). In their work, they assumed that side slopes of breach are 1H: 1V except for dams that have cohesive shells or very wide cohesive cores, where slopes of 1:2 or 1:3 (H: V) are more acceptable.

The relation proposed by Von Thun and Gillette is for the average breach width, and it is given by.

$$B_{avg}\ (m) = 2.5h_w + C_b$$

Where:

B_{avg}= average breach width (m)

h_w = the depth of water at the dam at the time of failure,

C_b = is dependent on the reservoir storage (see table 2-6):

Table 2.5: Values of C_b according to the reservoir size

Size of reservoir (m^3)	**C_b (m)**
<1,23*106	6.1
1.23*106-6.17*106	18.3
6.17*106-1.23*107	42.7
>1.23*107	54.9

They plotted the volume of the eroded embankment versus water outflow volume and water depth above the breach invert, with upper bounds of reasonable breach geometry estimates. These methods are dependent on the amount of erosion that occurs:

$$t_f\ (hr) = 0.020h_w + 0.25 \text{ (erosion resistant)}$$

$$t_f\ (hr) = 0.015h_w \text{ (easily erodible)}$$

Where:

t_f should be in hours

$h_{w=}$ the depth of water at the dam at the time of failure (m)

Moreover, they have suggested other equations that estimate the time of failure using the average lateral erosion rate (the ratio of the final breach width to breach formation time) and depth of water above the breach invert. They conclude that there is abettor estimation using these equations than the first ones that they developed. These new equations are:

$$t_f = \frac{B_{avg}}{4h_W} \quad \text{(Erosion resistant)}$$

$$t_f = \frac{B_{avg}}{(4h_w + 61)} \quad \text{(Highly erodible)}$$

2.9.2. Empirical Models for Predicting Breach Outflows

Some other researchers have conducted studies to determine the peak outflow as a function of the breach parameters (dam height, reservoir storage volume). A discussion of each of these methods is given below.

Kirkpatrick (1977)

Using data from 13 failed embankment dams and 6 other hypothetical failures; he related the peak flow versus the depth of water behind the dam at failure. This equation is written as:

$Q_P = f\ (h_W)$

Where:

Q_P = peak flow (m^3/s)

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

But the flaw of this method is that among the case study failures he used is the St.Francis Dam in California, which was a concrete gravity dam.

SCS (1981)

The Soil Conservation Service used the 13 cases studied by Kirkpatrick in order to develop another method, for earth dam, that relates the peak dam failure outflow to the depth of water at the dam at the time of failure. The equation is given by:

When $H_w > 31.4$ m

$Q_p = 16.6 H_w^{1.85}$ ……………………………………………… (1)

Where:

H_w = the height of water directly at the reservoir before breach measured from the bottom of the final breach.

Q_p = peak outflow through the breach (m^3/s).

When $H_w < 31.4m$

$Q_p = 0.000421\ (V_w H_w / WH)^{1.35}$ ……………………………… (2)

Where:

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

W=average width from the bottom of the final breach to the top of the embankment (m)

H=distance from the bottom of the final breach to the top of the embankment (m)

But the flow calculated in (2) should not exceed the value given by (1) and not less than

$Q_p = 1.77 H_w^{2.5}$

Where:

H_w = the height of water directly at the reservoir before breach measured from the bottom of the final breach (m)

Q_p = peak outflow through the breach (m^3/s).

From the plot of the results of this method with that of the observed flows, it appears that there is a good matching between calculated and measured peak flows except at the low peak flows.

The problem of this method is that it does not provide a way for determining a peak outflow that provides a factor of safety when evaluating downstream flooding.

Reclamation (1982)

Used the work done by SCS and proposed a similar envelope equation for peak breach outflow using case study data from 21 failed dams.

Singh and Snorrason (1982 and 1984)

They established methods relating the peak outflow to the dam height and stored water in the reservoir. These relations were found using the results of eight simulated dam failures analyzed using DAMBRK and HEC-1. Therefore these equations were developed using simulation.

MacDonald and Langridge-Monopolis (1984)

They did a best-fit analysis and boundary curves on 42 failed earth dams in order to determine peak outflow. The developed equation is:

$Q_p = 3.85 (V_w H_w)^{0.41}$

Where:

Q_p = peak outflow through the breach (m^3/s).

V_w= the total quantity of stored water at failure (m^3)

H_w = the hydraulic height of water directly at the reservoir before breach. This formula will exaggerate the peak flow for embankment dams (m)

They have also tried to establish similar relations on non-earthen dams, but this attempt did not succeed because the standard deviation of the data was large.

Costa (1985)

This method is mainly based on regression analysis. It applies for both embankment and concrete dams, because the 31 cases studied to develop this method were a mix of both embankment and concrete dams. The peak outflow is given by:

$$Q_p = 0.763(V_w H_w)^{0.42}$$

Where:

Q_p = peak outflow through the breach (m3/s).

V_w= the total quantity of stored water at failure (m^3)

H_w = the hydraulic height of water directly at the reservoir before breach. This formula will exaggerate the peak flow for embankment dams (m)

But this formula overestimates the peak outflow for the embankment dams because a concrete dam will have bigger breach than a similar embankment dam having the same volume.

Froehlich (1995)

The equation is found by running a multiple linear regression on 22 dams where discharge data were available. This equation is given by:

$$Q_p = 0.607 V_w 0.295 H_w^{1.24}$$

Where:

Q_p = peak outflow through the breach (m^3/s).

V_w= the total quantity of stored water at failure (m^3)

H_w= the hydraulic height of water directly at the reservoir before breach. This formula will exaggerate the peak flow for embankment dams (m)

This equation gives a good agreement with the measured computed peak flows over the entire range.

2.10. Overview of Dam Breach Hydrograph Model

Performing a dam breach model involves prediction of the dam breach hydrograph and the routing of that hydrograph downstream. 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 can be divided into two categories, each of which has a number of models, tools, or equations, ranging from simple to advanced:

- Tools that generate the dam breach peak discharge and/or hydrograph only; and
- Tools that develop a breach hydrograph and perform downstream flood routing

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. Most simplified models do not consider complicated downstream conditions such as backwater effects. Additionally, reservoir routing (if present) uses level pool routing methods; in other words, the reservoir water surface is considered level during drawdown. This simplification is not applicable to all situations. The main benefit of simplified numerical models is that substantially less time is required to set up and execute these models.

2.10.1. Dam breach hydrograph and peak outflow generation tools

The most common methods for either breach hydrograph generation or dam breach peak outflow computation are discussed in this section. These models/methods do not include the capability of a hydraulic routing of the breach hydrograph downstream. The NWS BREACH model is no longer supported by the NWS.

WinDAM B

The ARS recently developed WinDam B in cooperation with the NRCS and Kansas State University, which expands on the capabilities of WinDam A.

NWS BREACH model

The NWS BREACH model was the first widely applied and most well-known, physically based model to predict the breach characteristics and the discharge hydrograph emanating from a breached earthen dam. Since 2005, the NWS has not supported code development; however, given the model's significance in dam breach studies and its ongoing use for some dam breach studies, a description of the model is included in this document. The model was initially developed in 1987 with updates in 1988, 1991, and 2005. The BREACH program is no longer supported by the NWS and is not available for download on the NWS Web site. It is still used because it is known to more accurately predict breach progression than other available methods and perhaps because it has not yet been replaced by another freely available, non-proprietary program that performs the same function.

BREACH couples the conservation of mass of the reservoir inflow, spillway outflow, and breach outflow with the sediment transport capacity of the unsteady uniform flow along an erosion-formed breach. The growth of the breach, as shown in Figure 4-6, is dependent on the dam's material properties and the assumed location of the downstream face of the dam. Sediment transport equations are used in the model to compute the rate of erosion and size of a breach based on supplied soil characteristics of the dam material and the inflow hydrograph. Enlargement of the breach is further evaluated by a sudden collapse due to excess hydrostatic pressure and breach width expansion by slope stability (Gee, 2010). The outflow hydrograph is obtained through a time-stepping solution.

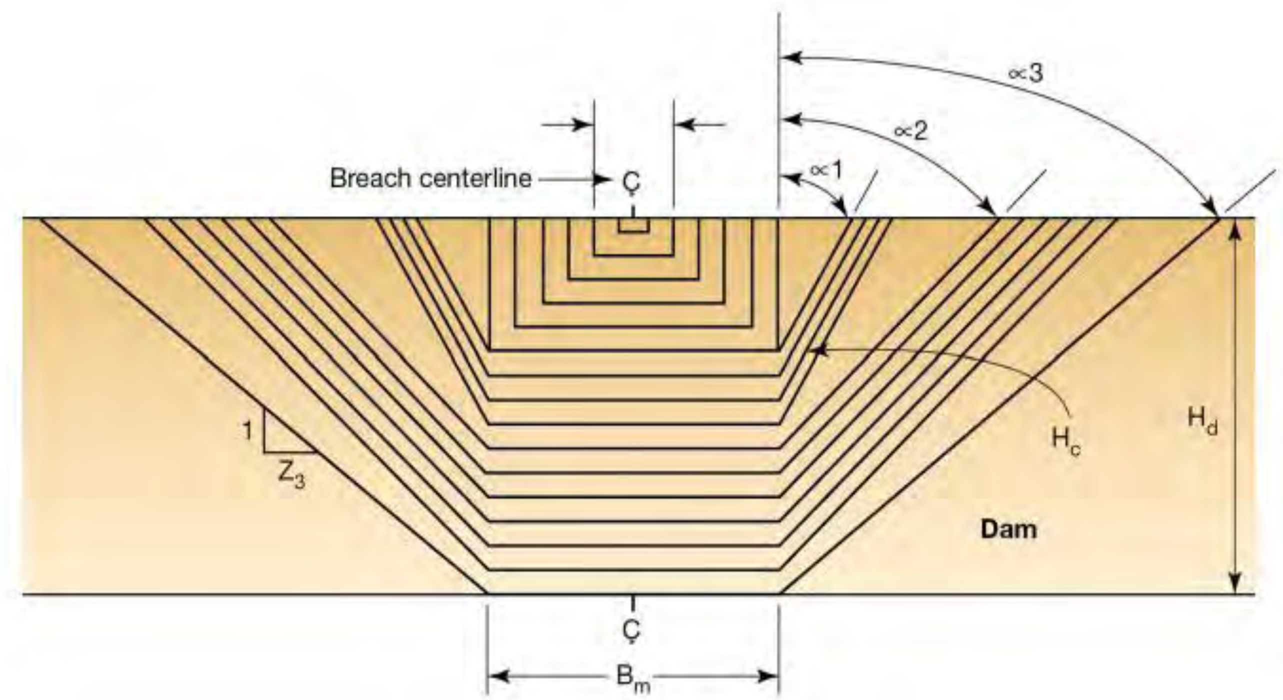


Figure 2.6: The growth of the breach

As documented in the BREACH Manual developed by Fread in 1991, the BREACH model considers the possible existence of the following complexities:

- Core material having properties that differ from those of the outer portions of the dam

- The necessity of forming an eroded ditch along the downstream face of the dam prior to the actual breach formation by the overtopping water
- The downstream face of the dam having a grass cover or being composed of a material of larger grain size than the outer portion of the dam
- Enlargement of the breach through the mechanism of one or more sudden structural collapses due to the hydrostatic pressure force exceeding the resisting shear and cohesive forces
- Enlargement of the breach width by slope stability theory
- Initiation of the breach via piping with subsequent progression to a free surface breach flow
- Erosion transport for either non-cohesive (granular) materials or cohesive (clay) materials

Wahl (2004) suggests that the BREACH model is constrained, as other similar models, in that it does not adequately model head cutting erosion processes that dominate the breaching of cohesive soil embankments. Another limitation of the BREACH model is that the breach hydrograph prediction is simulated without incorporating downstream effects, such as tail water and dynamic effects on the flow within the upstream reservoir, because it uses level pool reservoir routing. This program may be used in conjunction with other programs to simulate downstream dynamic effects using the breach parameter results (i.e., breach width and development time) as input into a separate flood routing model that can determine the breach hydrograph itself, while accounting for dynamic water-level effects of the reservoir and downstream tail water effects (Fread, 1988; Wahl, 2010).

The NWS DAMBRK and FLDWAV software contain a BREACH subprogram that simulates piping and overtopping failures in earthen dams when users provide the typical dam and reservoir characteristics, thus generating breach parameters.

USACE HEC-HMS program

HEC-HMS is a hydrologic modeling program typically used to conduct hydrologic simulations of the precipitation-runoff process of dendritic drainage basins. The program can also be used to perform dam failure analysis. HEC-HMS was developed by the USACE in 1992 to replace the HEC-1 program. The program has been updated several times since its initial release and the most current version of the HEC-HMS program can be found at the USACE’s HEC’s The following paragraphs have been adapted from HMS user support documents developed by the USACE.

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

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

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

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

2.10.2. Breach hydrograph generation and downstream hydraulic routing

This section addresses the most common tools that can perform both breach hydrograph generation and downstream routing of the associated breach hydrograph.

USACE HEC-1 Program

The USACE's HEC-1 program was first developed in 1968 and last updated in 1998, after which it was replaced by the hydrologic modeling software HEC-HMS, developed in 1992. Although, the HEC-1 has been superseded, some Federal documentation still references the use of the HEC-1 model for dam failure analysis, primarily because most of these documents pre-date the common application of the USACE HEC-HMS or HEC-RAS programs for dam failure simulation. For this reason, a discussion of HEC-1 is included within this document.

The program includes a dam safety analysis capability that uses simplified hydraulic techniques to estimate the potential for and consequences of dam overtopping or structural failures on downstream areas in a floodplain. A dam failure analysis has two main components: the reservoir component and the dam safety simulation component. The reservoir component is employed in a stream network model to simulate a dam failure. Most of the modeling effort is characterizing the inflows to the dam under investigation, specifying the characteristics of the dam failure, and routing the dam failure hydrograph to a desired location in the downstream

Floodplain. The dam safety simulation differs from reservoir routing in that the elevation-outflow relation is computed by determining the flow over the top of the dam (dam overtopping) and/or through the dam breach (piping/internal erosion), as well as through other reservoir outlet works. The elevation-outflow characteristics are then combined with the level pool storage routing to simulate a dam failure.

