

**Dam Breach Analysis Using HEC-RAS and HEC-
GeoRAS: The Case of Kesem Kebena Dam**

By:

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December 2015



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A thesis submitted to the School of Graduate Studies of Addis Ababa University in Partial fulfillment of the Degree of Masters of Science in Civil Engineering under Hydraulic Engineering.

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Dr.-Ing. Asie Kemal (Advisor)

Date

ABSTRACT

Dam breach analysis is generally used to predict flood on the downstream of the dam but also it is crucial to predict dam breach parameters, outflow hydrograph and its downstream nature of propagation. Results from dam breach analysis can be used to protect downstream population and it can also be used while designing and implementing future infrastructure. Kesem Dam, found in the Kesem sub-basin of Awash Basin in Ethiopia has been selected as a case study dam in this paper.

Within this study the dam has been checked for both overtopping and piping using one dimensional river analysis model HEC-RAS and empirical equations are used to predict dam breach parameters for the use in the model. PMF inflow with a peak $9237.77\text{m}^3/\text{s}$ is used for overtopping failure. The spill way have adequate capacity for the flood due to the PMF and breaching of the embankment was not possible in HEC-RAS. Piping failure was simulated in HEC-RAS using breach parameters obtained from the empirical equations.

The modeling process was towards performing unsteady flow calculations in the intent of routing the breach outflow downstream of Kesem Dam from the dam up to the downstream boundary which is 60km from the dam. The models HEC-RAS and HEC-GeoRAS are used alternatively. HEC-GeoRAS extracts topographic data from Digital Elevation Model (DEM) and prepares geometric file in ArcGIS. The geometric file is imported into HEC-RAS. Using the geometric file and unsteady flow data (PMF inflow, initial flow and normal depth) unsteady flow calculation is performed in HEC-RAS. Geometric file with water surface elevation attached to it is exported to ArcGIS where HEC-GeoRAS uses it to prepare a flood map. The produced map is overlaid on an aerial map to see towns and infrastructures that are affected by the flood.

Key words: Dam Breach Analysis, Flood Mapping, HEC-RAS, HEC-GeoRAS

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LIST OF ACRONYMS

BEED: Breach Erosion of Earth Dam

BFF: Breach Formation Factor

DEM: Digital Elevation Model

FEMA: Federal Emergency Management Agency

FERC: Federal Energy Regulatory commission

GIS: Geographical Information System

HEC: Hydrologic Engineering Center

MOWIE: Minister of Water, Irrigation and Energy

PMF: Probable Maximum Flood

RAS: River Analysis System

SDF: Standard Project Flood

TIN: Triangulated Irregular Network

USBR: United States Bureau of Reclamation

USACE: United States Army Corps of Engineers

UTM: Universal Transvers Mercator

WWDSE: Water Works Design and Supervision Enterprise

1. INTRODUCTION

1.1. Background

While dams provide the ability to control the flow of fresh water and function to simplify our lives in many ways, they also pose an inherent and inevitable threat to the environment and to public safety. Since the creation of the first dams, dams have been failing due to unpredictable environmental conditions, poor engineering, or improper management. Unfortunately, when dams fail they often do so catastrophically because of the large amount of potential energy involved.

Dams are complex structures subjected to several forces that can cause failure, these forces re active over the entire life of the dam, and the fact that a dam has stood safely for years is not necessarily an indication that it will not fail. One of the forces inducing failure is seepage through the dam or its foundation. All dams seep, but if the seepage is too high in the dam it can cause a structural failure (“landslide” of the materials in the dam). If the seepage comes to the ground surface on or below the dam and exits too fast, it can carry soil out of the dam or foundation, and cause an internal erosion or “piping” failure. Another way a dam can fail is by being overtopped and washed out. Overtopping is the result of having inadequate emergency spillway capacity or a clogging of spillways.

Many efforts have been made to reduce the potential hazard of dams as well as to provide emergency action plans for the event of a dam failure. Dam breach analysis can provide basic information about flood events that can be beneficial in dam engineering, emergency action planning, and floodplain management.

Major rehabilitation of the dam is not normally necessary if the dam was designed in accordance with good engineering practice, was built using good construction standards, and is operated and maintained properly. Engineers generally agree that the design of a dam is not complete until after the dam has been built and the reservoir has filled with water. Design engineers should inspect their dam periodically after construction to ensure that the design is working and the structure is properly operated and maintained.

Different organizations and researchers have contributed their findings in the analysis of dam break and its consequence. The have derived regression equations based on data from historical

dam failure events that are used in predicting the breach geometry. This include Macdonald and Langridge Monopolies and Froehlich. Development of analytical models using the principle of hydraulics and sediment transport are also useful in simulating the breach process and downstream flooding.

The use of geographic information systems (GIS) has become more mainstream and data have become more readily available. In particular, the availability of terrain data has improved the proficiency with which skilled engineers can develop hydraulic models capable of simulating a dam breach scenario and evaluating the resultant flood wave.

U.S. Army Corps of Engineers River Analysis System (HEC-RAS) developed by Hydrologic Engineering Center which models dam failure scenarios and HEC- GeoRAS which is a set of Arc GIS tool which models development and analysis of the flooded area using GIS have been used in different dam breach analysis studies. And have provided information for floodplain managers and emergency management personnel which they use to protect against the loss of life and property damage.

Kesem dam and irrigation project is located 225 km east of Addis Ababa and 40 km NW of Metehara town. The project involves 90 m high rock earth fill dam to impound half a billion cubic meter of water to irrigate 20,000 hectare of land for sugar cane plantation (MOWR, 2007).

Since sugarcane plantations and small towns are present downstream of the dam, dam breach analysis should be done as a precaution for reasons that may result due to dam failure.

1.2. Problem statement

Several problems cause dam failure like overtopping, piping, earth quake, land slide etc. Due to this reasons our world have experienced some catastrophic dam failures. Like the Banqiao dam which failed in august 8, 1975 killed an estimate of 171,000 people and 11 million people lost their homes (Fish, 2013). As dams pose a serious threat to residents, businesses, infrastructures, landowners, crops etc. downstream of them, it has always been important to analyze the causes and results of dam failure. In Ethiopia, in contrary to the development of dams, such per event analysis is not being carried out by designers or researchers. Hence, dam breach modelling is vital to identify the possible causes of dam failure, simulate the breaching process so that design parameters can be reviewed, map the area that shall be flooded in order

to demarcate prone areas while planning the downstream are for various infrastructures, alert concerned bodies to a precaution on dam safety plans and formulate a hazard management system.

Kesem dam impounds half a billion cubic meter of water to irrigate 20,000 hectare of land. Kesem dam is classified under high hazard dam since there are towns and wide irrigation lands which if the dam fails leads to loss of life and property.

1.3. Objective of the research

General objective:-

- ✓ Estimating the breach outflow of Kesem dam and preparing a flood map for the use in flood mitigation.

The specific objectives of this research are as follows

- ✓ Identify dam failure scenarios due to which Kesem dam may fails.
- ✓ Estimate dam breach parameters: define the size, shape and timing of the breach using appropriate empirical formulas.
- ✓ Rout the outflow hydrograph from the breached dam throughout downstream from the dam to the downstream boundary of the simulation using HEC-RAS (US Army Hydrologic Engineering Center-River Analysis System)
- ✓ Map the inundation area with ARC GIS and HEC-GEO RAS.

1.4. Significance of the research

Dams are an important part of this nation's infrastructure, providing flood control, water supply and irrigation and hydropower 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. This research is going to be used by flood plain managers to save lives and property, if Kesem dam fails in the future.

1.5. Scope and Limitation of the research

This thesis covers dam breach parameter estimation which includes dam breach dimensions and breach formation time. Dam breach scenario will be selected and outflow hydrograph from the breach is routed.

One dimensional flood simulation model which is based on unsteady flow equations will be used for analysis. The flood resulting from the breach will be mapped using software's.

Since high resolution Digital Elevation Model (DEM) is not available in our country Digital Elevation Model with a resolution of only 30x30m grid will be used to prepare geometric data which represents elevations on the study area with some error.

1.6. Structure of the thesis

This thesis is organized in six chapters. Chapter one deals with the general introduction, statement of the problem and objective of the study. Chapter two gives the literature review on dam breach analysis, hazard classification, topography of the study area, and hydrology of the study area. Chapter three gives an overview on methodology of the thesis, it includes location of the study area, data collection, processing and analysis. In Chapter four is the result and discussion part of the work namely dam breach analysis, unsteady flow analysis and flood mapping. Chapter five is about conclusion and recommendation. And finally in Chapter six list of the References are included.

2. LITERATURE REVIEW

In this part of the work literatures related to Dam breach analysis and the area under study (documents which can help to know topography, hydrology and others) are reviewed.

2.1. Dam breach History

A dam is a barrier across flowing water that obstructs, directs or slows down the flow, often creating a reservoir, lake or impoundment. Dams are considered “installations containing dangerous forces” due to the massive impact of a possible destruction on the civilian population and environment. Hundreds of dams have failed and every year many dikes breach due to high flow in the river, sea storm surges, etc. often leading to catastrophic consequences (Zagonjulli, 2007). List of the major dam failures are shown on the table below.

Figure 2-1 List of major dam failures in the world

Dam	year	Location	Fatalities	Details
Marib Dam	575	Sheba, yemen		Unknown cause, possibly neglect. The consequent failure of the irrigation system provoked the migration of up to 50,000 people from Yemen.
Puentes Dam	1802	Lorca, Spain	608	1,800 houses and 40,000 trees destroyed.
Bilberry reservoir	1852	Holme, valley, United Kingdom	81	Failed due to heavy rain.
Dale Dike Reservoir	1864	South, Yorkshire, united kingdom	244	Defective construction, small leak in wall grew until dam failed. More than 600 houses were damaged or destroyed.
Mill River Dam	1874	Williamsburg, United states	139	Lax regulation and cost cutting lead to an insufficient design, which fell apart when the reservoir was full. 600 million gallons of water were released, wiping out 4 towns and making national headlines. This dam break lead to an increased regulation of dam construction.
South Fork Dam	1889	Johnstown, United States	2,209	Blamed locally on poor maintenance by owners; court deemed it an “Act of God”. Following heavy rainfall.
Walnut Grove dam	1890	Wickenburg, United States	100	Heavy snow and rain following public calls by the dam’s chief engineer to strengthen the earthen structure.
Austin Dam	1911	Austin, United States	78	Poor design, use of dynamite to remedy structural problems. Destroyed paper mill and much of the town of Austin.
Tigra Dam	1917	Gwalior, India	1,000	Failed due to water infiltrating through foundation. Possibly more fatalities.
Gleno dam	1923	Province of Bergamo, Italy	356	Poor construction and design.
St. Francis Dam	1935	Santa Clarita, United States	600	Geological instability of canyon wall that could not have been detected with available technology of the time.
Secondary Dam of Sella Zerbino	1942	Molare, Italy	111	Geological unstable base combined with flood.
Mohne Dam	1943	Ruhr, Germany	1,579	Destroyed bombing during operation Chastise in world war 2.

Kurenivka Mudslide	1961	Kiev, Ukraine	1,500	Caused by heavy rains
Panshet Dam	1961	Pune, India	1,000	Dam burst due to pressure of accumulated rain water.
Vajont dam	1963	Monte Toc, Italy	2,000	The dam did not collapse but 110km/h landslide fill the reservoir, and water escaped over the top of the dam.
Sempor Dam	1967	Central Java Province, republic of Indonesia	2,000	Flash floods overtopped the dam during construction.
Banqiao Dam Shimantan Dam	1975	Zhumadian, China	171,000	Extreme rainfall, 11 million people lost their homes. worst dam failure
Machchu-2 Dam	1979	Morbi, India	5,000	Heavy rain and flooding beyond spillway capacity.
Kantale Dam	1986	Kantale, Sri Lanka	180	Poor maintenance, leakage and consequent failure.
Koshi Barrage	2008	Koshi Zone, Nepal	250	Heavy rain. The flood affected over 2.3 million people.
Situ Gintung Dam	2009	Tangerang, Indonesia	98	Poor maintenance and heavy monsoon rain.
Germano mine tailings Dams	2015	Minas Gerais, Brazil	13	Two tailings dam failed. One village destroyed, 600 people evacuated, 19 missing. Iron waste sludge polluted Doce River.

With most structural failures damage is limited to an area in the immediate vicinity of the structure, but the breaching of the dam and the consequent uncontrolled release of the impounded reservoir water can cause destruction over a large area downstream of the dam. The structural stability and security of such dams, therefore, is of major importance for public safety (J andrew charles, 2011).

2.2. Dam Breach Analysis

Dam breach analysis is the simulation of a breach hydrograph (flow through the breach in the dam versus time) and the subsequent downstream inundation. In recent years, dam safety draws increasing attention from the public. This is because floods resulting from dam failures can lead to devastating disasters with tremendous loss of life and property, especially in densely populated areas. Analysis of dam failures is of critical importance for disasters prevention and mitigation. Hence, a robust understanding of the characteristics of dam failures (e.g., failure mode, cause, and key influence factors) is needed (Y.Xu, 2007).

Studies have been carried out worldwide to understand causes of failure for different types of earth dam (zoned earth fill dam, earth fill dam with core wall and homogeneous earth fill dam). The world data as of 2000 indicate that there are about 50,000 large man-made dams in operation (J. Nemmert, 2010). Approximately 80% of the world dams are earth or rock fill dams (Zhang, 2010). In the past few years more than 1609 dam failure cases were seen and compiled these failure cases into a database. Among these cases, 66% are earth dams (Zhang, 2010).

2.2.1. Dam breach scenario

Dam breach inundation studies usually assume one of two failure scenarios “Fair Weather” or “sunny day” Failure and Hydrologic Failure.

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.

Hydrologic breaches that occur with extreme precipitation and runoff are termed “rainy day” or hydrologic failures.