A dam breach is simulated in the HEC-1 program using the methodology incorporated by Fread in the NWS DAMBRK program (Fread, 1979). Structural failures are modeled by assuming certain geometrical shapes for the dam breach. The outflow from a dam breach may be reduced by backwater from downstream constrictions or other flow resistances. HEC-1 allows a tail water rating curve or a single cross-section (and a calculated normal-depth rating curve) to be used to reflect such flow resistance. Submergence effects are calculated in the same manner as in DAMBRK. The dam-break simulation assumes that the reservoir pool remains level and routes the flood wave downstream using steady-state theory (USACE, 1998).

USACE HEC-RAS Program

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

The following discussion on HEC-RAS is adapted from user support documents developed by the USACE.

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

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

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

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

1. **Losses**: Steady-flow losses computations use absolute differences in velocity head at adjacent cross-sections multiplied by an expansion or contraction coefficient, whereas unsteady-flow loss computations are computed by the momentum equation.
2. **Friction** Slope: Average friction slope between cross-sections is determined by averaging the conveyance method for steady flow. For unsteady flow, the average friction slope between cross-sections is computed directly from a simple average of the computed friction slopes.
3. **Discharge:** Steady-flow computations compute losses through downstream obstructions, such as culverts and bridges, directly from the obstruction geometry and the type of flow conditions through the structure. In unsteady flow, a family of curves is developed for defining the headwater-tail water-discharge relationships through each obstruction for a full range of flow.

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

2.10.3. Recommendations for selecting modeling software

The selection of an appropriate model for computing a dam breach is dependent on type of results needed, the level of effort that can be expended, and the potential for loss of life and economic damages that can result from a dam failure.

For dams in rural areas where the potential for loss of life is low, a tier 1 level study using simplified methods may be appropriate. For areas where a potential dam breach can result in the loss of life an intermediate tier 2 level or advanced tier 3 should be performed. The intermediate tier 2 level study may be used for areas where more detailed calculations are justified because of the potential for loss of life. Advanced tier 3 level studies may be needed to develop dam breach inundation zone mapping for urbanized areas and for unconfined floodplains.

CHAPTER THREE

GENERAL DESCRIPTION OF THE STUDY AREA

3.1. Location

Melka Wakana Dam is located in the highlands of Bale Zonal Administration about some 280Km.south-east of Addis Ababa. It lies 70N of the equator. The Dam is at the Wabe Shebelle River a large water course, flowing along the south-east cost of the country towards the territory of Somalia.

The 2300m-2400m elevation of altitude has highly influenced the climatic situation of the vicinity. The mean annual temperature is not greater than 13-14C (Max 28C).the dam is in the upper course of the river where the mean annual flow is 827x106 m3 and the maximum high-water flow is 530 m3/s. The terrain conditions of the region are favorable for creating a reservoir for the over-year regulation, capacity 763 m3, and for installing a derivation hydro power plant.

The rock-fill dam with a length of 2,000 m and maximum height of 42 m is filled with local materials with the central loamy core and the rock apron slopes. The areal cement grouting and the cement-grout curtain to a depth of 25 to 30 m are provided in the dam foundation. The automatic flood gate without shutters on the top edge of the discharge structure is designed for flood discharge with flows of up to 640 cubic m/s. The water diversion chute (horizontally curved) has a variable grade over the length and ends in a ski jump spillway, which dumps water into the river channel. Construction of dam started on 1983 and commissioned on 1988.

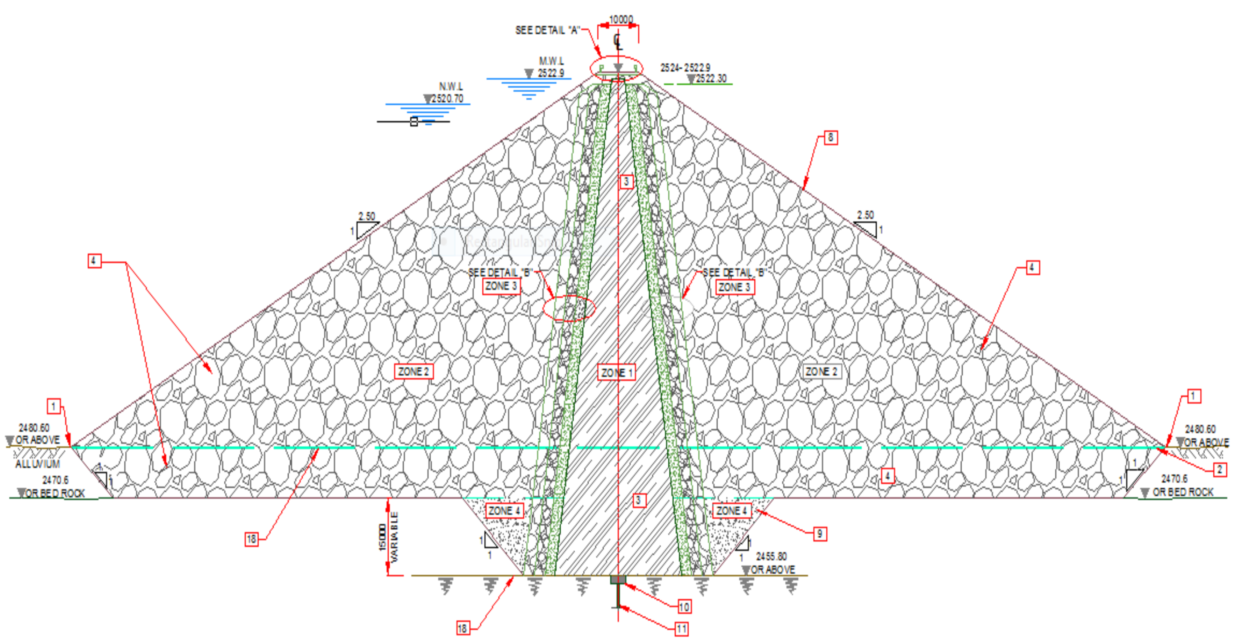


Figure 3.1: Melka Wakana Embankment Dam

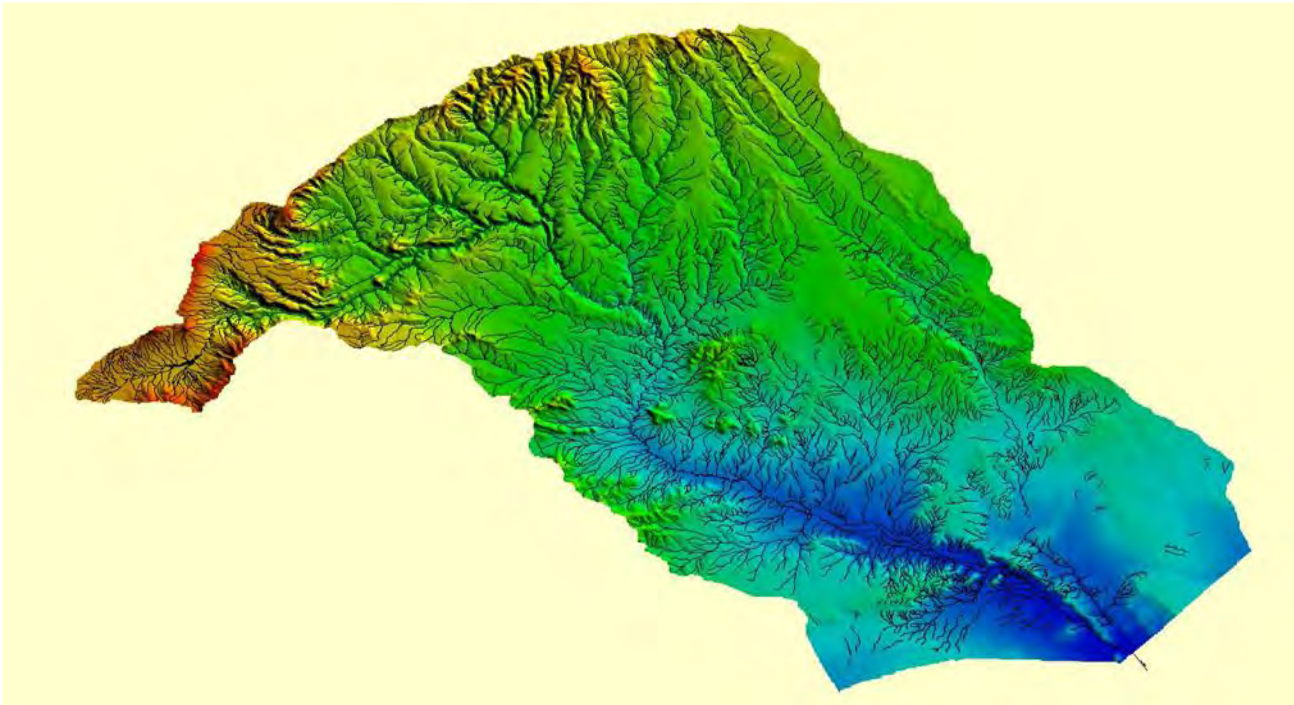


Figure 3.2: Wabe Shebelle River basin

Melka Wakana Reservoir as seen from VHR satellite images

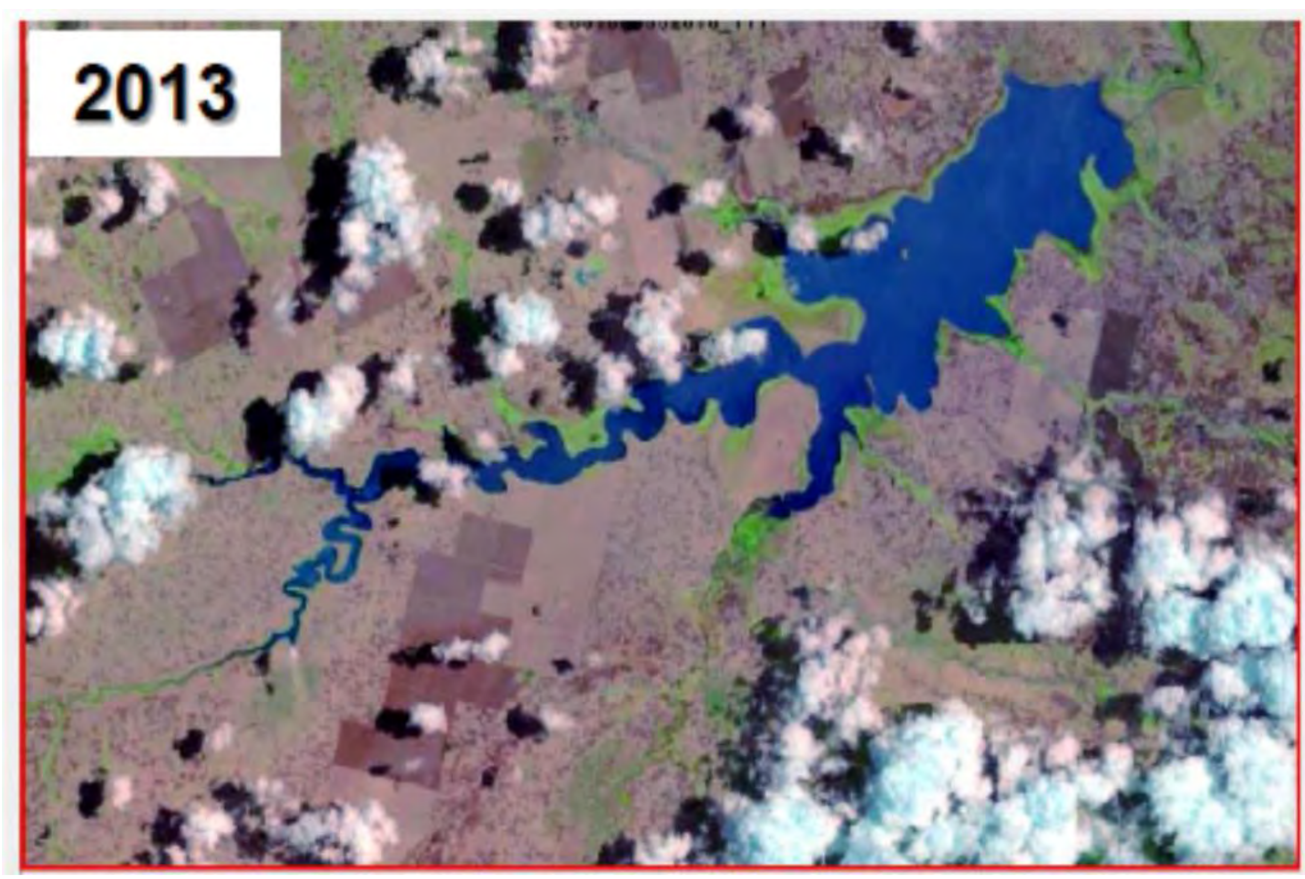


Figure 3.3: Melka Wakana reservoir location

3.2. Climatic Characteristics

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. In the highlands of Ethiopia, temperatures are reasonably warm the year round but rarely hot.

In Ethiopia in general there are three seasons: the first is the dry season (locally known as Bega) which prevails from October to January; the second is the small rainy season (Belg) that runs from February to May and the third is the main rainy season (Kiremt) which prevails from June to September. Rainfall is above 1,000 mm a year almost everywhere in the highlands and it rises to as much as 2,000 – 3,000 mm in the wetter southwestern parts. Annual rainfall decreases when one moves to the east and north of the country. Night time temperatures fall nearly or below freezing in mountainous areas (higher than 2,500 m). In the northern lowlands, Danakil depression, the southern lowlands, and Ogaden, rainfall is low (below 300 mm/year) and temperatures are high (higher than> 30 °C) during the whole year. Since Ethiopia is situated in the north-eastern part of Africa, it is influenced from the northeast to the Southeast by monsoons bringing moisture from the Indian Ocean. In the northern hemisphere summer, moisture laden winds gradually penetrate into the countries as the African sector of the Inter-Tropical Convergence Zone (ITCZ) progresses northward. The orographic influence on rainfall depth is also marked in the mountainous areas that surround the Project. As a result, rainfall varies highly both seasonally and annually. Nearly 79% of the annual rainfall occurs in the period of June-September.

3.3. Wabi Shebelle River

Wabi Sheble river basin has an area of 202,697 Km2, covering parts of the regions Oromia, Harari and Somali. This river basin has a lowest elevation of 184 m. and a highest elevation of 4182 m. The total mean annual flow from the river basins is estimated at about 3.16 BMC.

CHAPTER FOURE

DAM BREACH ANALYSIS AND INUNDATION MAP METHODOLOGIES

4.1. Data Gathering and Processing

Most of the original data gathered and compiled by the Ministry of Water, Irrigation and Energy and Electric Power Corporation is used in this analysis. The main purpose of gathering data in creating a modeling methodology is for defining the size, type, elevation and storage relations of the subject dam, and the geometries of the downstream river reaches. Data gathered for modeling were grouped into the following categories:

Reservoir characteristics: The reservoir characteristics consist of reservoir storage elevation curve and reservoir surface area elevation curve.