Based on approximately 900 cases of dam failures, a statistical analysis of the failure characteristics has been conducted for earth dams. The modes and causes of failure for carefully classified earth dams, as well as their relation to potential locations at risk, have also been studied (Y.Xu, 2007). Several features can be observed:

1. The most common causes of earth dam failures are overtopping and piping in the dam body or foundation.
2. For homogeneous earthfill dams and zoned earthfill dams, piping in the dam body/foundation is a dominant failure cause. Overtopping is also identified as an important failure cause.
3. For earth fill dams with core walls, failures are mostly attributed to overtopping. Piping in the dam body/foundation appears to be less likely.
4. For homogeneous earthfill dams and zoned earthfill dams, spillways, foundations, and downstream slopes are believed to be potential locations at risk for overtopping failure; while any part of the dam body/foundation can be a potential location at risk for piping failure.
5. The overtopping failure of earthfill dams with core walls is similar to that of homogeneous/zoned earth fill dams, except that the core wall may erode more slowly.
6. For the piping failure of earth fill dams with core walls, foundations, abutments, or their interfaces with the dam body are noticeable potential locations at risk; the piping paths may still pass through the dam due to either hydraulic fracturing of the core wall or poor contact with the embedded structures.

Overtopping Failure: In general, during an overtopping failure of an earthen dam, a head cut erosion process will first starts on the downstream side of the dam embankment. While water is going over the dam crest, the dam crest acts like a broad-crested weir. The head cut begins to cut into the dam crest, the weir crest length will become shorter, and the appropriate weir coefficient will trend towards a sharp crested weir value. When the head cut reaches the upstream side of the dam crest, a mass failure of the upstream crest may occur, and the hydraulic control section will act very much like a sharp-crested weir. The head cut will continue to erode upstream through the dam embankment, as well as erode down through the dam and widen at the same time (Brunner, 2014).

Piping failure: Water is seeping through the dam at a significant enough rate, such that it is initially eroding material and transporting it out of the dam. As the material is eroded, a large hole is formed, thus able to carry more water and erode more material. The movement of water through the dam during this process is modeled as a pressurized orifice type flow. During piping

flow process, erosion and head cutting will begin to occur on the downstream side of the dam as a result of flow exiting the pipe. As the piping hole grows large, materials above the hole will begin to slough off and fall into moving water. The head cutting and sloughing processes will continue to move back towards the upstream side of the dam while the piping hole continues to grow simultaneously. If the piping hole is large enough, the weight of the material above the hole may be too great to be maintained, and a mass caving of material will occur. This will result in a large rise in outflow through the breach and will accelerate the breaching process. Also at this point, the hydraulics of the flow transitions from orifice type flow to open air weir type flow. The head cutting and erosion process then continues back through the dam, as well as downward. Additionally, the breach will be widening. Depending on the volume of water behind the dam, the breach may continue to cut down and widen until the natural channel bed is reached. Then the breach will go into a widening phase (Brunner, 2014).

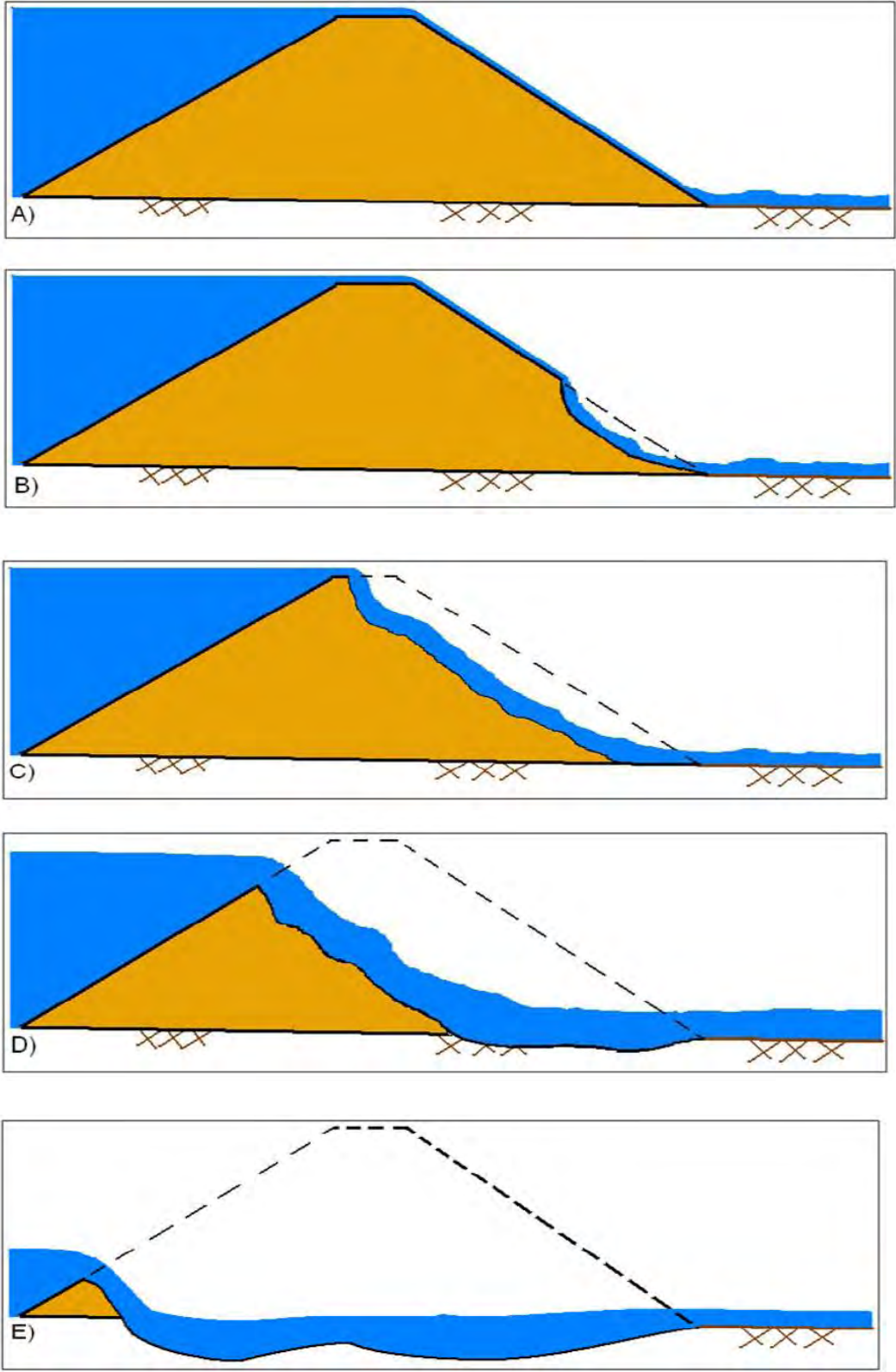


Figure 2-2 Breach process for an overtopping failure

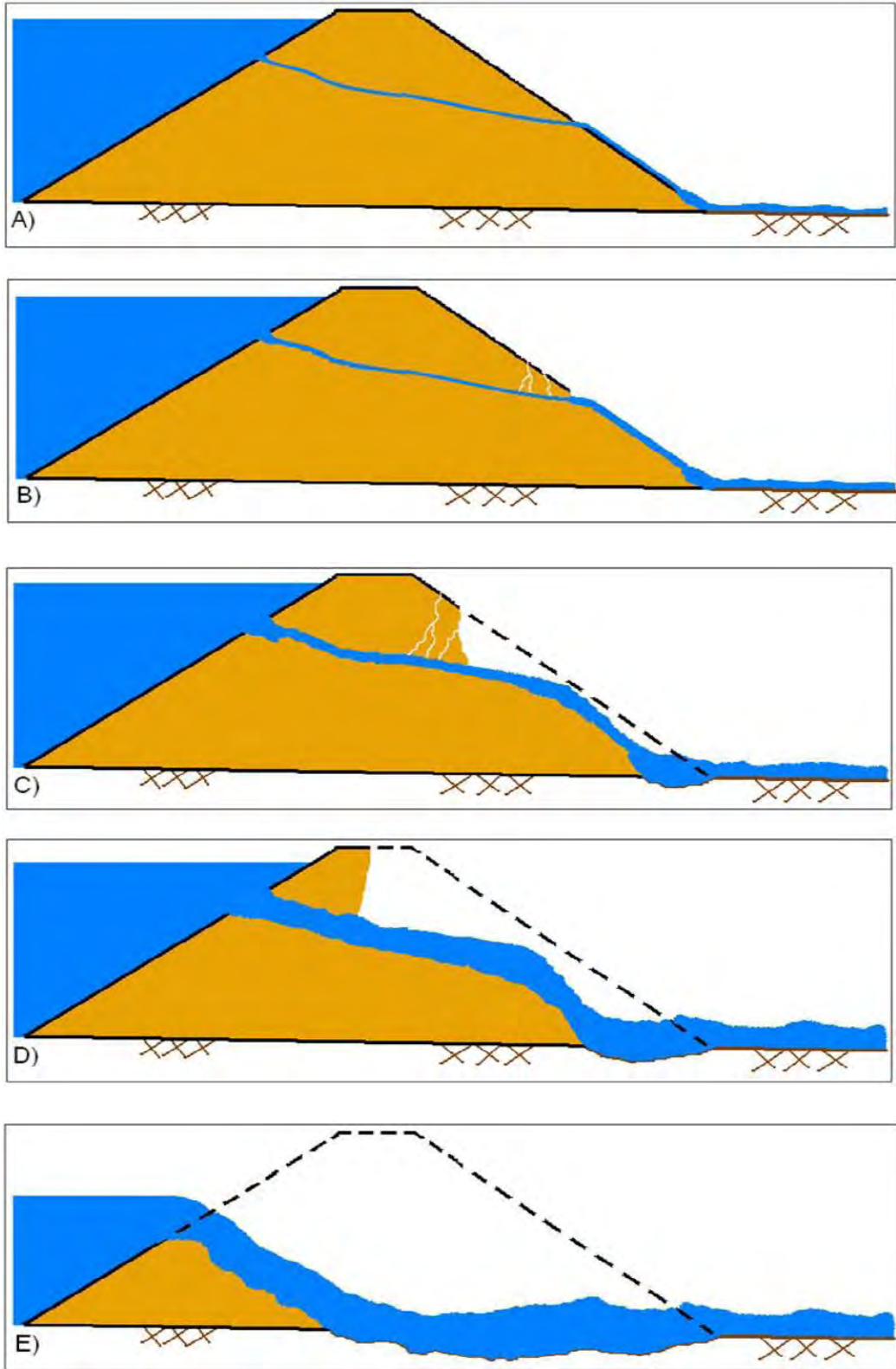


Figure 2-3 Breach process for a piping failure

2.2.2. Dam breach parameter estimation

The term breach parameters 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 (Wahl, 1998).

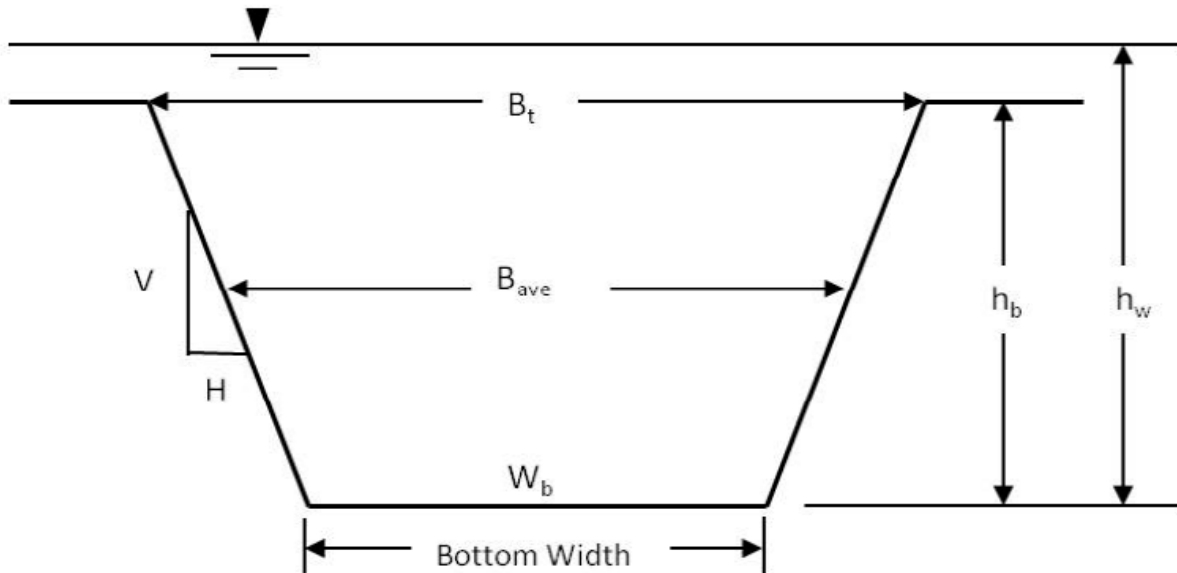


Figure 2-4. Dimensions of a dam breach

The estimation of dam breach location, dimensions and development time are crucial in any assessment of a dam's potential risk. This is especially true in a risk assessment where dams will be ranked based on the potential for loss of life and property damage. The breach parameters will directly affect the estimation of the peak flow coming out of the dam, as well as any possible warning time available to downstream locations. Unfortunately, the breach location, size, and formation time, are often the most uncertain pieces of information in a dam failure analysis (Brunner, 2014).

The dam breach dimensions and breach development time must be estimated for every failure scenario that will be evaluated. This requirement includes different failure modes as well as different hydrologic event. The breach parameter associated with a PMF (probable maximum flood) hydrologic event will be greatly different from the breach parameters for a sunny day failure at a normal pool elevation. Therefore, for each combination of pool elevation (hydrologic event) and failure scenario, a corresponding set of breach parameters must be developed.

There are numerous methods for predicting the breach parameters that serve as input to an analysis using different models. Three basic approaches can be identified: - 1) Comparative analysis of similar case studies 2) The use of predictor equations based on numerous case studies. 3) The use of a physically based dam breach simulation model that uses principles of hydraulics and sediment transport to simulate the development of the breach. This approach is more difficult, but also offers the potential for more detailed results, such as prediction of breach initiation time and prediction of intermediate breach dimensions as well as ultimate breach parameters (Wahl, 1998).

The empirical approach relies on statistical analysis of data obtained from documented failures. The four most widely used and accepted empirically derived enveloping curves and/or equations for predicting breach parameters are: - 1) MacDonald & Langridge – Monopolis (1984) 2) USBR (1988) 3) Von Thun and Gillette (1990) 4) Froehlich (1995a, 1995b, 2008). These methods have reasonably good correlation when comparing predicted values to actual observed values (CDWR, 2010).

Froehlich (1995) utilized 63 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rock fill data sets to develop a set of equations to estimate average breach width, side slopes, and failure time (Gee, 2009). Froehlich (2008) updated the breach equation based on addition of new data. In the application of these equations reported herein, the height of the breach is calculated by assuming that the breach goes from the top of the dam to the natural ground elevation at the centerline of the breach location. Froehlich method is more preferable since it is only dependent on volume of the reservoir, height of the breach and assumed side slope. The method also distinguishes between piping and overtopping failures using a variable coefficient termed the Failure Mode Factor, K_o . The Froehlich method breach development time does not distinguish between overtopping or piping breach failure modes. The development time estimate is inversely related to the breach height while being directly related to the reservoir volume. This means dams with greater height tend to produce shorter failure times for a given reservoir volume which appears to be a valid conclusion considering the greater head driving the breach formation (CDWR, 2010).

Froehlich's equations are as follows:

$$\text{Average breach width: } B_{avg} = 0.27K_o V_w^{0.32} H_b^{0.04} \quad (\text{Froehlich 2008})$$

Where B_{avg} Average breach width

K_o Failure mode factor 1 for piping and 1.3 for overtopping

V_w Volume of the reservoir above the bottom of the breach

H_b Breach height which is the vertical distance from the dam crest to the breach invert.

$$\text{Breach Development Time: } T_f = 63.2 \left(\frac{V_w}{gH_b^2} \right)^{0.5}$$

Where T_f Breach development time in hours g Gravitational acceleration

Froehlich recommends breach side slope of 0.7:1 (horizontal: vertical) for piping and 1.0:1 for overtopping.

The MacDonald & Langridge-Monopolis method computes a volume of embankment eroded during breach formation, based on the product of the reservoir volume (V_w) and maximum water depth (H_w). This product, termed the Breach Formation Factor (BFF), loosely represents the erosive potential of the water stored in the reservoir. The breach dimensions are calculated based on the volume of embankment material eroded and the dam geometry. This method considers the dam geometry (height, crest width, and embankment slopes), and the breach development time computed is directly related to the embankment volume eroded (CDWR, 2010). MacDonald & Langridge-Monopolis (1984) Empirical formula is as follows.

$$\text{Volume Eroded } V_{er} = 0.0261 BFF^{0.769} \quad (\text{Best fit all data})$$

$$\text{Volume Eroded } V_{er} = 0.00348 BFF^{0.852} \quad (\text{Rock fill})$$

$$\text{Bottom Breach Width } B_b = \frac{V_{er} - H_b \left(CZ_b + \frac{H_b Z_b Z_3}{3} \right)}{H_b \left(C + \frac{H_b Z_3}{2} \right)}$$

$$\text{Breach Development Time } T_f = 0.0179 V_{er}^{0.364} \quad \text{Breach Side slopes 0.5:1}$$

$$\text{BFF} = V_{out} * H_w$$

2.2.3. Dam breach analysis using HEC-RAS and HEC-GEORAS

Hydrologic Engineering Center's River Analysis System (HEC-RAS) is a one-dimensional river hydraulics model used for steady flow and unsteady-flow water surface profile computations through a network of open channels (HEC, 2010).

In dam break study HEC-RAS is used to route the inflow hydrograph throughout the reservoir and the outflow hydrograph of the breach throughout the downstream of the river.

HEC-RAS can be used to route an inflowing flood hydrograph through the reservoir with any of the following three methods.

- One-dimensional unsteady flow routing (full Saint Venant equation)
- Two-dimensional unsteady flow routing (full Saint Venant equation or diffusion wave equation); or
- With level pool routing

Full unsteady flow routing (one or two dimensional) is more accurate for both with and without breach scenarios since it can capture the water surface slope through the pool as the inflowing hydrograph arrives, as well as the change in the water surface slope that occurs during a breach of the dam (Brunner, 2014).

HEC-RAS can be used to model the reservoir using fully dynamic wave routing. If it is not possible, or reasonable to perform full dynamic wave routing through the reservoir, or if the presumed difference between level pool routing and dynamic routing is small, then level pool routing can be performed with HEC-RAS.

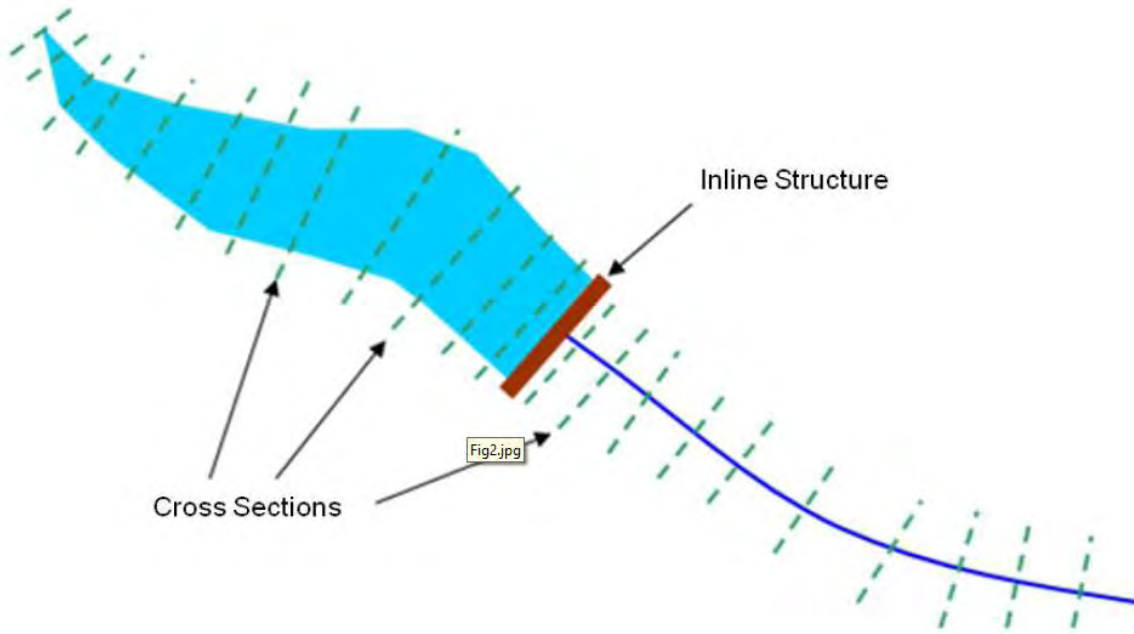


Figure 2-5 One-Dimensional Full Dynamic Routing

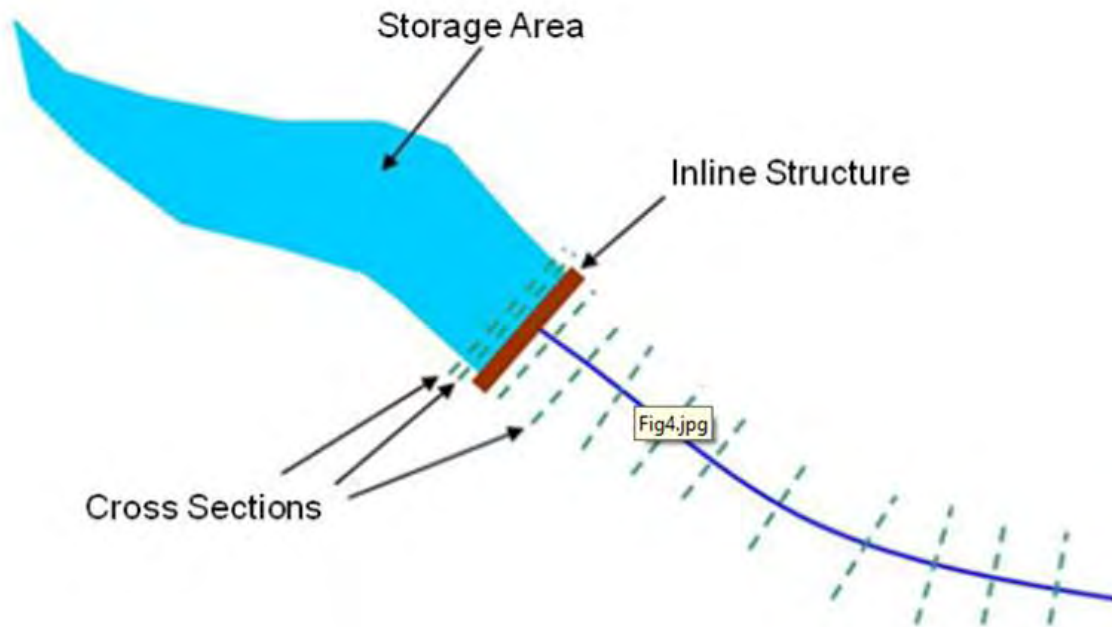


Figure 2-6 Level Pool Routing

HEC-RAS can model the downstream area as a combination of one-dimensional stream and storage area; a combination of one-dimensional stream, storage areas, and two-dimensional flow area; or as a single two-dimensional flow area (Brunner, 2014).

When developing an unsteady flow model for dam break application the following list of things should be considered (Brunner, 2014);

- Cross section spacing and hydraulic properties
- Computational time step
- Manning's Roughness Coefficient
- Modeling bridge and culvert crossings
- Modeling steep streams
- Drop in the bed profile
- Initial condition (low flow)
- Downstream boundary condition

HEC-GeoRAS is a geographic river analysis system developed using ArcGIS Desktop and ArcGIS Spatial Analyst and 3D Analyst extensions. The geodatabase design supports analysis of spatial data for hydraulic modeling and floodplain mapping. GeoRAS uses ArcGIS Desktop to develop spatial data input for HEC-RAS models from digital terrain models and other GIS datasets. After the model results are calculated in HEC-RAS, they can be post processed in GeoRAS, then the floodplain depths and extents can be mapped.

Dam break is a complicated and comprehensive process and the actual failure mechanics are not well understood. Neither current physical based models nor empirical models could fully explain dam break mechanisms and impacts (Xiong, 2011). The dam break tool in HEC-RAS was applied to Foster Joseph Sayers Dam break simulation and analysis based on given geometry data. The dam break due to piping elongates the time period of high water surface level, which increases the duration of risk. However, the dam break does not increase the downstream maximum water surface elevation significantly at previous design PMF. Foster Joseph Sayer dam break has greater impact on the downstream location where is nearer the dam in accordance with the comparison of the hydrographs at different locations (Xiong, 2011).

An analysis of dam failure models provides a scenario generating tool for identifying the resulting hazards. Floodplain managers and emergency management personnel may then utilize the resulting contingencies to protect against the loss of life and property damage. 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. Because of the availability of digital terrain data and processing capabilities, GIS is well suited to assist in performing dam failure analysis. The proper analysis of the hazards associated with dam failure will assist in land use planning and in developing emergency response plans to help mitigate catastrophic loss to human life and property (Cameron T., 2008).

2.3. Hazard Potential Classification

Hazard Potential Classification is a system that categorizes dams according to the degree of adverse incremental consequences of a failure or mis-operation of a dam. The hazard potential classification does not reflect in any way on the current condition of the dam (e.g., safety, structural integrity, flood routing capacity) (FEMA, 2004).

According to FEMA Three classification levels are adopted as follows: LOW, SIGNIFICANT, and HIGH, listed in order of increasing adverse incremental consequences. The classification levels build on each other, i.e., the higher order classification levels add to the list of consequences for the lower classification levels.

Dams assigned the low hazard potential classification are those where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses, Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns and Dams assigned the high hazard potential classification are those where failure or mis-operation will probably cause loss of human life.

When selecting hazard class for a dam the following considerations must be taken into account (Bruce, 2007).

- Population at Risk
- Property Damage and Economic loss

- Environmental Damage
- Current Development

2.4. Topography

The Kesem river catchment to dam site covers about 3000 km² and extends from an altitude of almost 3600 m down to 860 m elevation (WWDSE, 2007). It rises on the high Ethiopian plateau and descends the western scarp of the Great Rift Valley to join the Awash.

The most prominent feature of the Kesem valley is the steepness of slopes linking the plateau of the north, west and south extremities of the catchment with the gorges and canyons of the central area. The channels of the upper Kesem, narrow and highly sinuous features in fine sediments, cascade into narrow, flat-floored trenches where they braid amongst the boulder and cobble bed materials.

For a large part of the central Kesem catchment the valley floor broadens, up to half a kilometer wide, allowing a complicated braided channel pattern. Channels seldom impinge on the bordering steep slopes. Often, in the upper parts of this section, traditional irrigated agriculture interposes between the channels and side slopes. Very little material appears to be contributed by the bank erosion in this section. For the final section of its course the character of the Kesem changes once more as it becomes constrained in narrow gorges and canyons, interrupted at irregular intervals by wider reaches which have become depositional areas for coarse fluvial sediments (WWDSE, 2007).

The channels are narrow and highly sinuous at higher levels before cascading into narrow flat floored trenches. The dam site lies on a steep sloped valley. The topography of the site has been shaped by tectonic activity with subsequent erosion. The slopes on the valley walls range 70° to 85° forming plateau surface at the top of the abutments. The study area has a wide variety of topographic features ranging from Moderate to high hills, faulted lava platform steps, dissected gorges and high volcanic pediments hills, an upland plateau, and escarpments, deep dissected gorges.