Dam 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.

General Information: This category of data is for general information purposes. It includes jurisdictions of the dam owner (city, town, and country area), geographic information, watershed boundary, and others.

Downstream Information: Data gathered under this category includes bank stations, reach stations, downstream developments, cross section plots, Manning roughness coefficients, and other pertinent hydraulic structures.

Inflow Hydrograph: The inflow hydrograph data category includes the flood events hydrograph provided by the dam owner.

4.2. 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. Overtopping and piping mode of failure is assumed in this study. Hydraulic analysis of dam breach includes two primary tasks, the prediction of the reservoir outflow hydrograph and the routing of that hydrograph through the downstream valley.

4.2.1. Predicting the Outflow Hydrographs

For flood hydrograph estimation, the breach modeled by defining acceptable dam breach parameters is the core of this project. Predicting the outflow hydrographs at the dam location is done using HEC-RAS under different scenarios. The process of predicting the outflow hydrograph is a multi-steps approach and began with defining the river geometry, the reservoir characteristics, physical description of Melka Wakana Dam, and its detailing breach characteristics

Several researchers have developed peak flow regression equation form historic dam failure data. The peak flow equation were derived from data for earthen, zoned earthen, earthen with impervious core. In general, the peak flow equation should be used for comparison purposes.

Once a breach hydrograph is computed in HEC-RAS, the computed peak flow from the model can be compared to these regression equations as a test for reasonableness.

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 by one method with other methods.

First, using MacDonald-Langridge-Monopolis method the maximum breach outflow should be calculated because the breach parameters are estimated using this method.

MacDonal and Langridge-Monopolis (1984)

$$Q=3.85(V_w h_w)^{0.411}$$

Where

Q= peak breach outflow (cubic meters per second)

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

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

For verification of the reasonableness of the value of breach out flow obtained by the analysis we have to compare it with the value obtained by empirical formula as shown above or with the envelope. But the envelope as discussed in chapter three will not be the true upper bound because it only taken in to account fourteen historical dam failure incidents. Therefore, the value obtained using the empirical relationship suggested by MacDonald-Langridge-Monopolis as show above will be used as an upper peak breach outflow.

4.2.2. Defining the River Geometry

When 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 called HEC-GeoRAS which is modeled to integrate the two software. The geometry of the Wabi Shebelle River generated from ArcGIS was exported to HEC-RAS as shown in the figure below

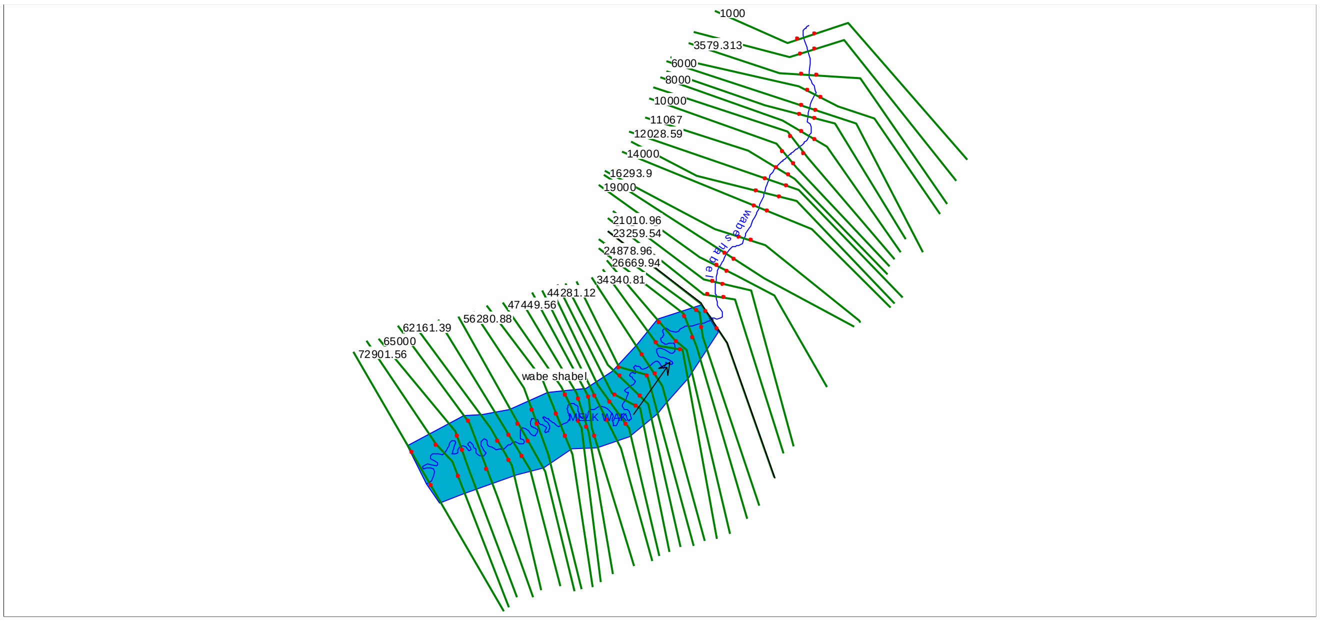


Figure 4.1: HEC-RAS geometry data

4.2.4. Describing Reservoir Characteristics

The original data is the input in the HEC-RAS model. The collected data concerning the reservoir characteristics is Elevation -Volume curves.

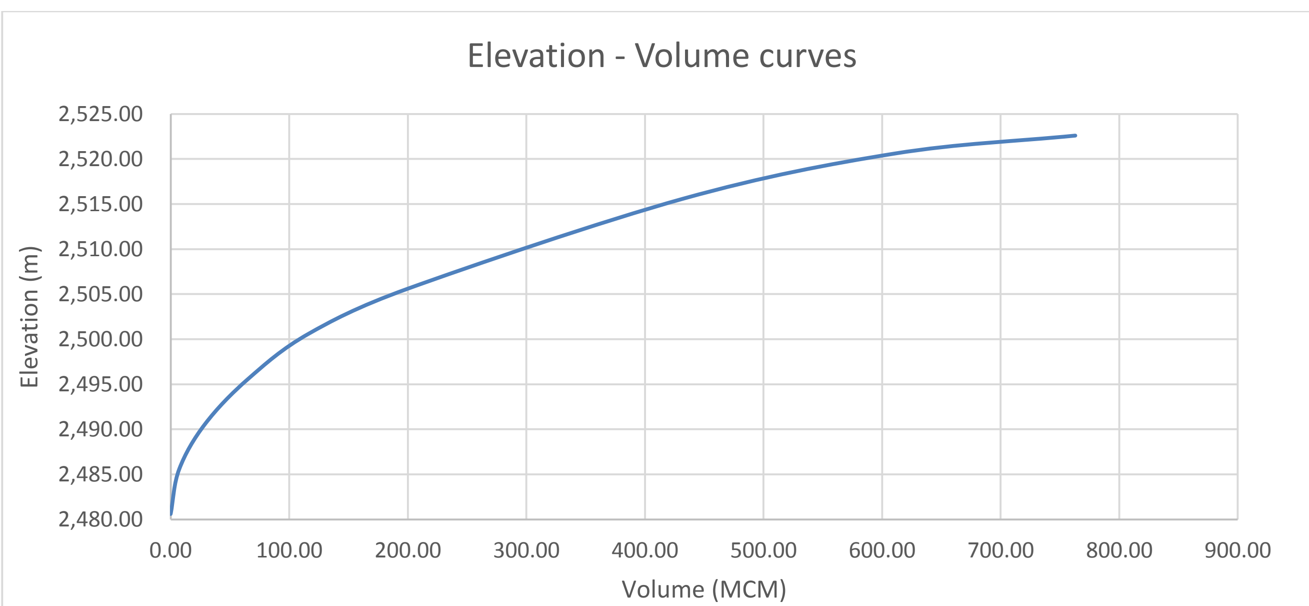


Figure 4.2: Elevation – Volume curves

4.2.4. Physical Descriptions of Dam

This step includes the identification of dam height, dam crest width. In this study, data about the physical characterizes of the Melka Wakena Dam are used as an input in HEC-RAS modeling.

- Crest Length: 2000m
- Crest Width: 10m
- Maximum Height above river bed: 42m
- Average Upstream Embankment slope: 2.5H:1V
- Average Downstream Embankment slope: 2.5H:1V
- Embankment Material: Rock fill
- Storage Capacity: $763x10^6m^3$

4.2.5. Determining Inflow Hydrograph to the Reservoir

The inflow design flood is expected to cause the dam to breach in order to analyze the worst case of dam breach analysis. Inflow hydrographs generated from three days half PMF was used to design the spillway and this hydrograph was used for the breach analysis.

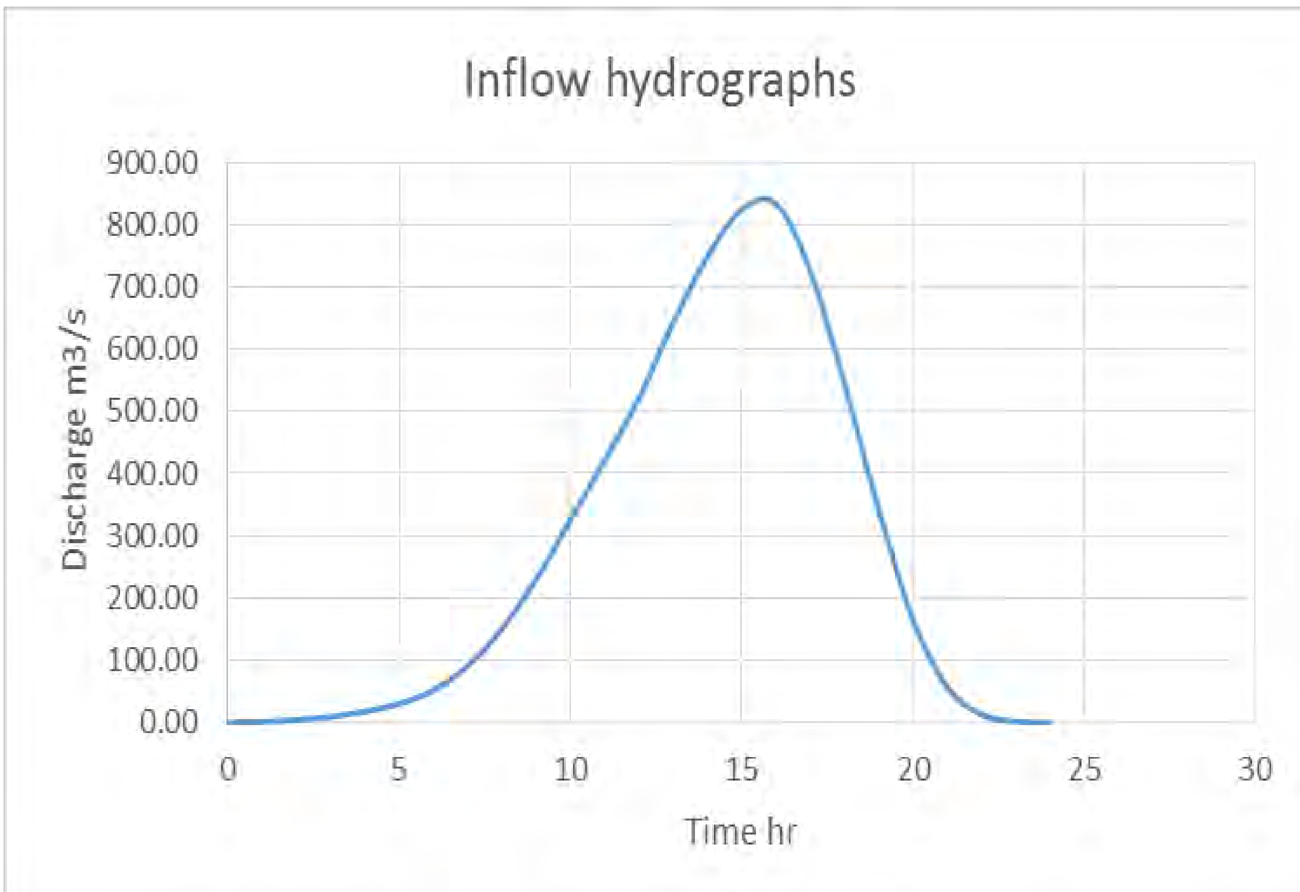


Figure 4.3: Inflow Hydrograph (72 Hours 0.5PMF)

4.2.6. Estimating Dam Breach Characteristics

The following regression equation have been used to estimate breach parameter for embankments dam

- Froehlich (1995a)
- Froehlich (2008)
- MacDonald and Langridge-Monopolis (1984)

For this these, the Froehlich (1995a), Froehlich (2008) MacDonald and Langridge-Monopolis (1984) regression equations for predicting breach size and development time were used. This dam is within the range of the data used to develop these regression equations, therefore the equations are considered to be an appropriate methodology for estimating the breach parameters.

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

- Height of the dams: 3.66 --- 92.96 meters
- Volume of water at breach time: 0.0130 --- 660.0 m^3 x 10^6

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

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

$$t_f = 0.00254\ V_w^{0.53}\ h_b^{-0.90}$$

Where:

B_{ave} = average breach width (meters)

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

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

h_b = height of the final breach (meters)

t_f = breach formation time (hours)

Froehlich states that the average side slopes should be:

1.4H:1V overtopping failures

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

Froehlich (2008): In 2008, Dr. Froehlich updated his breach equations based on the addition of new data Dr. Froehlich utilized 74 earthen, zoned earthen, earthen with a core wall (i.e., clay), and rock fill date sets to develop as set of equations to predict average breach width, side slopes, and failure time. The data that Froehlich used for his regression analysis had the following ranges:

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

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

$$B_{ave} = 0.27K_o\, V_w^{0.32}\, h_b^{0.04}$$

$$t_f = 63.2(V_w/(ghb2))^{0.5}$$

Where:

B_{ave} = average breach width (meters)

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

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

h_b =height of the final breach (meters)

g = gravitational acceleration (9.80665 meters per second squared)

t_f = breach formation time (seconds)

froehlich's 2008 paper states that the average side slopes should be:

1.0H:1V overtopping failures

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

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

- Height of the dams: 4.27-92.96 meters
- Breach outflow volume: 0.0037-660.0$m^3 x 10^6$

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

For earthfill dam:

$$V_{eroded} = 0.0261\ (V_{out}*h_w)^{\ 0.769}$$

$$t_f = 0.0179\ (V_{eroded})^{\ 0.36}$$

Where:

V_{eroded} = volume of material eroded from the 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)

MacDonald and Langridge-Monopolis stated that the breach should be trapezoidal with side slopes of 0.5H: 1V. The base width of the breach can computed from the dam geometry with the following equation (State of Washington, 1992):

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

Where:

W_b = bottom width of the breach (meters)

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

Z_3 = Z_1+Z_2

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

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

Z_b = side slopes of the breach (Z_b:1),0.5 for the Macdonald method

4.2.7. 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 is 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:

4.2.8. Defining Channel Geometry and Boundary Conditions

During modeling of the downstream channel of the Melka Wakana Dam using HEC-RAS, the first step is to establish the external boundary conditions. The upstream boundary is selected at a location such that it is independent of the downstream conditions. The downstream boundary was selected at a location that is independent of flow conditions below the boundary. The last downstream cross section is set at a reasonable distance and a normal depth is chosen to define the downstream boundary conditions.

After the routing reach is established by the boundary locations, cross sections are obtained to represent the reaches. Cross section locations are measured from downstream to upstream.

For the purposes of this study, default values of expansion and contraction coefficients are used throughout the unsteady state analysis. The program by default assigns a value of 0.3 and 0.1 for expansion and contraction coefficient, respectively.

4.2.9. Selecting Manning Coefficients, "n" 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 and so on.

In unsteady state river routing simulation, results were often very sensitive to the Manning n values. Selection of the Manning n is aimed to reflect the influence of bank and bed materials, channel obstructions, irregularity of the river banks and to minimize potential biasness of the results.

By referring the Wabe Shebelle river bed and bank materials and Chow, Manning n values 0.048 and 0.042 are taken for the banks and flow channel respectively.

CHAPTER FIVE

RESULTS AND DISCUSSIONS

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

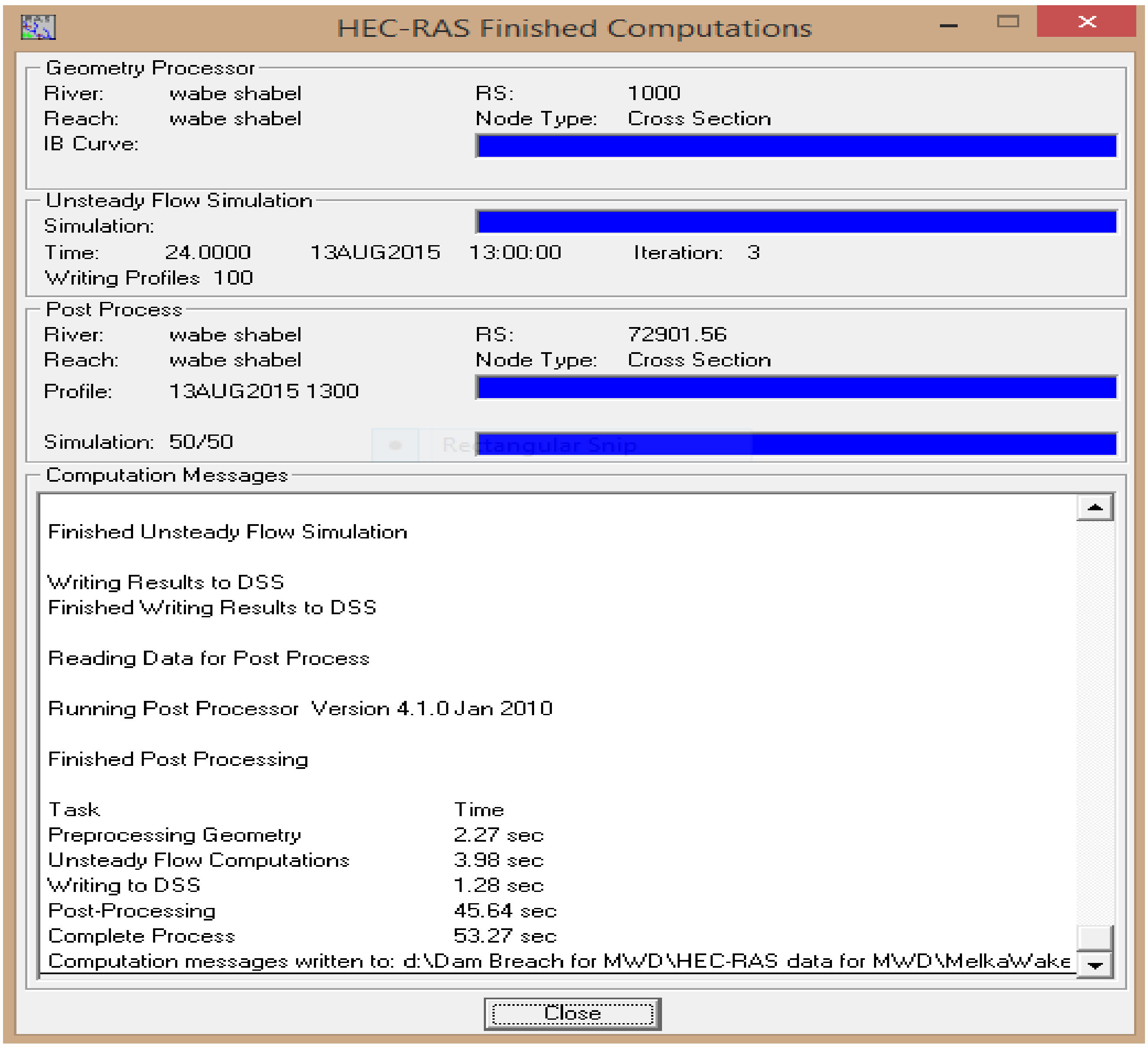


Figure 5.1: HEC-RAS Unsteady Flow Simulation Computer Run

5.1. Breach Parameter Estimates

For this thesis

- Froehlich (1995A)
- Froehlich (2008)
- MacDonald and Langridge Monopolis (1984)

The following are the calculations for each method

Froehlich (1995a)

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

$B_{ave} = 0.1803 * 1.4 * (827x10^6)^{0.32} * (42)^{0.19}$

$\mathbf{B_{ave} = 366.56}$ **meters** (for over topping failure)

$B_{ave} = 0.1803 * 1.0 * (827x10^6)^{0.32} * (42)^{0.19}$

$\mathbf{B_{ave} = 261.83}$ **meters** (for piping failure)

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

$t_f = 0.00254 * (827x10^6)^{0.53} * (42)^{-0.90}$

$\mathbf{t_f = 4.68}$ **hours**

The Froehlich (1995a) method assumes a side slope of 1.4H:1V for an overtopping breach and 0.9H:1V for a piping breach. Given the breach height of 42 meters, this yields a bottom width for the breach of $\mathbf{W_b = 307.76}$ **meters** for overtopping and $\mathbf{W_b = 224.03}$ **meters**

Froehlich (2008)

$B_{ave} = 0.27\ K_o\ V_w^{0.32}\ h_b^{0.04}$

$B_{ave} = 0.27 * 1.3 * (827x10^6)^{0.32} * (42)^{0.04}$

$\mathbf{B_{ave} = 290.96}$ **meters** (for over topping failure)

$B_{ave} = 0.27 * 1.0 * (827x10^6)^{0.32} * (42)^{0.04}$

$\mathbf{B_{ave} = 223.82}$ **meters** (for piping failure)

$t_f = 63.2\ (\ V_w/(g\ h_b^2))^{0.5}$

$t_f = 63.2 * (827x10^6/(9.81 * (42)^2))^{0.5}$

$\mathbf{t_f = 3.84}$ **hours**

The Froehlich (1995a) method assumes a side slope of 1.0H:1V for an overtopping breach and 0.7H:1V for a piping breach. Given the breach height of 42 meters, this yields a bottom width for the breach of $\mathbf{W_b = 248.96}$ meters for overtopping and $\mathbf{W_b = 194.42}$ **meters**

MacDonald and Langridge Monopolis (1984)

$V_{eroded} = 0.00348\ (V_{out} * h_w)^{0.852}$

$V_{eroded} = 0.00348\ (606x10^6 * 40.10)^{0.852}$

$\mathbf{V_{eroded} = 2.45x10^6}$ **cubic meters of material**

To compute the bottom width of the breach, the method says to use side slopes of 0.5H: 1V.The user must also estimate an average side slope for both the upstream and downstream embankment of the dam. In our case average side slope of 2.5H:1V were used for both upstream and downstream. The bottom width equation is

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

Where:

$W_b = (2.45x10^6 - 42^2(10*0.5+42*0.5*5/3))/(42(10+42*5/2))$

$\mathbf{W_b = 493.76}$ **meters**

$t_f = 0.0179\ (V_{eroded})^{0.364}$

$t_f = 0.0179\ (2.45x10^6)^{0.364}$

$\mathbf{t_f = 3.79}$ **hours**

Table 5.1: Summary of Breach Parameter Estimates

Method	Breach Bottom Width (meters)	Breach Side Slopes (H:1V)	Breach Failure Time (hours)
Froehlich (1995a)	307.76m (overtopping) 224.03m (piping)	1.40 0.9	4.68h
Froehlich (2008)	248.96m (overtopping) 194.42m (piping)	1.00 0.7	3.84h
MacDonald and Langridge-Monopolis	493.76m	0.50	3.79h

From here, all three set of parameters should be entered into HEC-RAS software and run as separate breach plans. This will result in three different breach outflow hydrographs. 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.

In this Dam Break Analysis, using mixed flow regime simulation, both upstream and downstream boundary conditions (inflow hydrograph and rating curve, respectively) and dam dimensions were identified.

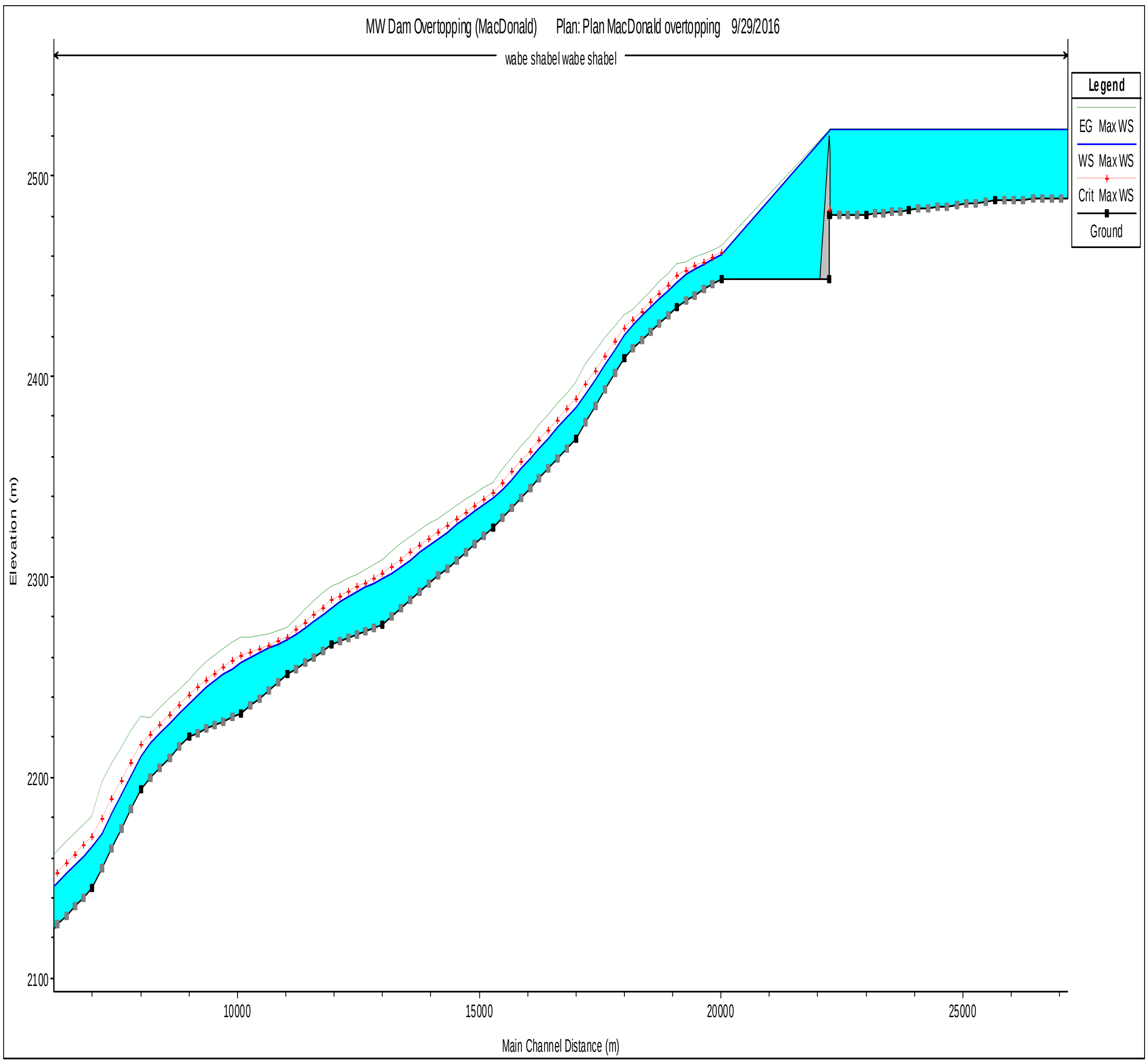


Figure 5.2: Dam Break Flood Profile

5.2. Outflow hydrograph due to dam breach.

Unsteady-flow simulations were performed for Probable Maximum Flood (PMF).Once a breach hydrograph is computed in HEC-RAS, the computed peak flow from the model can be compared to these regression equations as a test for reasonableness.