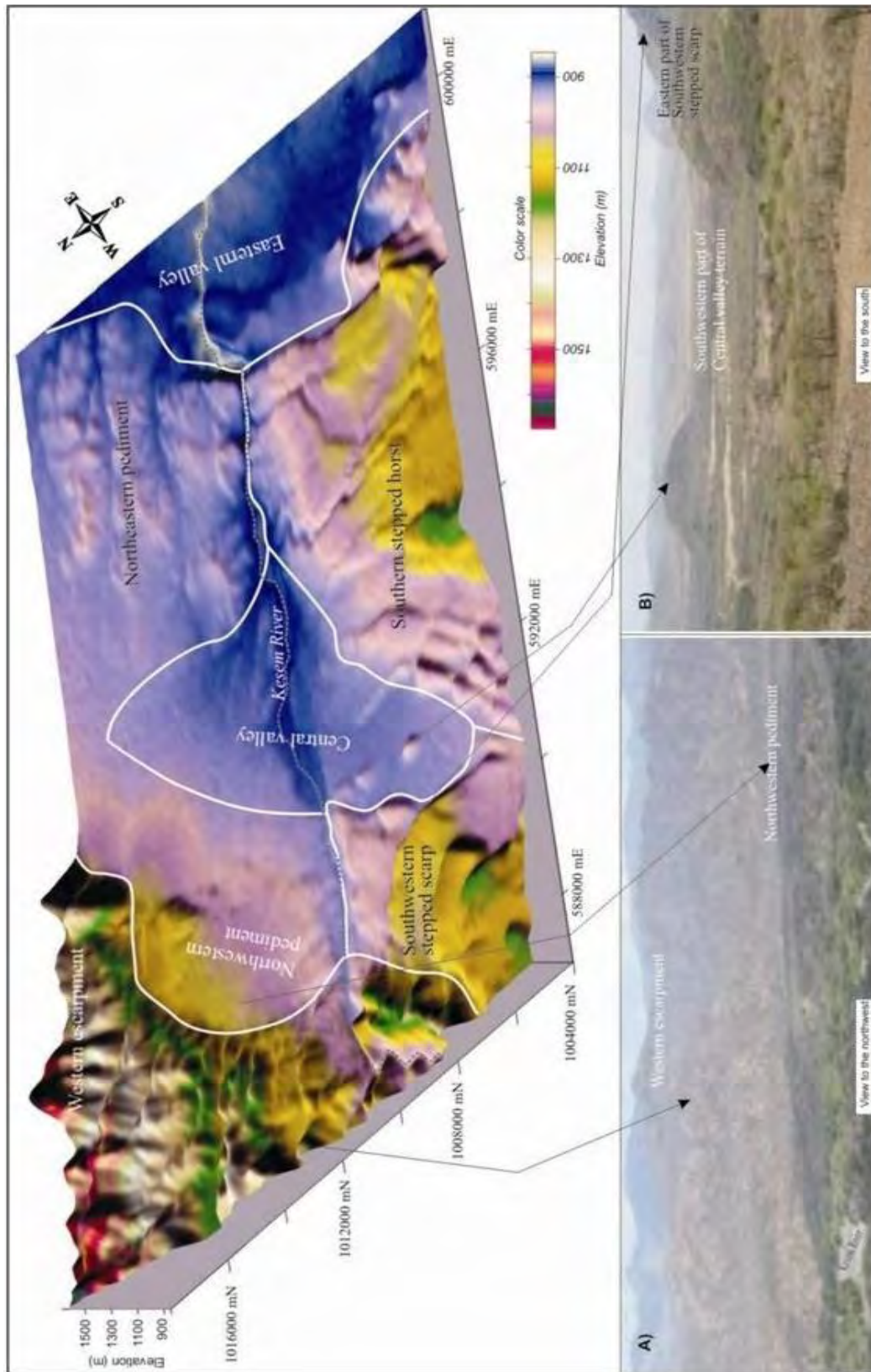


Figure 2-7 Surface Topographic Map of the Study Area

2.5. Hydrology

2.5.1. Hydrometeorology

A vivid coverage of the climatology of Ethiopia has been made in the Report on Survey of the Awash Basin by the Food and Agricultural Organization of the United Nations (1965), in an exclusive volume on Climatology and Hydrology (Vol. III). As per the detailed study, the country lies on the boundary between the subtropical high pressure belt and the actual tropical zone, that is, the area affected for at least part of the year by the inter-tropical front. The relief of east Africa is characterized by the great depression of the Rift, running approximately north-south between 30° E and 40° E and marked by lakes, and the Awash valley. This depression forms the boundary of the meteorological influence of maritime air masses originating in the Indian Ocean.

Movement of air masses from three zones influence the climate of east Africa the zone east of rift, the zone west of rift and the awash valley (WWDSE, 2007).

With respect to the zone east of Rift, Indian Ocean air stream, which is cool and damp but stable, affects the eastern tip of Africa. This stream does not give rise to any large scale precipitation except some local showers. In summer, the inter tropical front moves north and crosses this zone. Rainfall occurs along a succession of fronts coming from the southwest. This rainfall affects vast areas but just to the east of the Rift.

In the zone west of the Rift, the effects of north easterly continental air stream controlled by the large Egyptian zone of high pressure are felt in the winter. This cool air stream of desert origin gives the dry season. From the month of March onwards, the inter tropical front rapidly reenters the northern hemisphere and heavy rainfall accompanies these fronts and affects the high lands in the southwestern of the country.

In the middle and lower Awash also, the above processes continue to occur. Nevertheless, the position of the valley in leeward side of the high ground, considerably reduces the rainfall, because of the faults in the Rift. Further north, the Egyptian and Arabian subtropical anticyclones help to accentuate the desert character of the land by the divergence and subsidence with which they are associated.

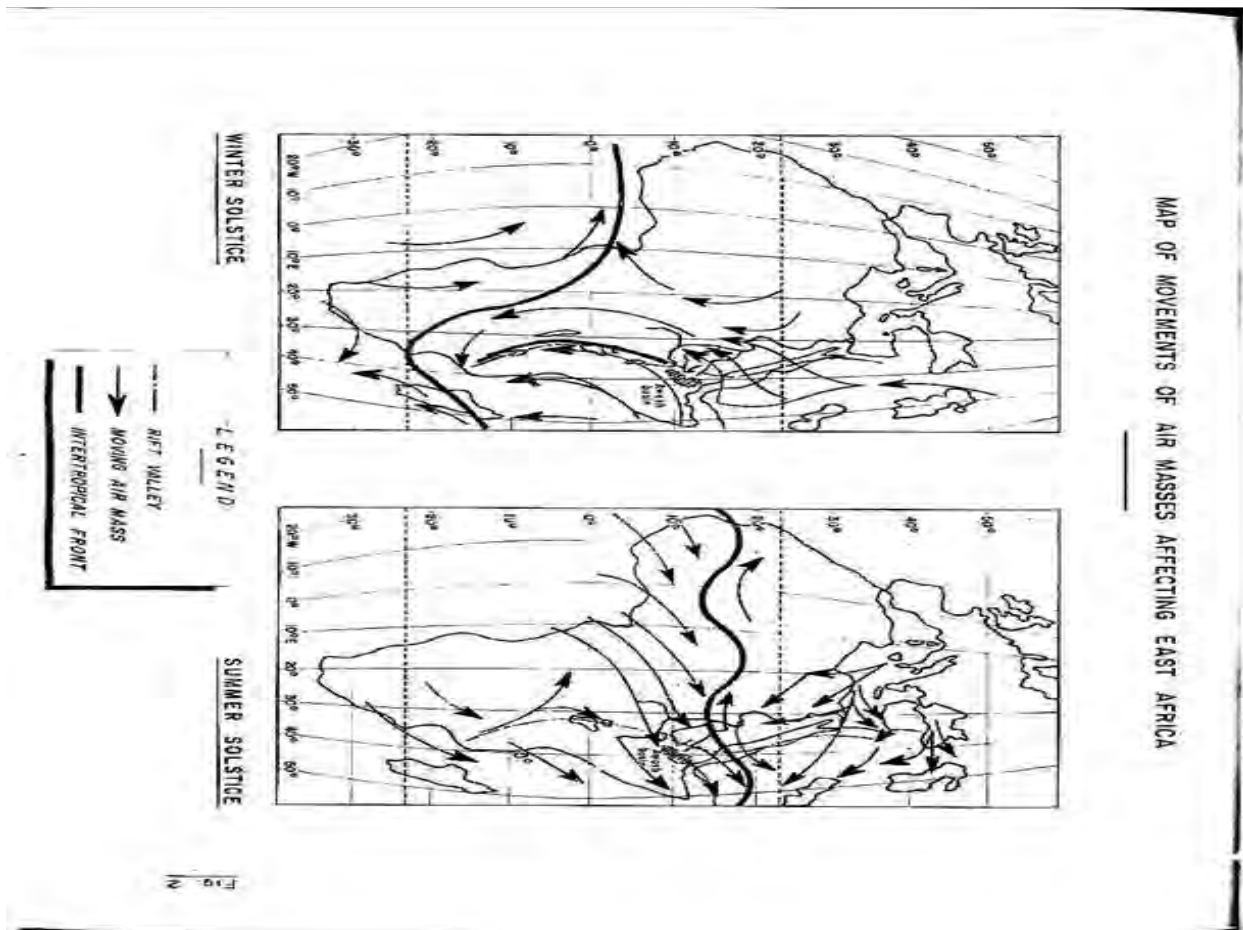


Figure 2-8 Map of Movement of Air Masses Affecting East Africa

Daily rainfall data were obtained for 14 stations in and around Kesem catchment and are used to estimate the average rainfall in the study area (WWDSE, 2007). The long-term rainfall station at Addis Ababa was also included in the study. Of these, five stations, namely, Sheno, Shola Gebeya, Balchi, ChefaDonsa and Aleltu are situated within the catchment. The Hydrometric network of awash basin is shown in Figure 2.8.

2.5.2. Inflow Design flood

Design floods are the hypothetical floods, which are adopted as the basis for the design of engineering structures on or along the streams or rivers (WWDSE, 2007). The cost and risk involved in particular design application determine the magnitude of this flood.

From the hydrologic analysis point of view, the design floods could be classified into two groups (WWDSE, 2007). In the first one, the design flood is conceptualized on hydrologic considerations such as the “Probable Maximum Flood” (PMF) and “Standard Project Flood” (SPF). The PMF is based on the concept, that there exists some kind of upper limits on flood producing factors like storm rainfall and/or snow melt, catchment type and wetness. Such floods are analyzed by detailed analysis of hydrological and hydro meteorological factors, using physics based methods of analysis. The concept of SPF is not as definite, but is based usually on maximum experienced values of the causative factors. In the second group, the statistically derived floods are included with the associated Return Periods (exceedance probability). A T year return period flood, by definition is that flood magnitude (peak, volume or any other element of flood), which will be equalized or exceeded on an average once in T years.

Based on Indian Standards Norms for design Flood and US Soil Conservation Service Norms the Kesem dam which is proposed to store about 500 Mm³ in its reservoir, has to be designed for PMF (WWDSE, 2007). Indian Standards Norms for design Flood and US Soil Conservation Service Norms are shown in table 2.1 and table 2.2 below.

Table 2-1 Indian standards norms for design flood

Type of structure	Temporary storage	Design storm precipitation for	
		Emergency spillway	Free board capacity
Type1, Low cost and low downstream hazard in the event of failure	25 year Frequency	0.25 PMF 6 hour Precipitation	0.375 PMF 6 hour Precipitation
Type 2, Moderate cost with no down stream hazard in the event of failure or low cost with moderate down stream hazard(serious loss of economy)	50 year Frequency	0.375 PMF 6 hours Precipitation	0.66 PMF 6 hour Precipitation
Type 3, high cost low or Moderate cost with high Downstream hazard in the event of failure (may involve loss of life)	100 year Frequency	0.5 PMF 6 hour Precipitation	1 PMF 6 hour Precipitation

Table 2-2. US Soil Conservation Service Norms

Classification	Gross storage (Mm ³)	Hydraulic Head	Inflow Design Flood
Small	Between 0.5 to 10	Between 7.5 and 12	100 year flood
Intermediate	Between 10 and 60	Between 12 and 30	SPF
Large	Greater than 60	Greater than 30	PMF

The essential requirement for derivation of the design PMF inflow hydrograph is a unit hydrograph (WWDSE, 2007). A unit hydrograph is a direct runoff hydrograph resulting from

one unit depth of rainfall excess occurring uniformly in time and space, within the “unit duration” (Chow, 1960).

The Kesem catchment is very peculiar in a way that to estimate the critical PMF hydrograph at the dam site the catchment area up to dam site was divided into up land area of 1462 km² and the balance low land area of 1673 km² (WWDSE, 2007). Synthetic Triangular unit hydrograph where derived for the upland catchment, low land catchment and the entire catchment. The following figures show the unit hydrograph for upland catchment, lowland catchment and the entire catchment.