For Froehlich (1995a)

Using HEC-RAS model computed peak flow is

Q_p = 35,494.02 m^3/s (overtopping)

Q_p = 25,255.13 m^3/s (piping)

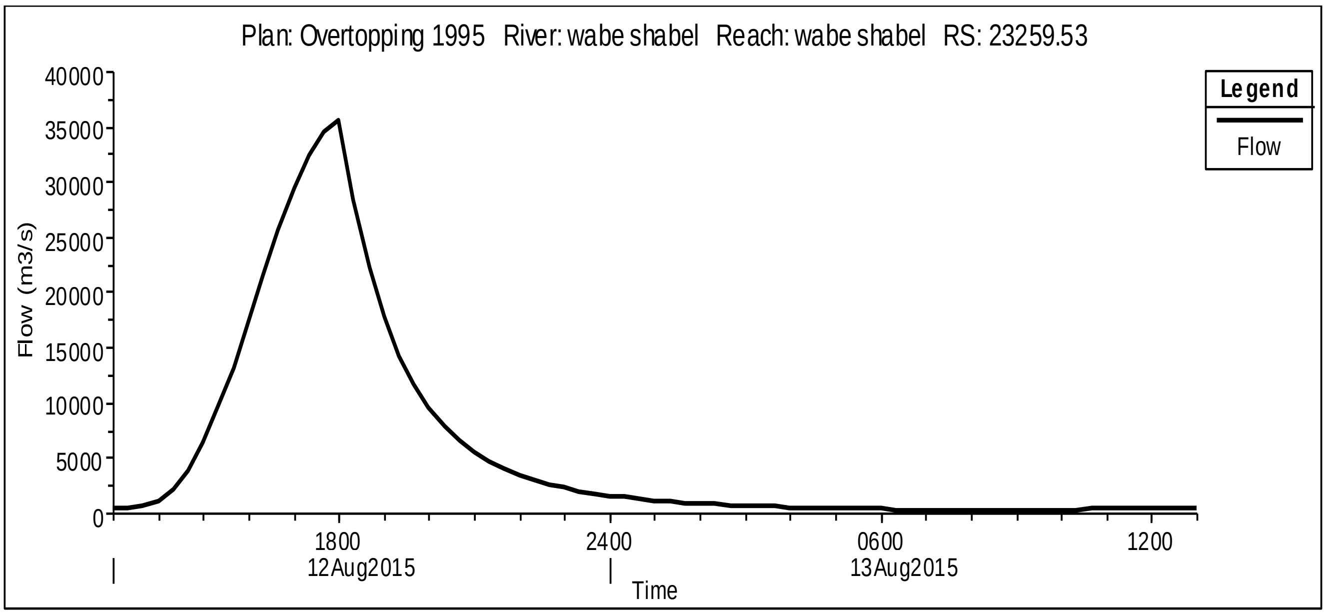


Figure 5.3: the discharge flowing out of the dam during the dam break for overtopping

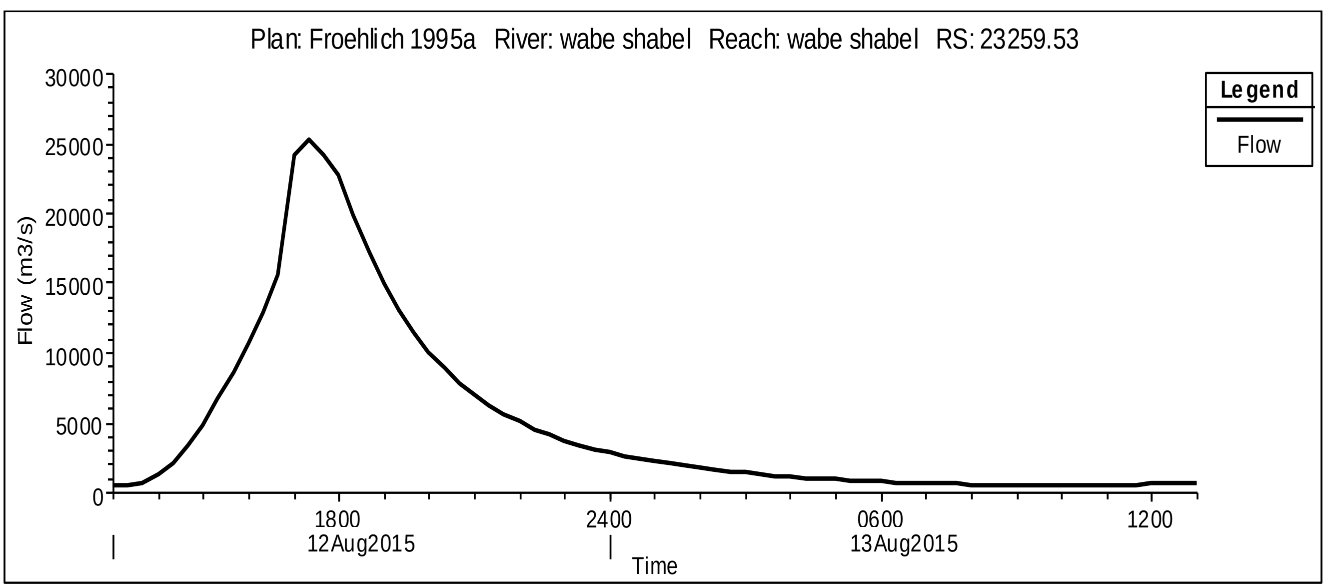


Figure 5.4: the discharge flowing out of the dam during the dam break for piping

For Froehlich (2008)

Using HEC-RAS model computed peak flow is

Q_p = 28,067.77 m^3/s (overtopping)

Q_p = 25,087.70 m^3/s (piping)

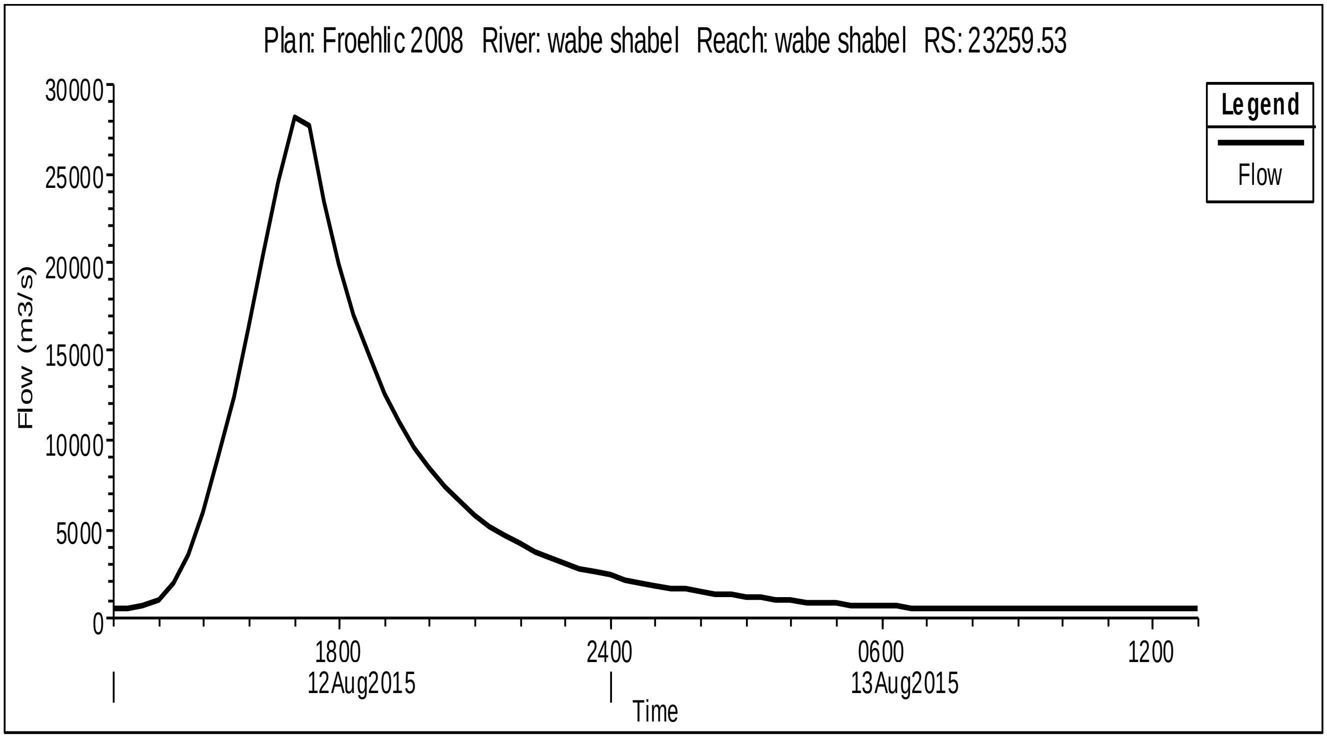


Figure 5.5: the discharge flowing out of the dam during the dam break for overtopping

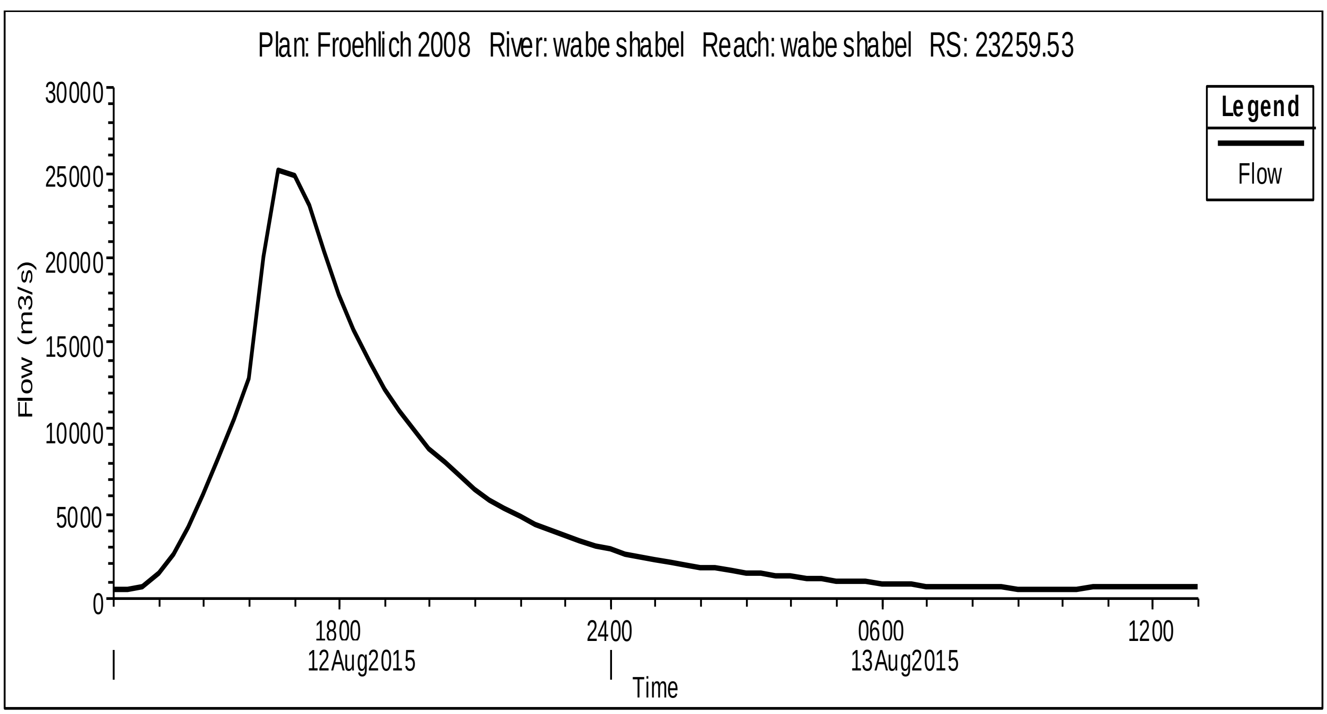


Figure 5.6: the discharge flowing out of the dam during the dam break for piping

For MacDonald and Langridge-Monopolis

Using HEC-RAS model computed peak flow is

$Q_p = 36{,}527.15 m^3/s$ (overtopping)

$Qp = 32{,}627.70\ m^3/s$ (piping)

Using regression equations

$Q_p = 3.85\ (V_w\ h_w)^{\ 0.412}$

$Q_p = 3.85 * ((763x10^6)\ (40.1))^{\ 0.412}$

$Q_p = 80{,}455.6\ m^3/s$

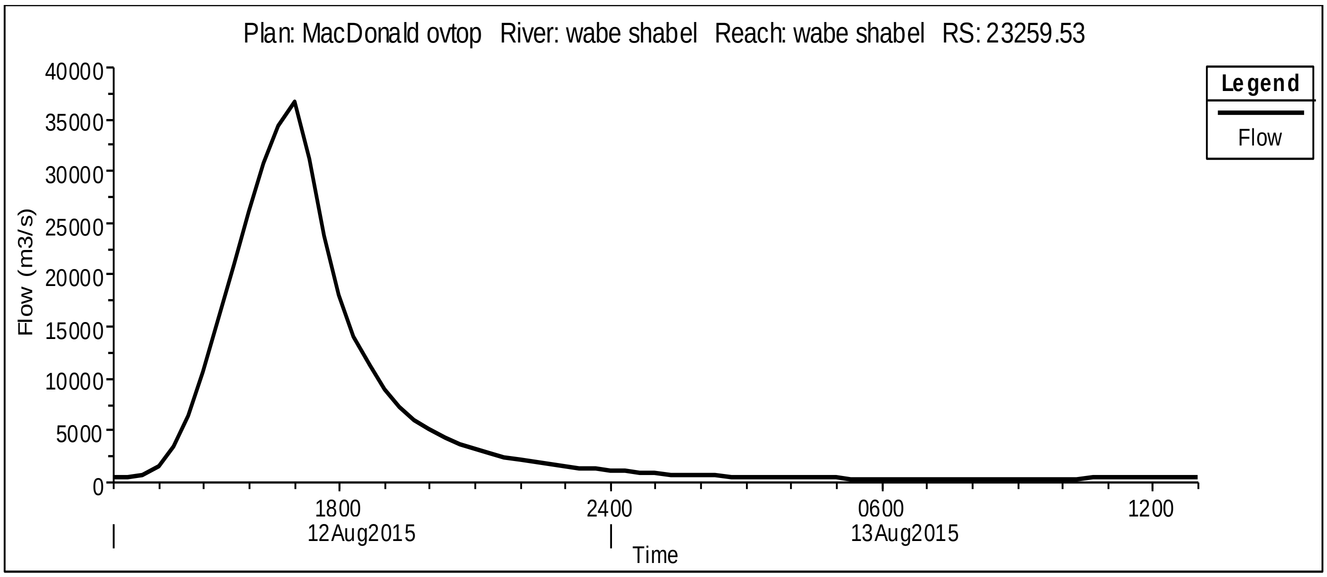


Figure 5.7: the discharge flowing out of the dam during the dam break for overtopping