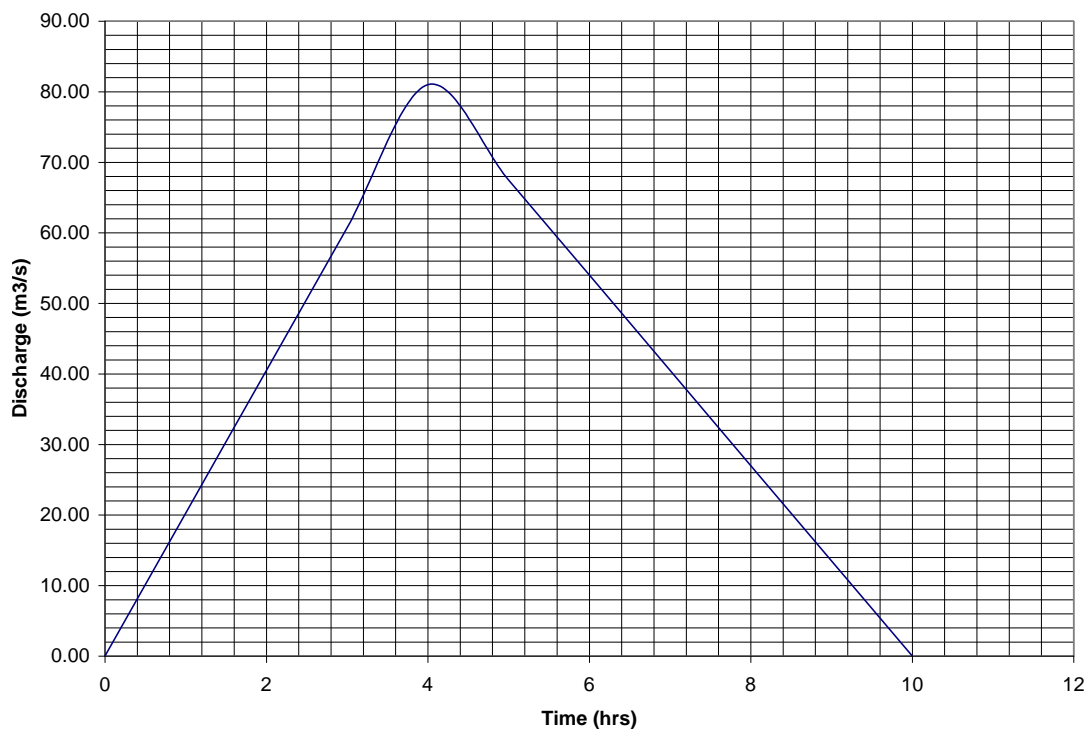


Figure 2-10 Synthetic Triangular Unit Hydrograph of Upland Catchment

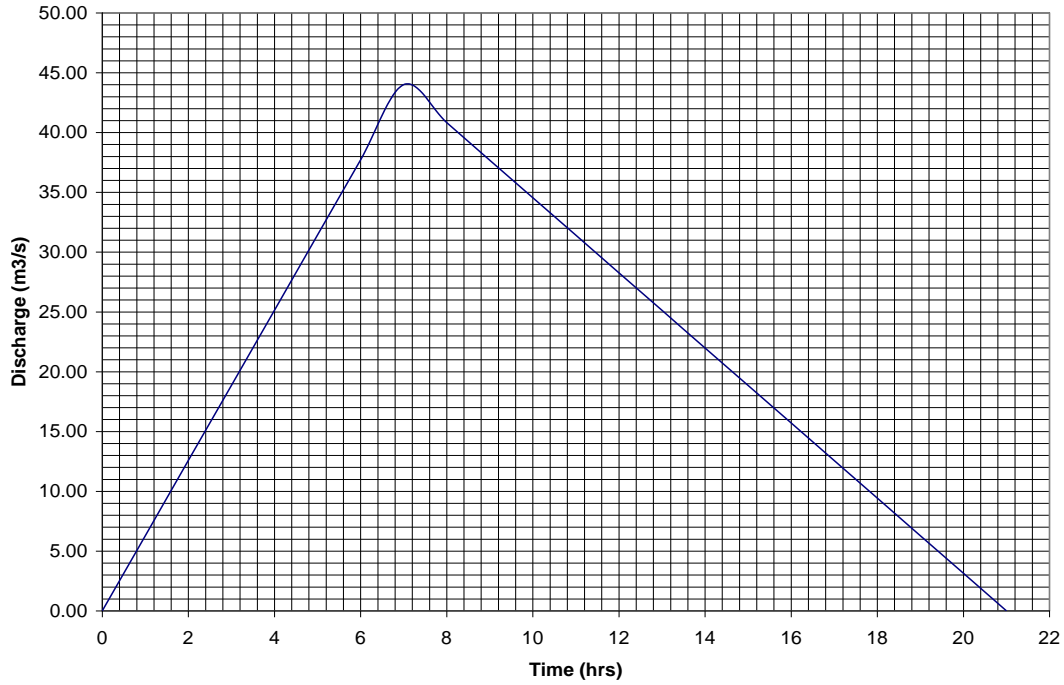


Figure 2-11 Synthetic Triangular Unit Hydrograph of Lowland Catchment

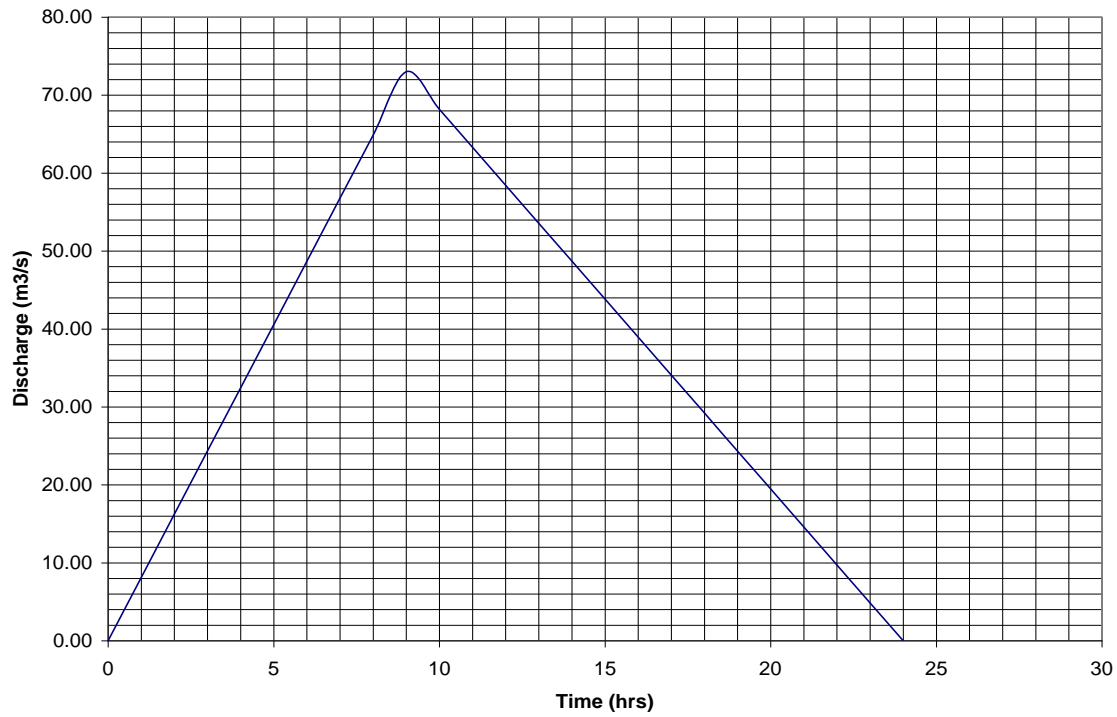


Figure 2-12 Synthetic Triangular Unit Hydrograph of Entire Catchment

PMF inflow Hydrograph is obtained by convoluting the unit hydrograph with appropriate Probable Maximum Precipitation ((Chow, 1960)).

To estimate the flood hydrograph of Kesem catchment upstream of the dam three alternative hydrographs were made. The first one develops the flood hydrographs for the up land catchment, the second for the lowland catchment and the third one for the entire catchment (WWDSE, 2007). The other two PMF inflow hydrographs represent the upland and lowland catchments separately. If these are judiciously combined, they can produce another PMF inflow hydrograph for the entire catchment. PMF hydrograph of the entire catchment and PMF hydrograph of upland and lowland catchment combined are both used for flood routing exercise since they are closer to actually observed shapes of floods from records (WWDSE, 2007). The following figures show PMF hydrographs of all the cases.

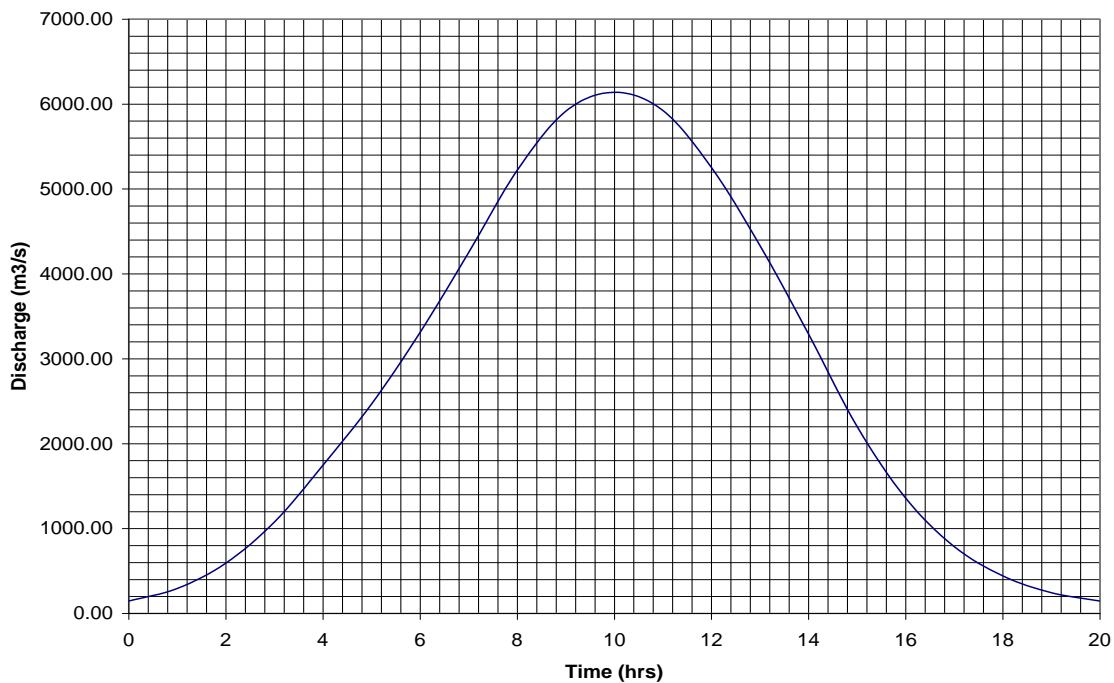


Figure 2-13 PMF Hydrograph of Upland Catchment by USBR Method

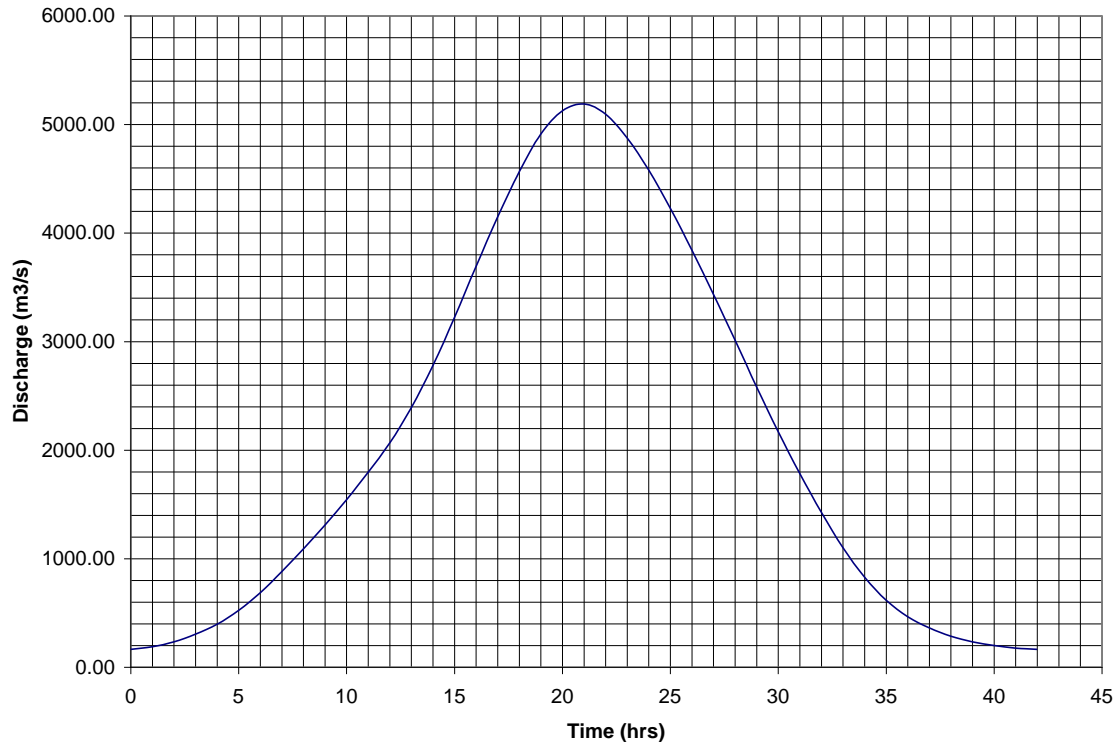


Figure 2-14 PMF Hydrograph of Lowland Catchment by USBR Method

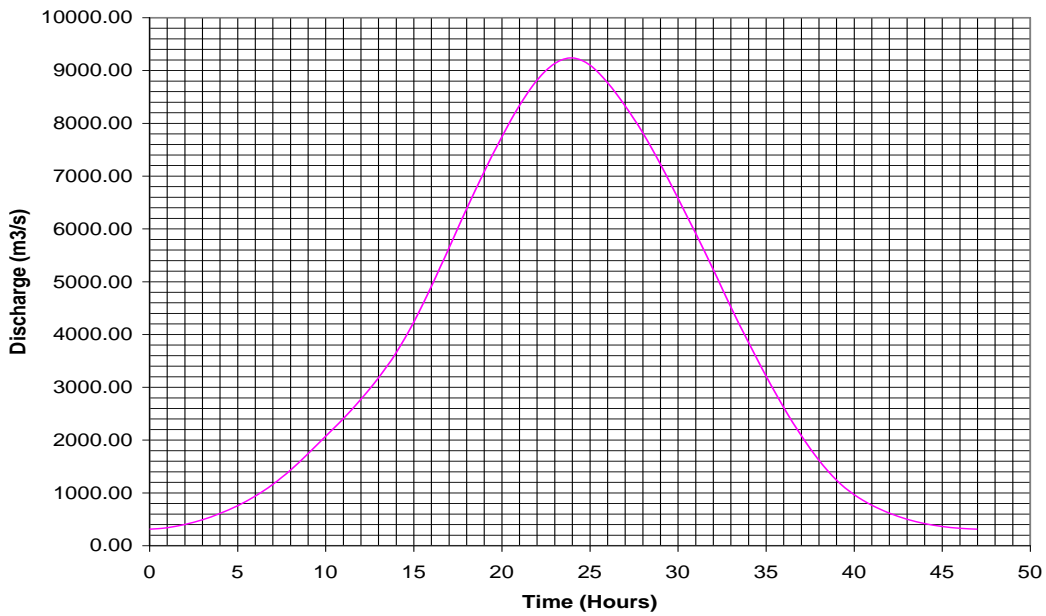


Figure 2-15 PMF Hydrograph of Entire Catchment by USBR Method

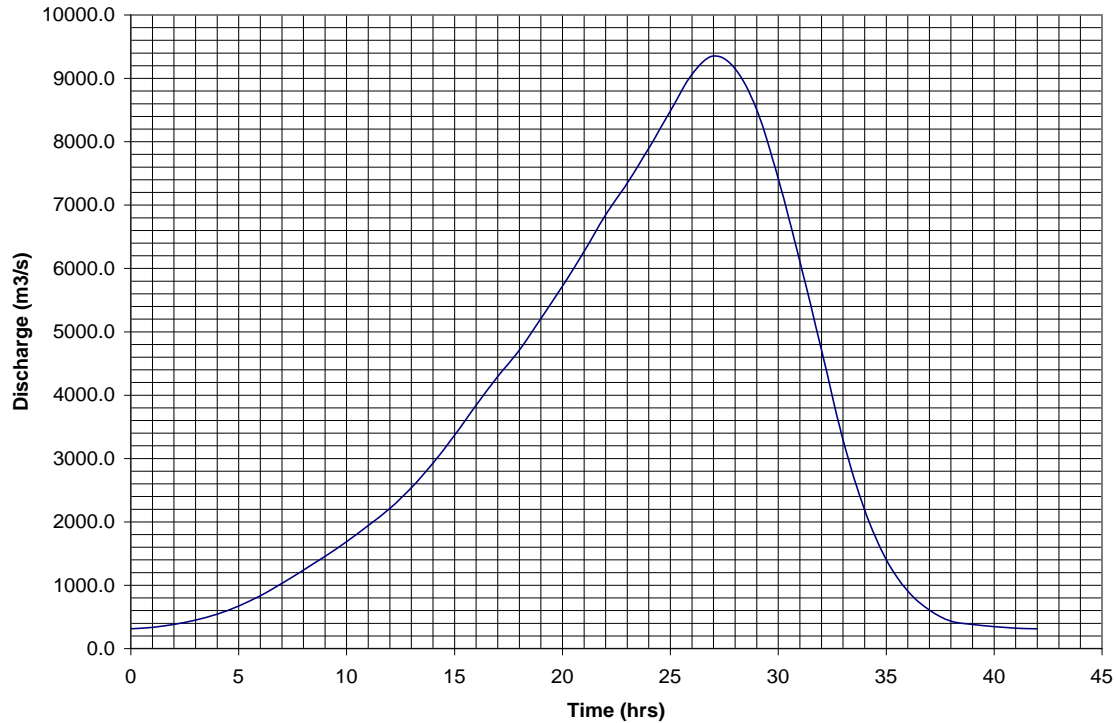


Figure 2-16 PMF Hydrograph of Entire Catchment by USBR Method (Upland and Lowland combined)

2.6. Previous work on Kesem dam

Dam breach analysis of Kesem dam for dam breach scenarios overtopping and piping has been done using the model BREACH to model the dam break and HEC-RAS to simulate the unsteady flow analysis.

PMF with a peak of 8888m³/s has been obtained and was used as an inflow in the BREACH model for overtopping failure mode. The spillway could accommodate the flood and breaching of embankment dam was not possible. Hence, piping failure mode was checked. Breach parameters and breach outflow hydrograph were performed by BREACH model.

Breach top width (m)	275.17m
Breach bottom width (m)	27.23m
Breach side slop	1.46
Breach formation time (hrs)	5.344hrs

Peak Breach outflow (m³/s) 92,037.88m³/s

This peak outflow through the breach is routed downstream and its propagation is analyzed in terms of area coverage and flood depth. HEC-RAS and HEC-GeoRAS are the models used for hydraulic computation and flood mapping, respectively.

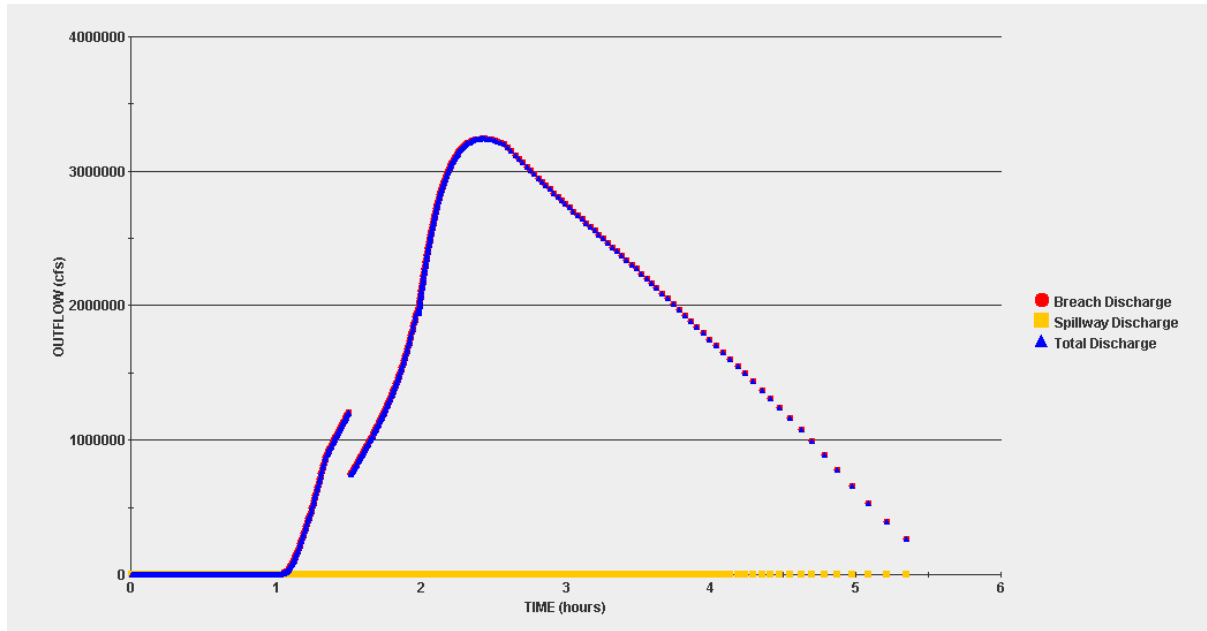


Figure 2-17 Outflow hydrograph of flow through the breach

3. METHODOLOGY

The methodology adopted in this study follows literature review, data collection, organization and analysis of data as per the requirement of the model in use. Figure 3-1 shows summary of the methodology.

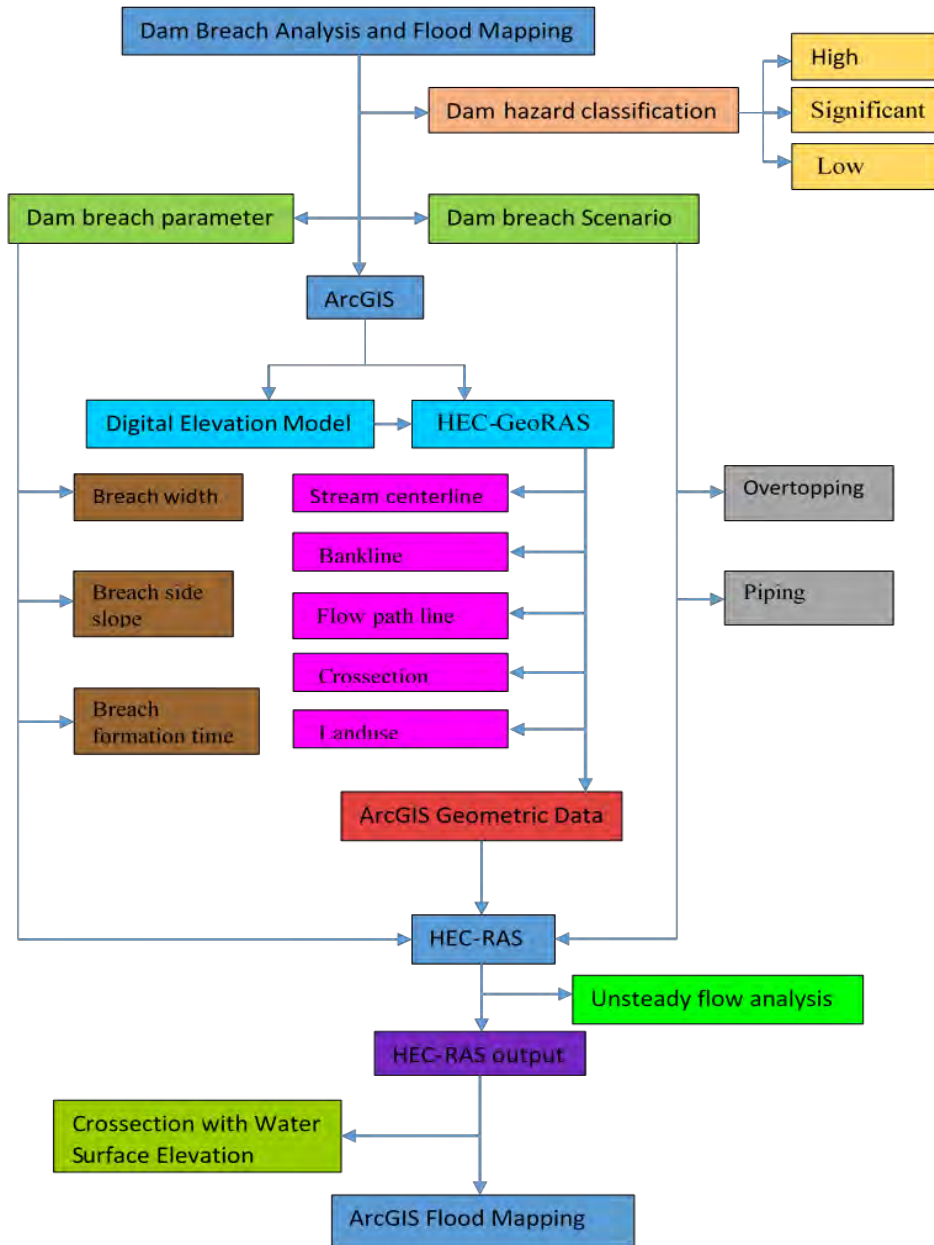


Figure 3-1 Summary of the methodology of Dam Breach Analysis

As the chart shows the first step in dam breach analysis is Dam hazard classification (High, Significant and low), the next step is to select dam failure Scenario (overtopping, piping, earthquake, land slide etc.). Before starting to work in the software's breach parameters are estimated for an input to HEC-RAS. In software analysis, first HEC-GeoRAS prepares ArcGIS geometric files using elevations from Digital Elevation Model for the use in HEC-RAS. Geometric files prepared using HEC-GeoRAS are imported into HEC-RAS and are used for unsteady flow analysis. After unsteady flow analysis the output from HEC-RAS which contains cross sections with water surface elevation attached to them is imported into ArcGIS and is used to prepare a flood map.

3.1. Location of the study area

The study area, Kesem Dam and Irrigation Project, is located at the southern end of the Afar depression (rift) in Afar regional state, 225km East of Addis Ababa and 40 Km NW of Metehara town.

The Kesem river catchment covers about 3,000 km² area and extends from an altitude of almost 3600 m down to 860 m a.m.s.l (above mean sea level). It rises on the high Ethiopian plateau and descends to the western scrap of the Great Rift Valley to join the Awash River at its middle section. Kesem dam is constructed on Kesem River and is geographically located at 9o8'45" Latitude and 39o53'31" Longitude.

It lies in between UTM 37 zone coordinates of 580000, 608000mE and 9810000-1020000mN covering 200 Km² of area in western part of Sabure sub-sheet (index name, 1:50,000 scale topographic map). The project involves construction of dam to impound half a billion cubic meters of water in order to irrigate 20,000 hectare of land for sugar cane plantation (WWDSE, 2007). Figure 3.2 shows location of the study area which includes Kesem dam and Kesem reservoir.

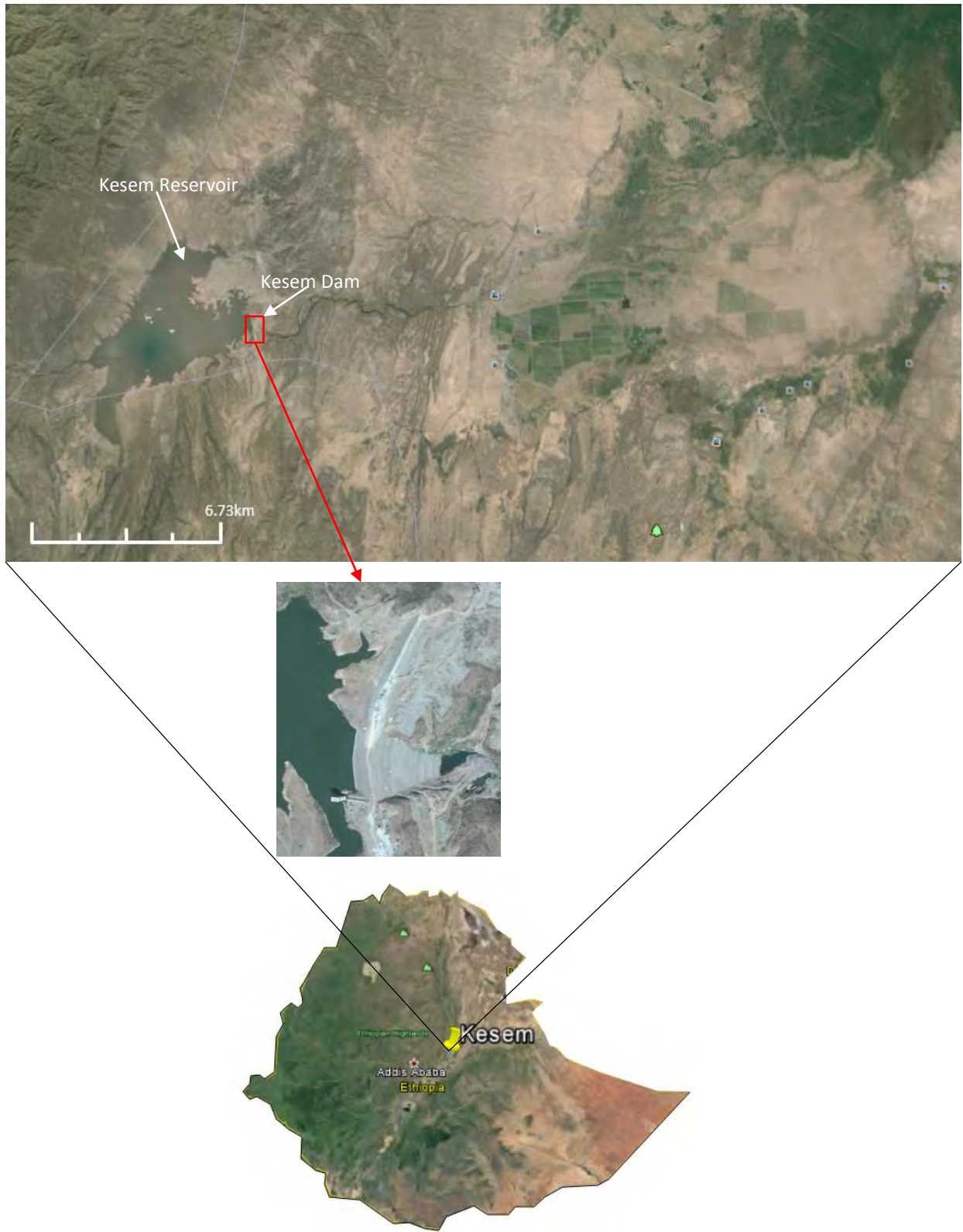


Figure 3-2 Location of the Study Area

Cross section along Kesem dam axis which is obtained from Digital Elevation Model of the study area is shown in figure-3.3.