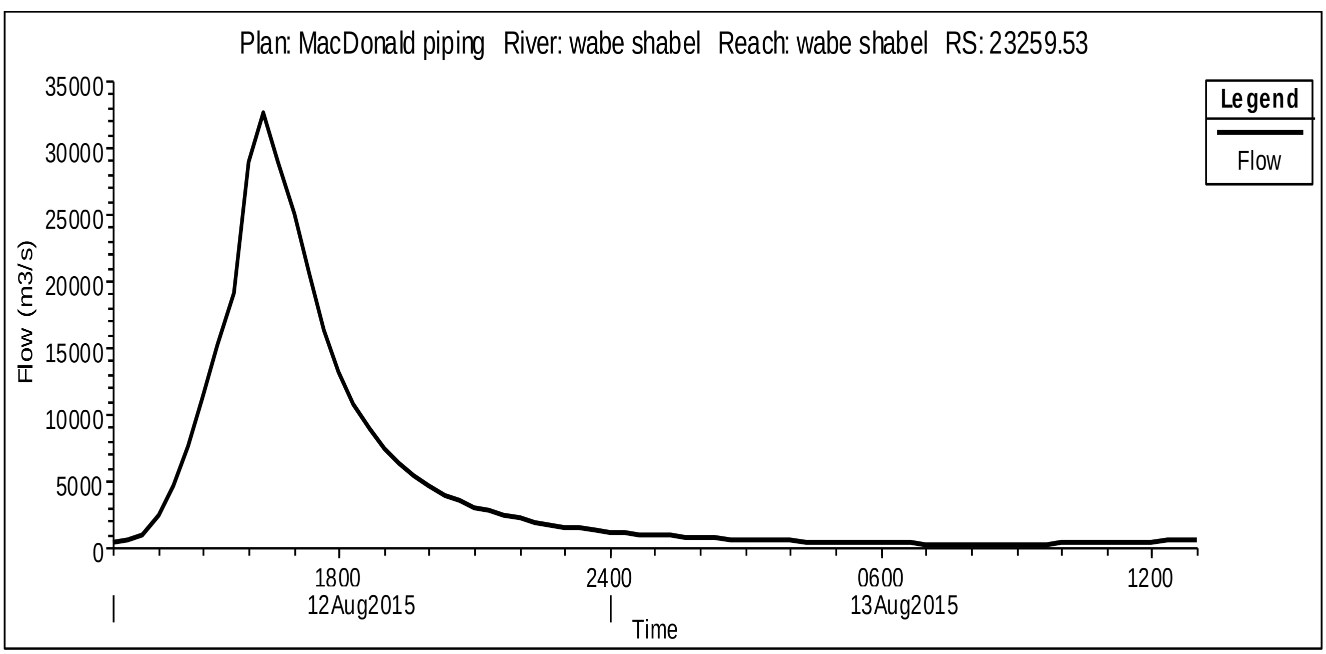


Figure 5.8: the discharge flowing out of the dam during the dam break for piping

From the above HEC-RAS model results we get three different peak flow, but selected the method which have larger value than the other. Ones we select used selected method to make downstream routing, inundation map and emergence action plane.

MacDonald and Langridge-Monopolis gives the largest peak flow, therefor use the result of MacDonald and Langridge-Monopolis for downstream routing, inundation map and emergence action plane.

Q_p= 36,527.15 m^3/s (for overtopping) and

Q_p = 32,627.70 m^3/s (for piping).

Table 5.2: Summary of Results for Peak Flow

Method	**Peak Flow (m^3/s)**	
	Overtopping	**Piping**
Froehlich (1995a)	35,494.02 m^3/s	25,255.13 m^3/s
Froehlich (2008)	28,067.77 m^3/s	25,087.70 m^3/s
MacDonald and Langridge-Monopolis	36,527.15 m^3/s	32,627.70 m^3/s

5.3. Routing of the hydrograph through the downstream

The results from MacDonald and Langridge-Monopolis method for the two failure case are presented as figures. The results are given for 4 selected locations. The maximum discharge decrease from 36,527.15 m^3/s at the outlet of Melka Wakana dam to 35,179.22 m^3/s downstream of the river for overtopping failure and the maximum discharge decrease from 32,627.70 m^3/s at the outlet of Melka Wakana dam to 30,159.74 m^3/s downstream of the river for piping failure.

For overtopping

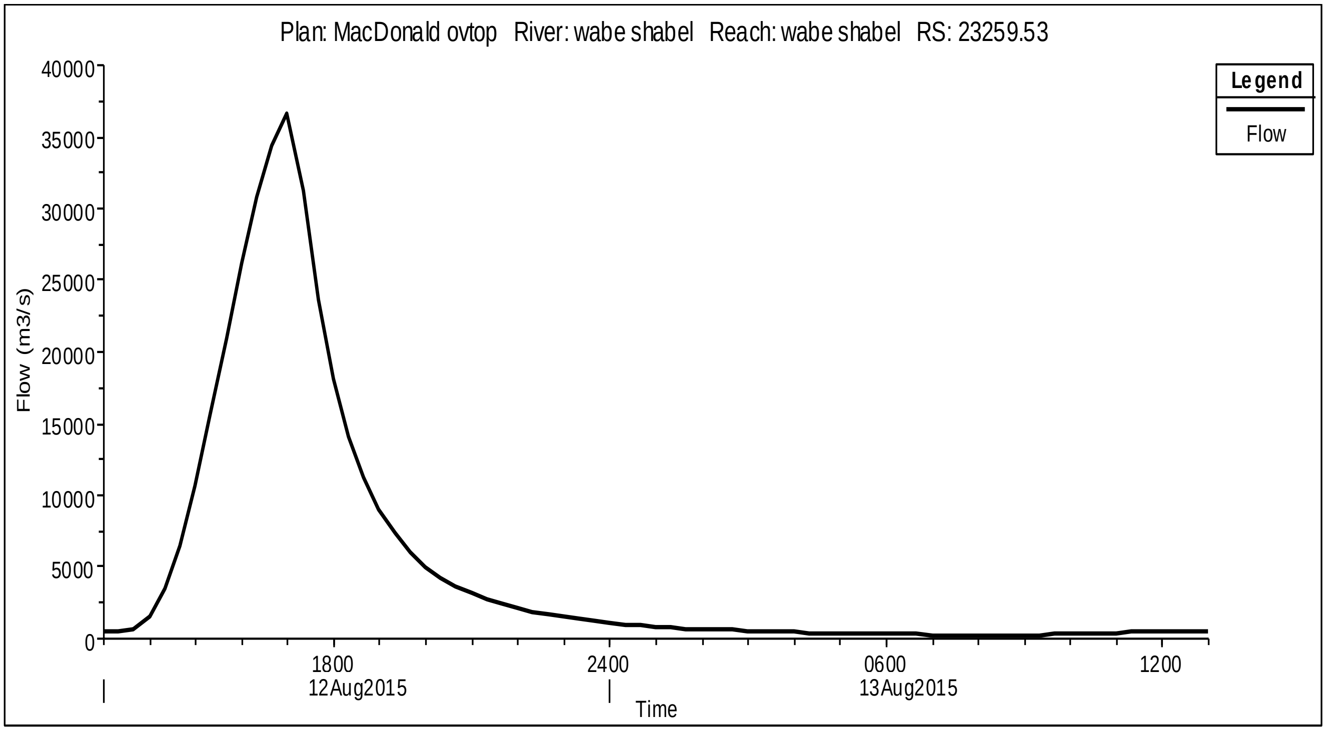


Figure 5.9: the discharge flowing out of the dam during the dam break for overtopping.

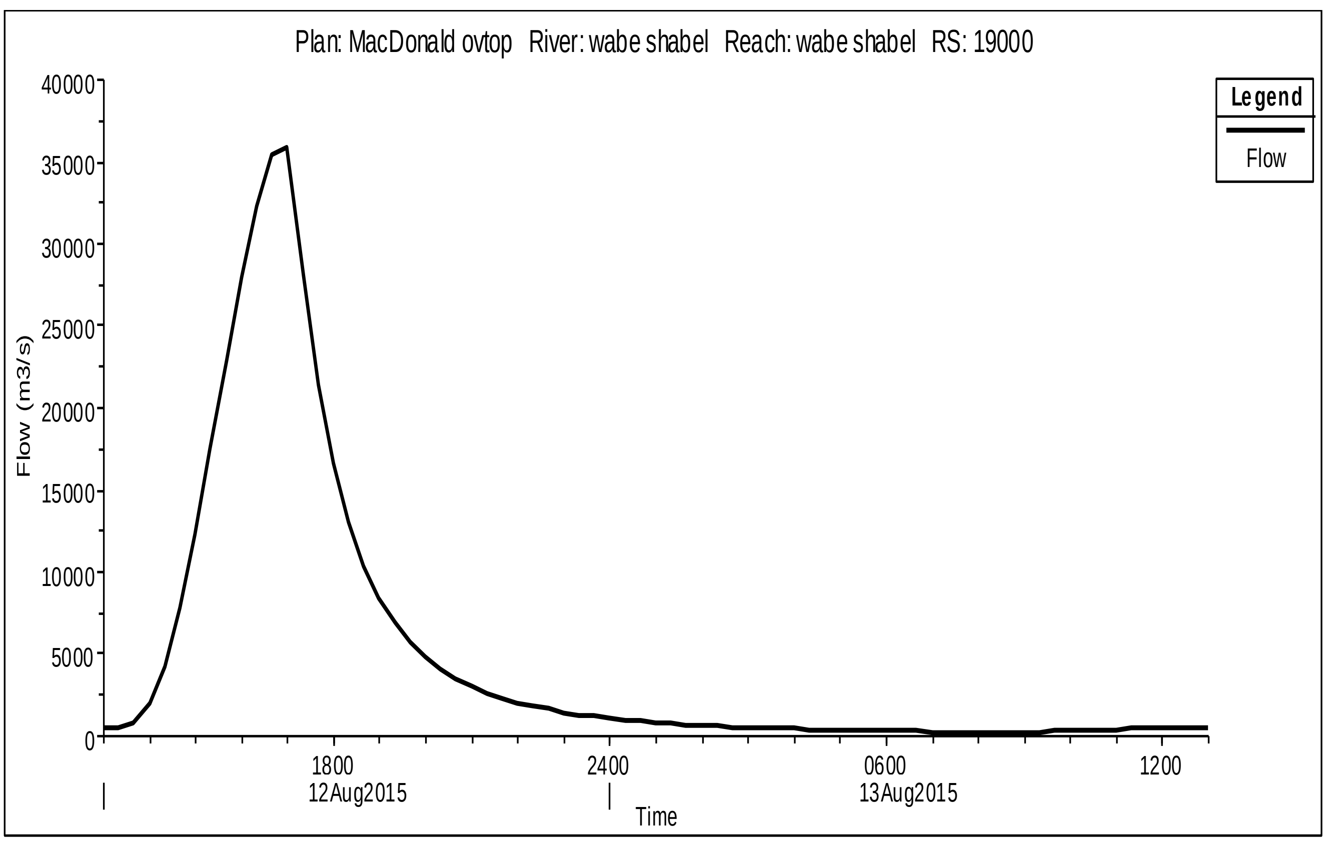


Figure 5.10: the discharge at the RS 19000 for overtopping.

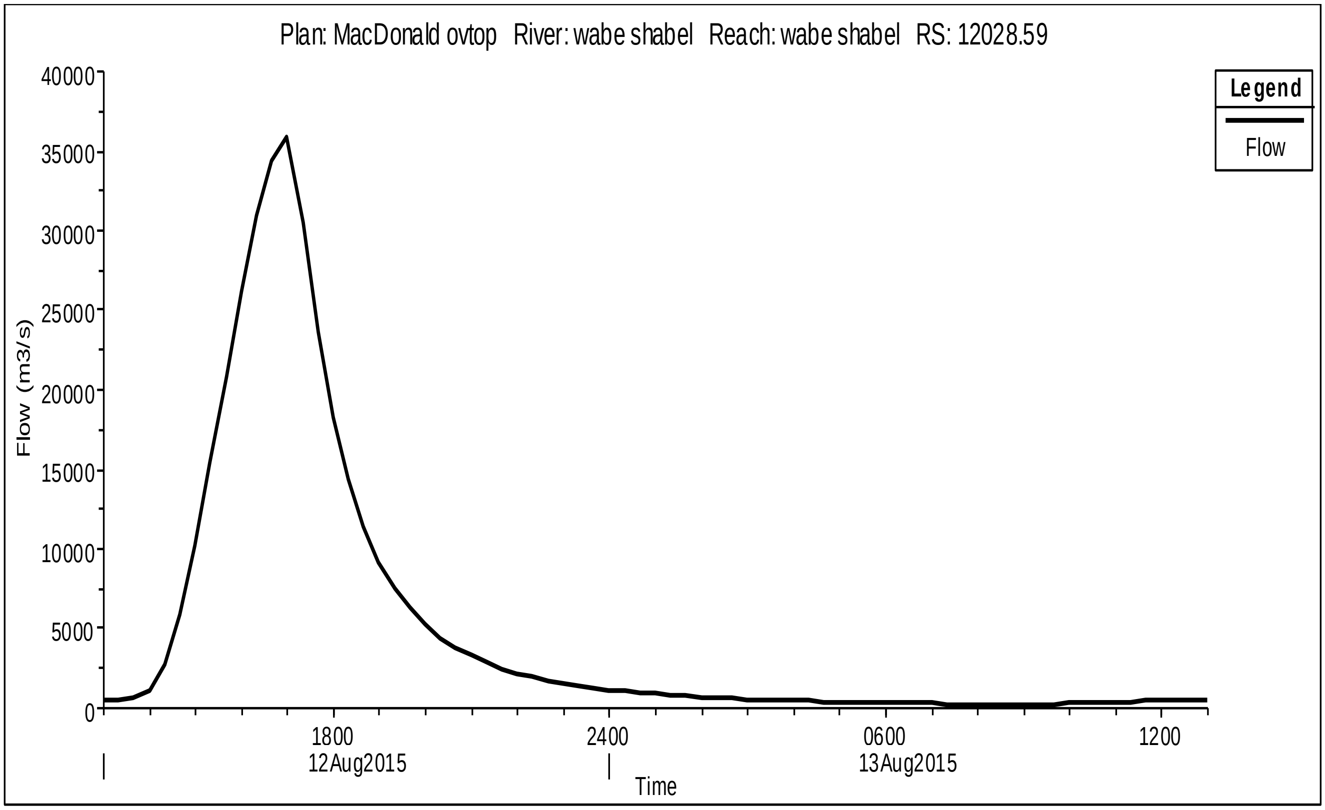


Figure 5.11: the discharge at the RS 12028.59 for overtopping

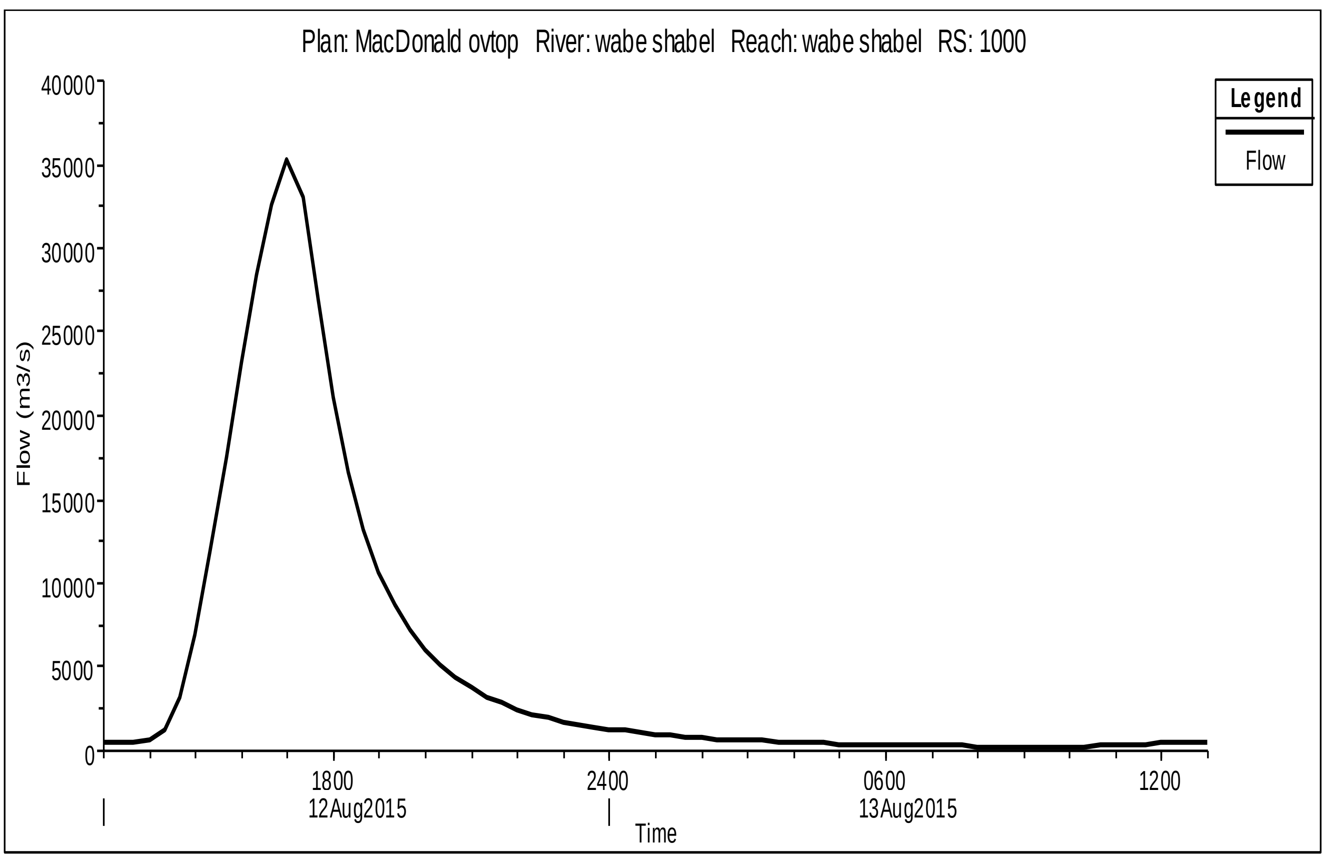


Figure 5.12: the discharge at the RS 1000 for overtopping

For piping

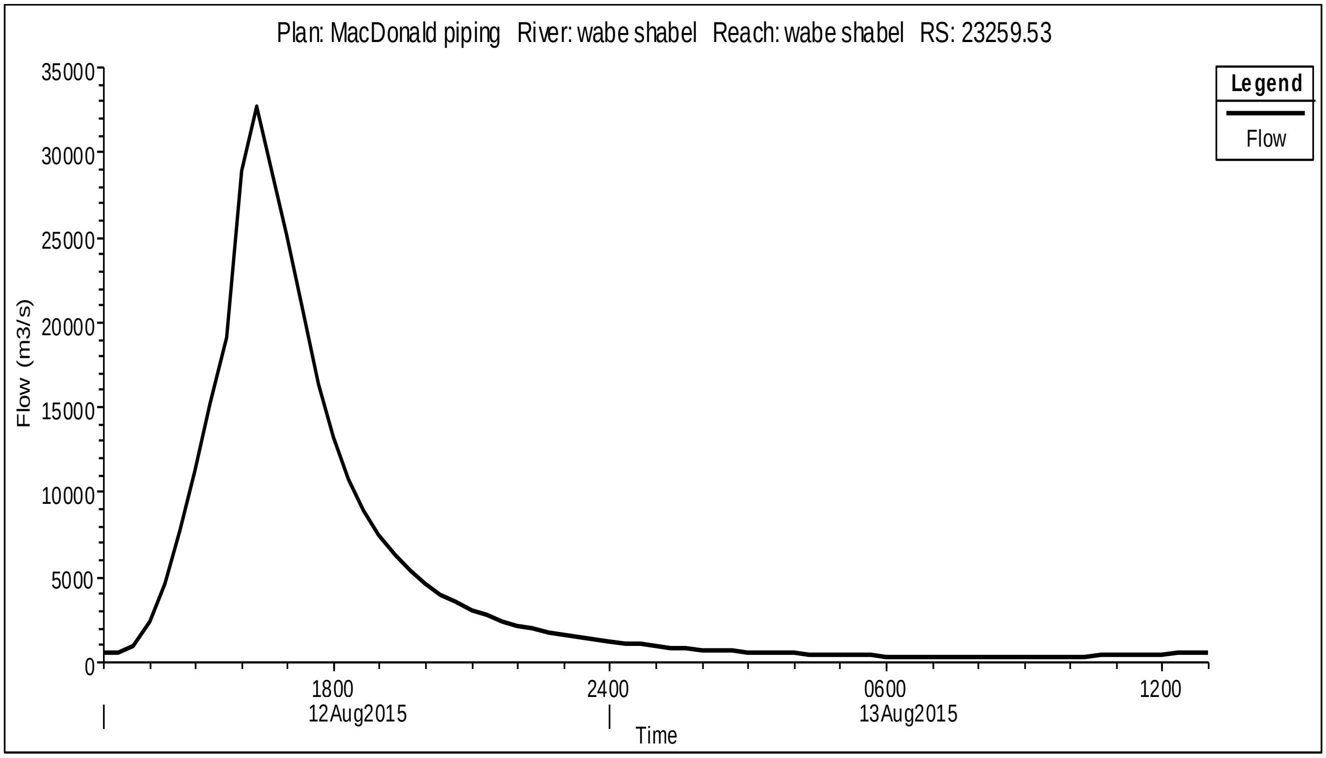


Figure 5.13: the discharge flowing out of the dam during the dam break for piping