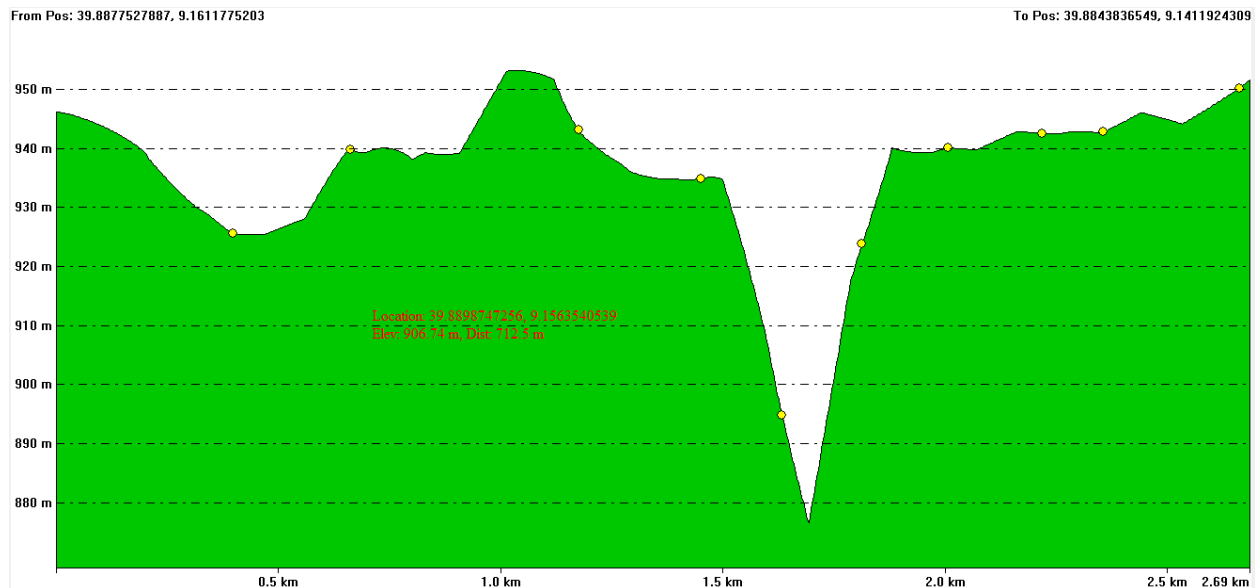


Figure 3-3 Cross Section Profile along Kesem Dam Axis

3.2. Data collection

The study is based on existing Meteorological, hydrologic and topographic data collected by different organizations. At this stage of the research the collection of data from different offices and field observation are done before desk work ((selection, organizing, literature review on dam break modeling and previous work).

Topographic data for awash basin which includes the Kesem catchment was collected from Ministry of Water, Irrigation and Electricity (MoWIE) in the form of Digital Elevation Model (DEM) which is a continuous representation of elevation values over a topographic surface.

Nowadays Digital elevation model (DEM) is being used for determining elevations at any point, slope and aspect and for finding features on the terrain, such as drainage basins and watershed, drainage networks and channels, peaks and pits and other landforms.

Hydrologic data of Kesem River including Probable Maximum Flood (PMF) inflow hydrograph, reservoir capacity, base flow and reservoir storage versus elevation curve are collected from Ministry of Water, Irrigation and Electricity (MoWIE) and from Water Works Design and supervision Enterprise.

3.3. Digital Elevation Model

Dam breach analysis involves routing the outflow hydrograph from the breached dam throughout downstream of the river from the dam up to the downstream boundary, this will require elevation data of the reservoir and elevation data of the cross section of the river including the flood plain. Digital Elevation Model (DEM) is used as a source of elevation data for this study.

Digital Elevation Model (DEM) consists of a sampled array of elevation for a number of ground positions at regular spaced intervals (USGC, 1993). Digital Elevation Model have various spatial resolutions, in this study 30m grid size of DEM is used. Digital Elevation model for the study area is shown in figure-3.4.

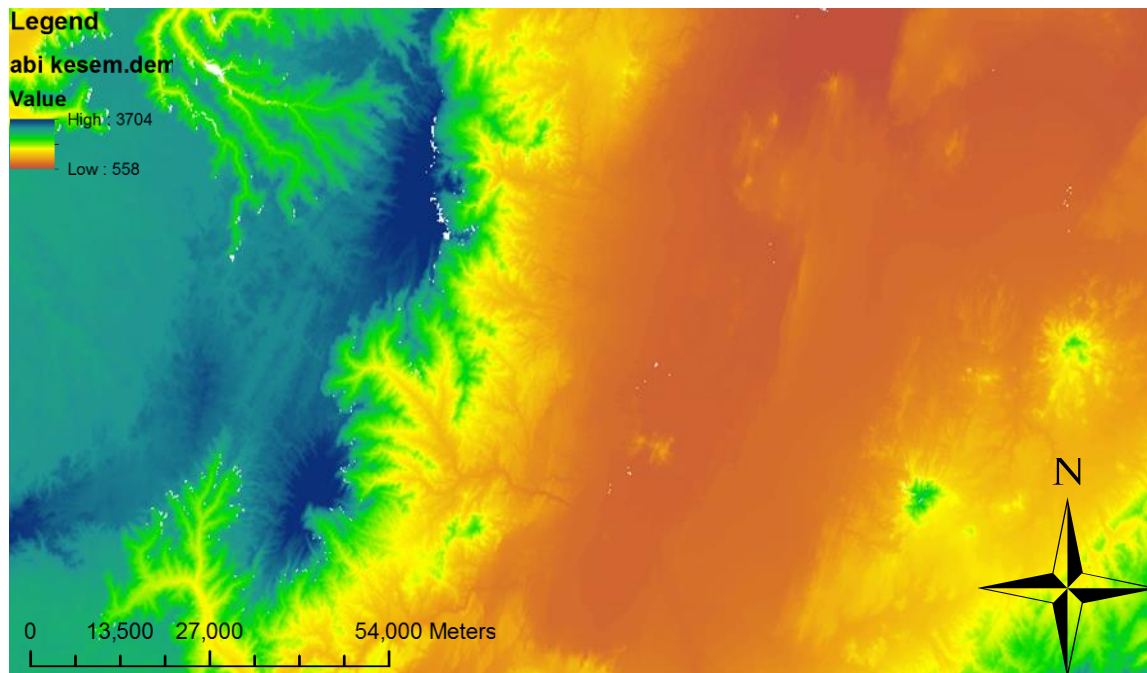


Figure 3-4 Digital Elevation Model of the Study Area

3.4. Dam breach parameter

The estimation of possible breach dimensions and development time is necessary in any assessment of dam safety since breach parameters will directly and substantially affect the estimate of the flow, inundated areas and warning time at the downstream locations (Gee, 2009).

The available breach parameter and peak breach flow estimation techniques can be classified into three categories, as follows: Comparative analysis, Regression-based methods based on

data collected from actual dam failures, and Physically-based simulation models (Chauhan, 2005).

Agency guidelines are generally in the form of suggested ranges (FERC 1987) or conservative upper bound estimates (USBR 1988). Therefore, they do not appear to be intended for obtaining accurate breach flow estimates. The physically-based embankment dam breach models, such as BREACH (Fread, 1988) and BEED (Singh and Scarlatos 1985) rely on bed-load type erosion formulas, which may be appropriate for some stages of the breach process, but are not consistent with the mechanics of much of the breaching process as observed in the field or laboratory (Wahl 1988). Therefore, in practice, most widely used methods for predicting breach parameters are based on regression analyses of data collected from dam failures (Chauhan, 2005).

In this study the MacDonald and Langridge-Monopolis (1984) and Froehlich (2008) which are empirical formulas developed from regression analysis of data collected from various dam failures, are used to estimate the Dam breach parameters of Kesem Dam.

Summary of MacDonald and Langridge-Monopolis (1984) and Froehlich (2008) empirical equations are shown in table-3.1.

Table 3-1. MacDonald and Langridge-Monopolis (1984) and Froehlich (2008) empirical equations

Breach Parameters	MacDonald and Langridge-Monopolis(1984)	Froehlich (2008)
Volume Eroded V_{er} (m^3)	$V_{er} = 0.0261(V_{out} \times h_w)^{0.769}$ (earth fill) $V_{er} = 0.00348(V_{out} \times h_w)^{0.852}$ (rock fill)	
Breach Width B (m)	$B_b = \frac{V_{er} - h_b^2(CZ_b + \frac{h_b Z_b Z_3}{3})}{h_b(C + \frac{h_b Z_3}{2})}$	$B_{avg} = 0.27 K_o V_w^{0.32} H_b^{0.04}$ $K_o = 1.0$ for piping $K_o = 1.3$ for overtopping
Breach Side Slope (H:V)	0.5:1	0.7:1 piping 1.0:1 overtopping

Breach Development Time T_f	$T_f = 0.0179V_{er}^{0.364} (hr)$	$T_f = 63.2 \sqrt{\frac{V_w}{gH_b^2}} (sec)$
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3.5. Hydraulic model development

Hydraulic model of Natural River could be successfully analyzed with four equations: continuity, energy, momentum, and manning. The manning equation is considered to be empirical and is used to estimate friction loss while the energy equation is considered semi-empirical (Saleh, 2009).

One-dimensional (1D) flow routing approaches such as HEC-RAS and some which are based on the saint venant equations or variations, still from the majority of traditional numerical hydraulic models used in practical engineering. The widespread usage in practice might be explained not only by the fact that 1D models are (in comparison to higher dimension hydraulic models) simpler to use and require a minimal amount of input data and computer power, but also because the basic concept and programs have already been around for several decades (US Army Corps of Engineering, 2001).

In this study, the hydraulic model used is HEC-RAS which is a public domain model developed by the US Army Crop of Engineers (USACE, 2002). It performs one dimensional steady and unsteady flow calculations on a network of natural or manmade open channel.

3.5.1. HEC-GeoRAS Modeling

HEC-GeoRAS is set of ArcGIS tools specifically designed to process geospatial data for use with the Hydrologic Engineering Center’s River Analysis System (HEC-RAS). The extension allows users to create an HEC-RAS import file containing geometric data from an existing digital elevation model (DEM) (Hydraulic Engineering Center, 2005).



Figure 3-5HEC-GeoRAS tool in ARCGIS

Essential data required to work with HEC-GeoRAS are terrain data (Digital Elevation Model DEM) and land use information. The geometry file for HEC-RAS contains information on cross-sections, hydraulic structures, river banks and other physical attributes of river channels. The pre-processing using HEC-GeoRAS involves creating these attributes in GIS, and then exporting them to the HEC-RAS geometry file. In HEC-GeoRAS, each attribute is stored in a separate feature class called as RAS Layer (Hydraulic Engineering Center, 2005). After creating RAS layers, these are added to the map document with a pre-assigned symbology. Since these layers are empty they are populated by digitizing each layer.

The Stream Centerline layer is used to identify the connectivity of the river system. It is created in the downstream direction and is used to assign river stations to the cross sections, bridges, and other structures to order computational nodes in the HEC-RAS model. The Cross-Sectional Cut Lines layer is the principal data constructed using HEC-GeoRAS. Cut lines are digitized across the floodplain area to capture the profile of the land surface. Cross sections should be digitized perpendicular to the path of flow in the channel and overbank areas to be consistent with one-dimensional flow characteristics. Having created the bank lines and flow path centerlines prior to laying out cut line locations is advantageous. A summary of RAS Layers and their use in building a hydraulic model is provided in table-3.2.

Table 3-2. Summary of HEC-GeoRAS layers and corresponding output for HEC-RAS

RAS layers	Description
Stream Centerline	Used to identify the connectivity of the river network and assign river stations to computation points.
Cross-Sectional Cut Lines	Used to extract elevation transects from the DEM at specified locations and other cross-sectional properties.
Bank Lines	Used in conjunction with the cut lines to identify the main channel from overbank areas.
Flow Path Centerlines	Used to identify the center of mass of flow in the main channel and overbanks to compute the downstream reach lengths between cross sections.
Land Use	Used to assign flow roughness factors (Manning's n values) to the cross sections.
Inline Structures	Used to extract the weir profile from the DEM for inline structures (dams).

Storage Areas	Used to define the extent of detention areas and develop the elevation- volume relationship from the DEM.
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Stream Centerline, Bank Lines and Flow Path layers that are digitized in ArcGIS using the HEC-GeoRAS tool are shown in figure-3.6.

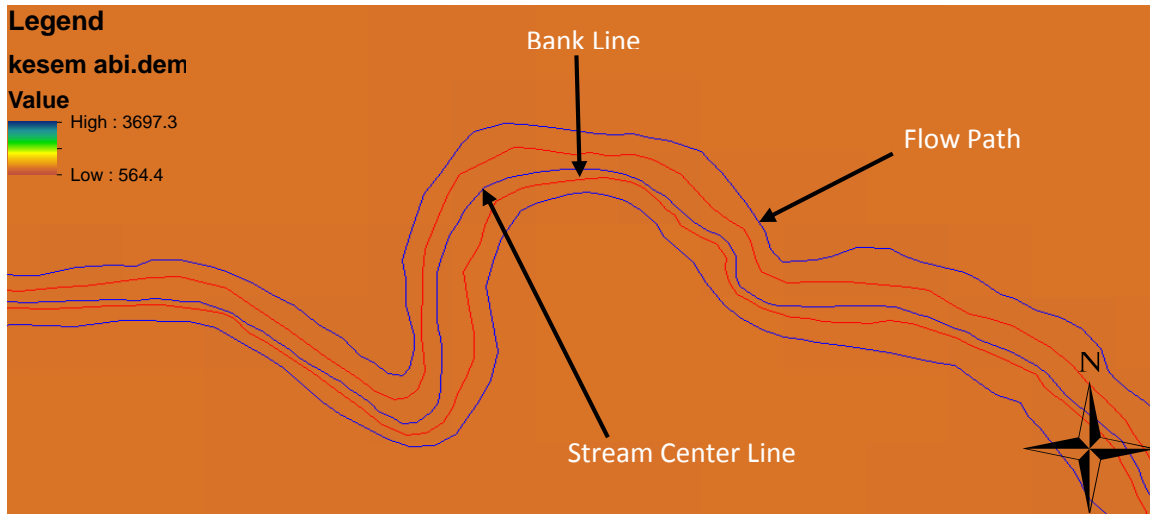


Figure 3-6HEC-GeoRAS layers on ARCGIS

The final task before exporting the GIS data to HEC-RAS geometry file is assigning Manning's n value to individual cross-sections. HEC-GeoRAS accomplishes this by using a land use feature class with Manning's n stored for different land use types. Figure-3.7 shows cross section cutline of Kesem River and their corresponding land use.