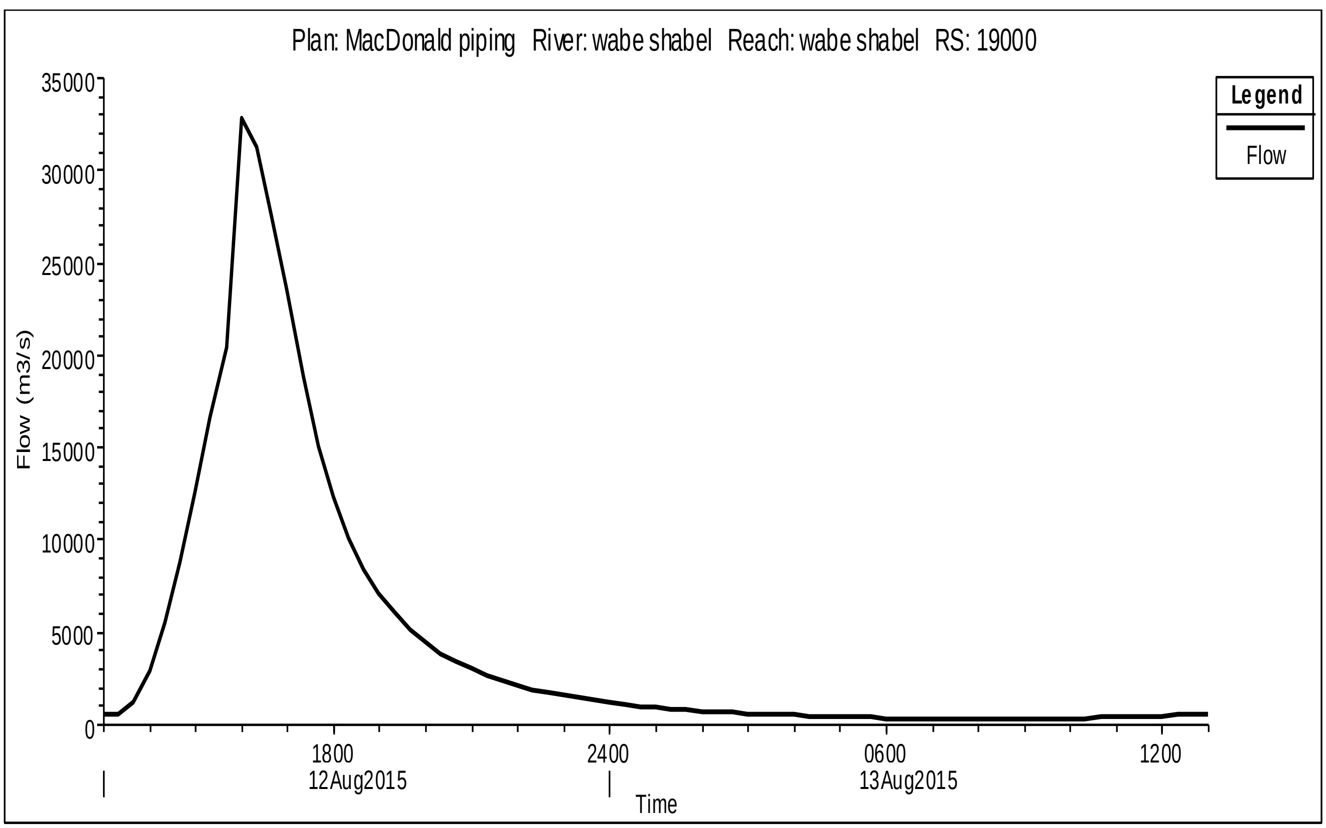


Figure 5.14: the discharge at the RS 19000 for piping

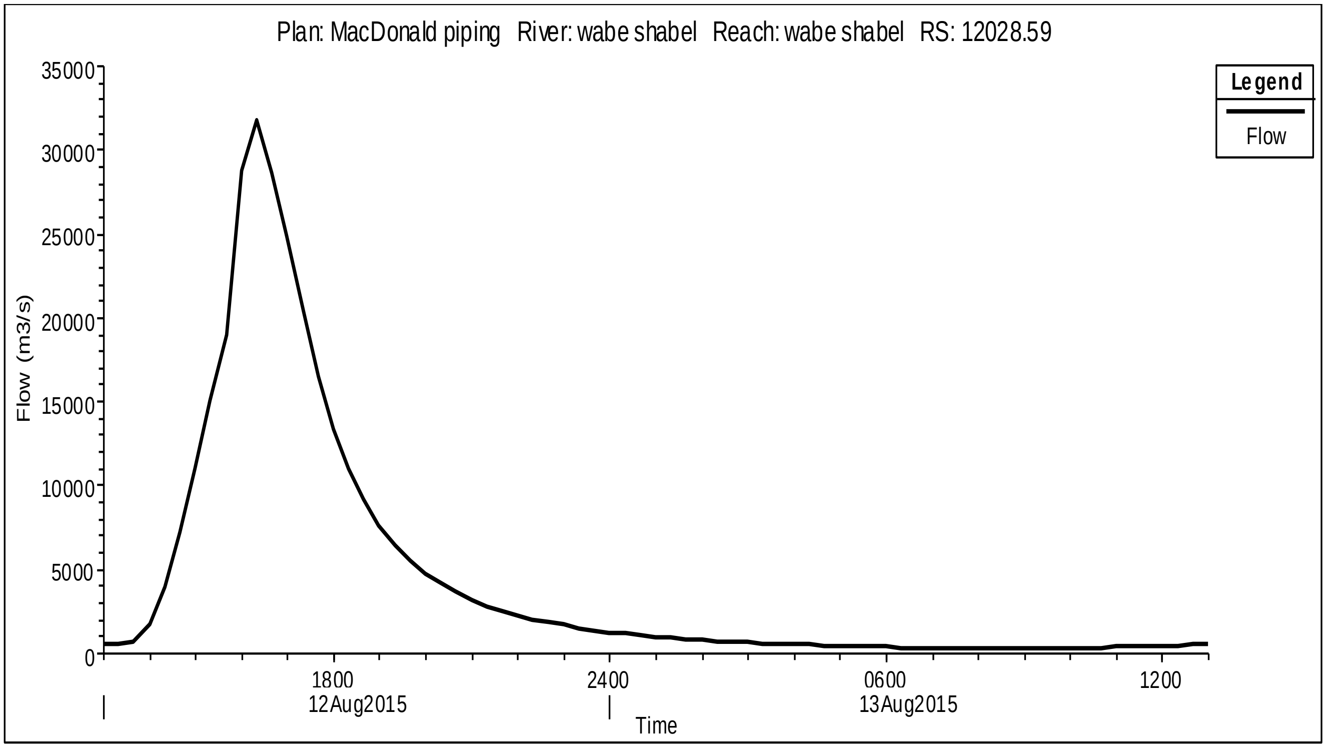


Figure 5.15: the discharge at the RS 12028.59 for piping

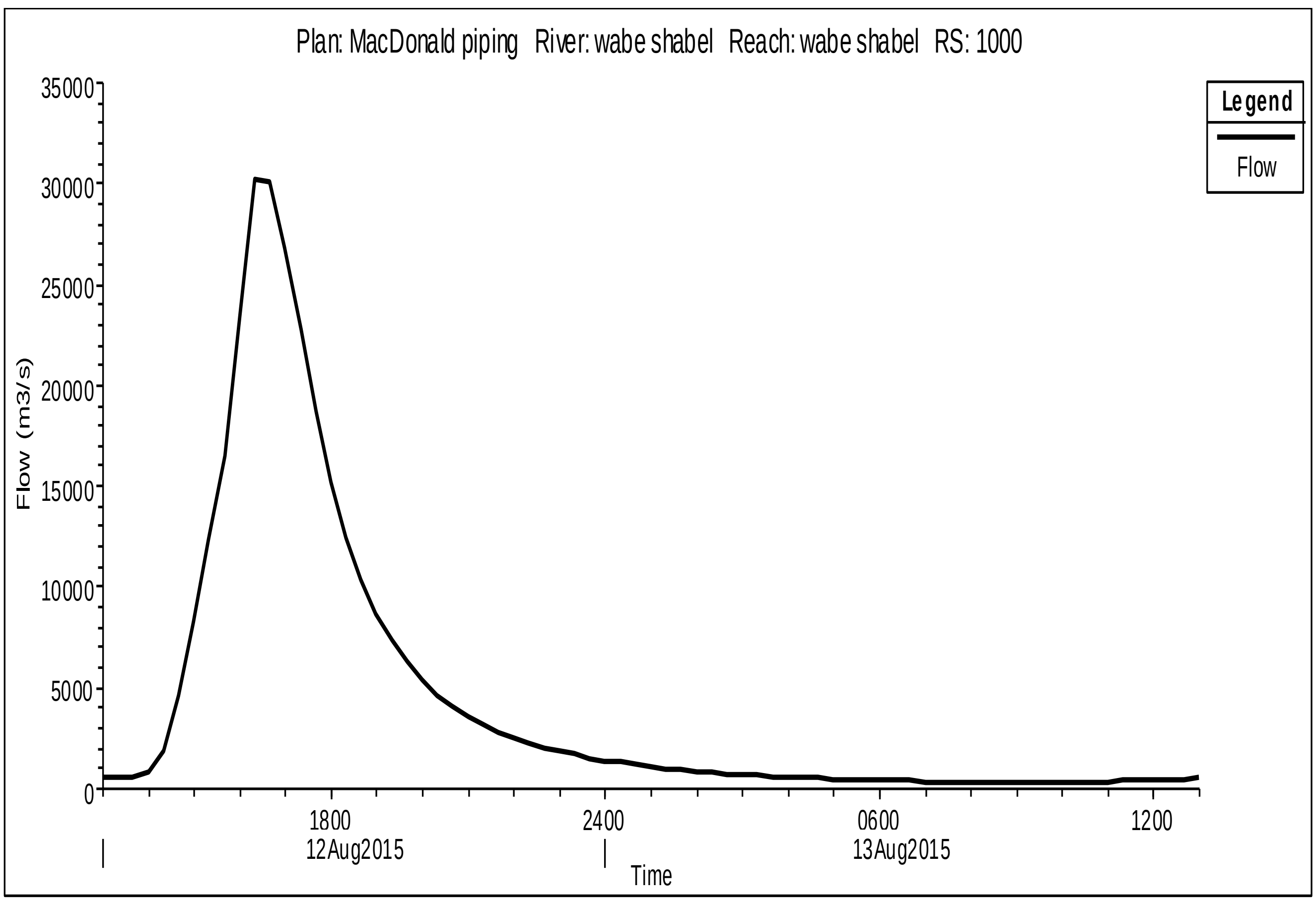


Figure 5.16: the discharge at the RS 1000 for piping

5.4. Inundation map for the downstream

The inundation map provides a description of the areal extent of flooding which would be produced by the dam break flood. It should also identify zones of high velocity flow and depict inundation for representative cross-sections of the channel.

For overtopping

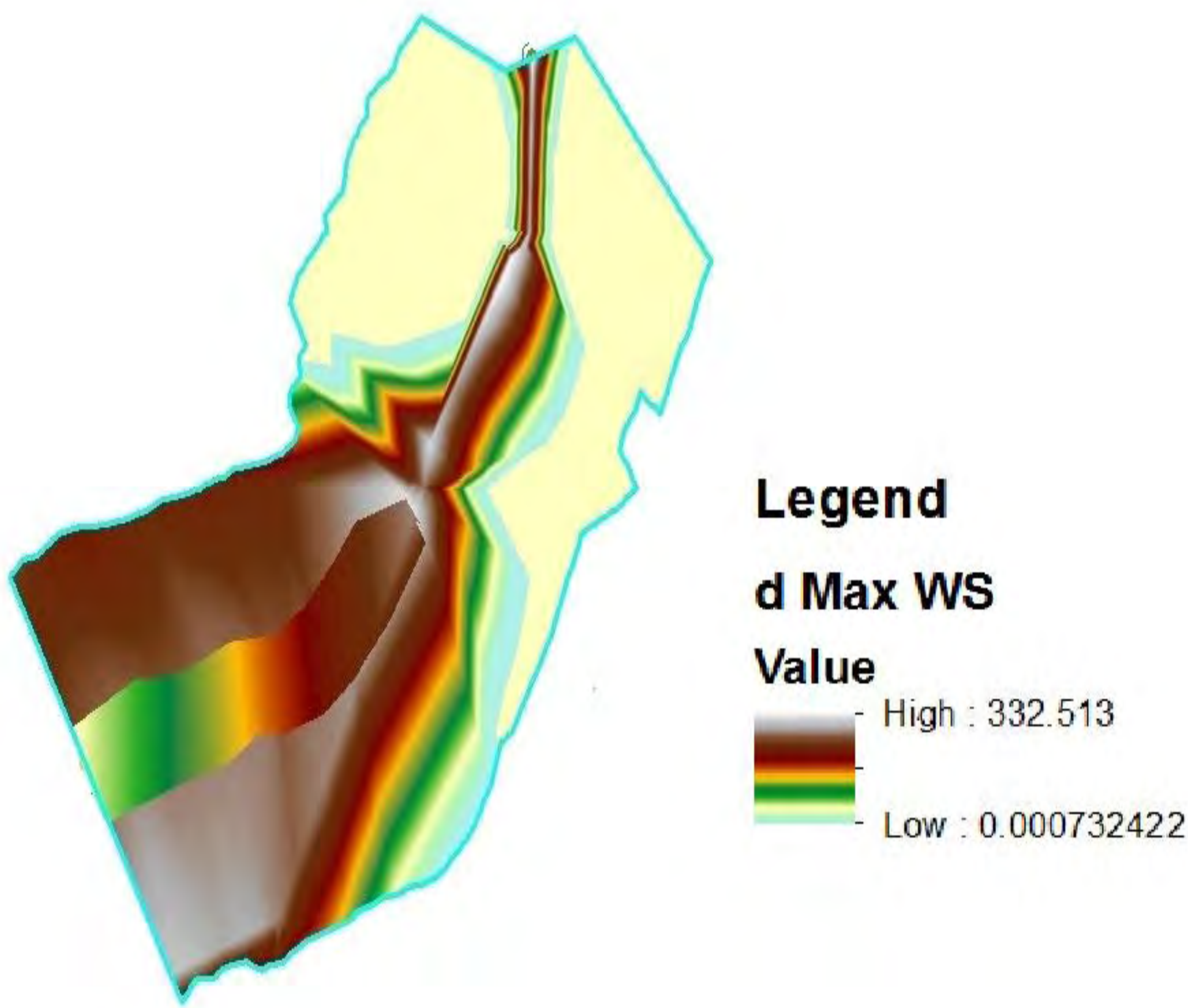


Figure 5.17: inundation map for overtopping failure and depth of water level in meter

For piping

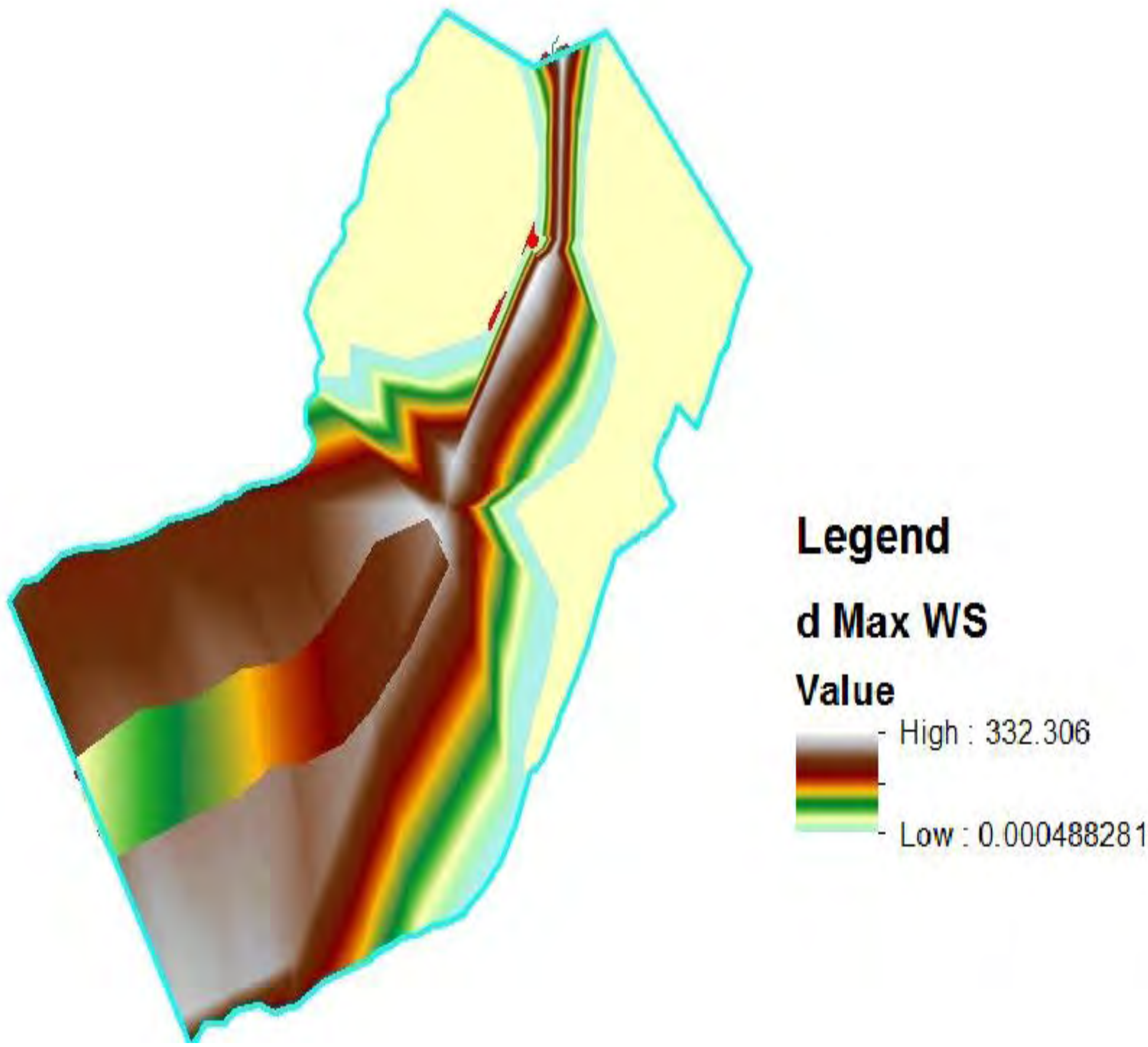


Figure 5.18: inundation map for piping failure and depth of water level in meter

5.5. Emergency Action Plan

The purpose of the Emergency Action Plan (EAP) is to safeguard lives and secondarily to reduce property damage in the event that Melka Wakana dam would fail. To carry out this mission, the EAP contains:

- Procedures to monitor Melka Wakana dam periodically.
- 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 Melka Wakana dam could cause significant damage to (all roads and isolated residences downstream of the dam within the danger reach) located downstream of the dam.

Operating procedure

The dam will be inspected periodically each year for maintenance and distress signals.

The dam observer will inspect the dam and 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 send massage to police to roadblock downstream roads and warn any isolated residences in the danger reach. The dam observer may contact to 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.

Preventative actions

Listed below are potential emergency actions which may prevent or delay the failure of the dam. They should be considered based on site-specific conditions, as well as the risk of failure and risk to employees.

Possible Actions to be taken in the event of:

Imminent Overtopping by Flood Waters:

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

Erosion of Dam by Seepage through the Embankment, Foundation, or Abutments:

- 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 a 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.

Supplies and resources

In an emergency situation, equipment and supplies may be needed. The following supplies and resources may be needed during an emergency:

- earthmoving equipment
- sand and gravel
- sandbags
- riprap
- pumps
- pipe
- Laborers.

CHAPTER SIX

CONCLUSIONS & RECOMMENDATION

6.1. Conclusions

Dam failure places populations at risk; however, tools exist to evaluate the contingencies. HEC-RAS used in concert with HEC-GeoRAS provide the capabilities to create a river hydraulics model, simulate a dam failure, and map the resulting flood wave. The main goal of this study was to create a flood hazard map for Melka Wakana Dam along with a flood protection measure framework. Based on these flood hydrographs, unsteady-flow simulations were performed in order to define areas where overtopping and piping will occur during large flood events. The hydraulic modeling results were incorporated into a representative flood hazard map. Based on the encountered hazard situation, a flood protection measure framework was developed as Emergency Action Plan (EAP).

The Melka Wakana Dam breach has been simulated for overtopping and piping breach. With a breach Bottom Width 493.76m and time of 3.79h.The simulated results reached peak discharges of 36,527.15m3/s and 32,627.70m3/s, for overtopping and piping breach respectively. The maximum discharge at the lower end, 22 km below the dam, was reduced to 35,179.22m3/s and 30,159.74m3/s, for overtopping and piping breach respectively and Dam overtopped by 18cm.

6.2. Recommendation

Dam breach analysis and inundation map for Melka Wakana Dam as a testing basis involved making a number of assumptions based on literature reviews and historic data. In the real world, there is a large degree of uncertainty associated with the breach parameters and breaching outflow estimation. It would be helpful to minimize the ambiguities associated with breach parameters estimation using different modeling software and analysis techniques for obtaining a wider range of dam and reservoir characteristics and downstream river characteristics data.

In this study, only one dimensional unsteady flow routing technique was used to carry out dam breach analysis. There are a number of assumptions in the modeling software. It would be helpful to utilize a different version of the software and enhance the findings of this study.

Finally, I would like to recommend the dam owner to give special attention to the Dam breach analysis and make a detail investigation by using the latest dam breach software's.

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