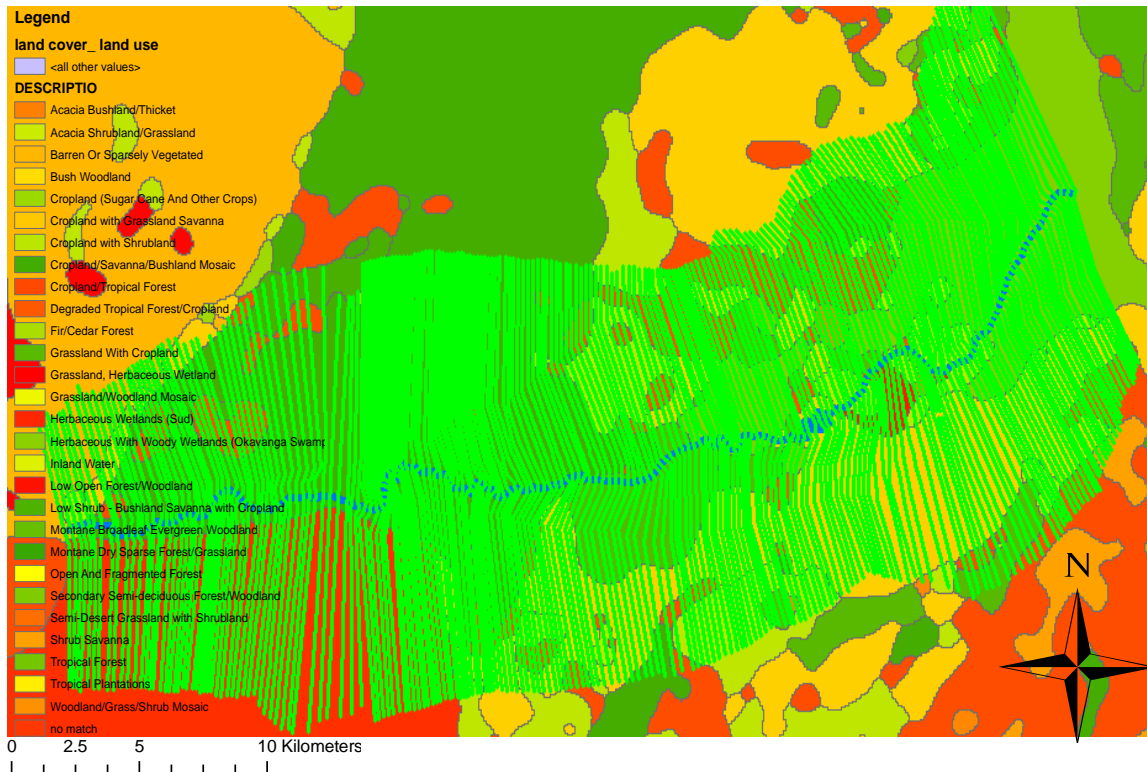


Figure 3-7 Cross section cut lines of Kesem River and land use layer of the study area

3.5.2. HEC-RAS Modeling

HEC-RAS is a one-dimensional river hydraulics model used for steady- flow and unsteady-flow water surface profile computations through a network of open channels (HEC, 2010). Because HEC-RAS solves the full Saint-Venant equations, it is well suited for computing the flood wave propagation resulting from a dam failure scenario (Cameron T., 2008).

3.5.2.1. Dam Profile

A dam is modeled in HEC-RAS as an inline structure. An inline structure is represented with a weir profile (that includes the spillway). An inline structure can be directly added to HEC-RAS or it can be imported from ARCGIS together with other geometric data. In this study the inline structure is imported from ARCGIS.

Inline structure data are entered in HEC-RAS. This data include a weir/Embankment profile, and any gated spillways that may be modeled (HEC, 2010). In this study only weir and

Embankment profile are entered since the spillway is not gated. Figure-3.8 shows profile of Kesem dam as an in line structure in HEC-RAS.

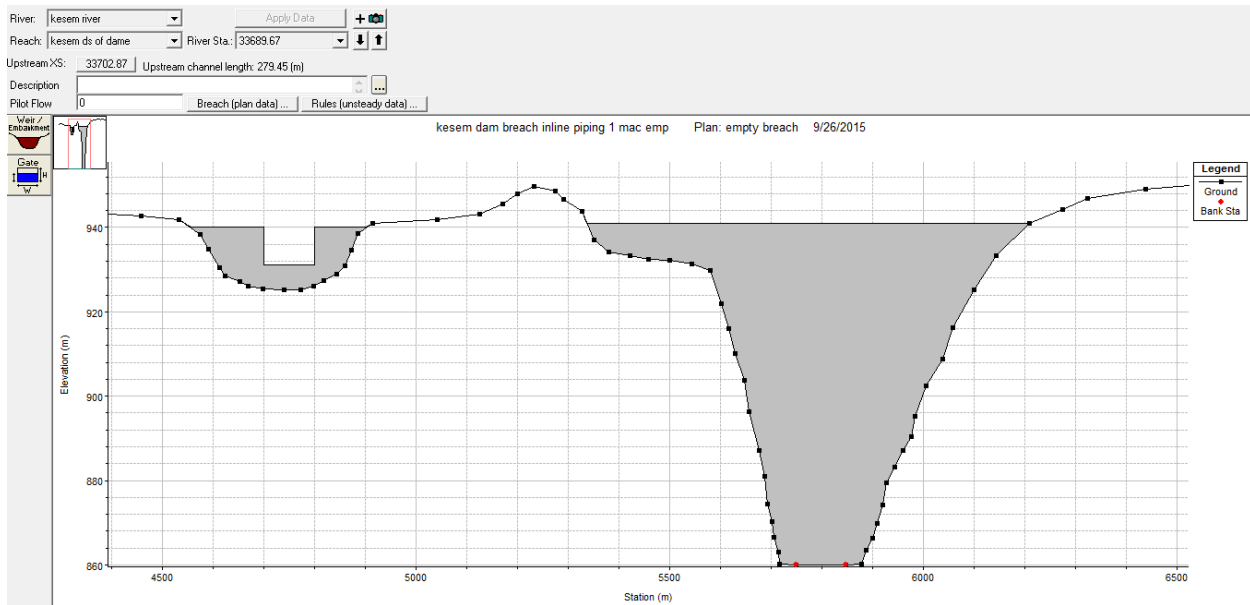


Figure 3-8 profile of Kesem dam in HEC-RAS

3.5.2.2. Dam Breach Data

To model dam failure in HEC-RAS Dam Breach parameters (breach shape and formation time) estimated using different empirical formulas and failure mode must be entered in HEC-RAS. Since HEC-RAS supports both over toping and piping failure mode breach parameters estimated for each failure mode is used for dam failure modeling in HEC-RAS. Data entry in HEC-RAS of breach information for Kesem dam is shown in Figure-3.9.

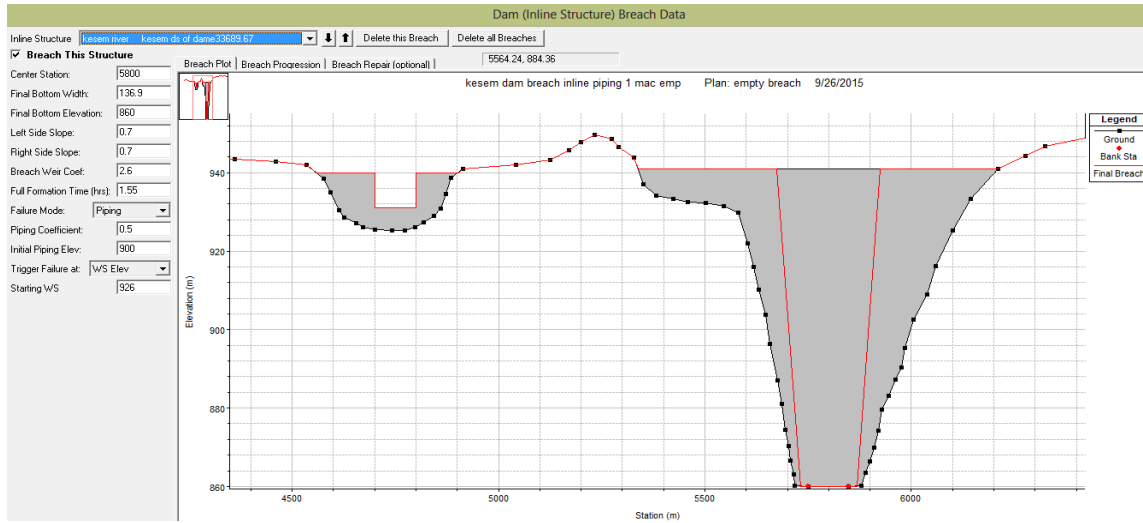


Figure 3-9 Dam breach data of Kesem dam in HEC-RAS

3.5.2.3. Channel Cross Section data

Cross sections are one of the key inputs to HEC-RAS. Cross sections are digitized in ArcGIS using the HEC-GeoRAS tool and are imported into HEC-RAS along with other geometric data. Cross section cutlines are used to extract elevation data from the terrain to create a ground profile across channel flow (Merwade, 2012). The intersection of cutlines with HEC-GeoRAS layers such as centerline and flow path lines are used to compute HEC-RAS attributes such as bank station (locations that separate channel from flood plain), downstream reach length (distance between cross sections) and Mannings (n) (Merwade, 2012). Cross sections provide useful information such as elevation across the flood plain, station points and Mannings roughness coefficient which are then used for Dam Breach analysis in HEC-RAS. In this study cross section cut lines are digitized every 200m along downstream of the river from the dam up to the downstream boundary 60km from the dam. Figure-3.10 show cross section cutline across Kesem River and other geometric data in HEC-RAS.

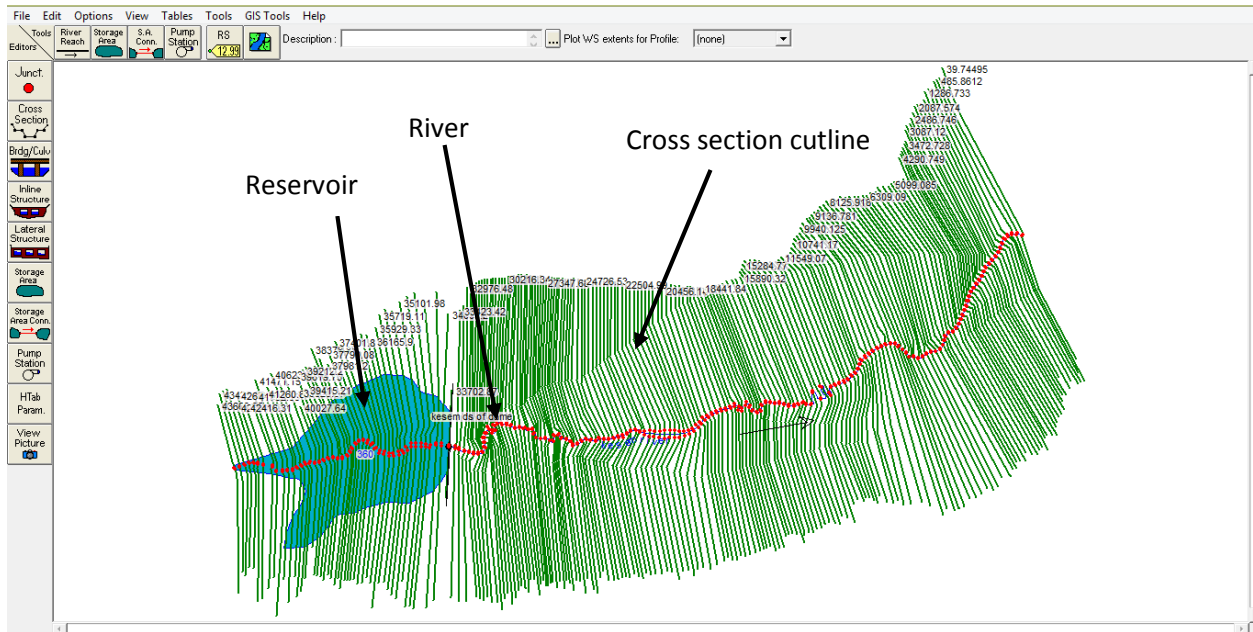


Figure 3-10 Geometric data of Kesem River

3.5.2.4. Unsteady flow analysis

Flood is a typical example of unsteady flow since the stage of the flow changes instantaneously as the flood wave pass by (Chow, 1960). In this study HEC-RAS is used to simulate unsteady flow throughout the downstream of Kesem River from Kesem dam up to the downstream boundary 60km from the dam. Once all of the geometric data are entered in to HEC-RAS, required unsteady flow data must be entered to undertake the unsteady flood simulation. Unsteady flow data includes boundary conditions at all of the external boundaries of the system, as well as any desired internal locations, and set the initial flow and storage area condition at the beginning of the simulation. Generally unsteady flow data required are boundary condition and initial condition.

There are different types of boundary conditions some of them are Flow Hydrograph, Stage Hydrograph, Stage and Flow hydrograph, Rating Curve, Normal Depth, Lateral Inflow hydrograph etc. Unsteady flow data used as a boundary condition in this study are PMF Inflow Hydrograph and Normal depth. The PMF Inflow Hydrograph is used as an upstream boundary

condition. Inflow Hydrograph boundary condition of Kesem River and its plot are shown on Figure-3.11.

Normal depth is used as a downstream boundary condition. Normal depth can only be used as a downstream boundary condition for an open ended reach. To use normal depth it is required to enter a friction slope for the reach in the vicinity of the boundary condition. The slope of the water surface is often a good estimate of the friction slope (HEC, 2010).

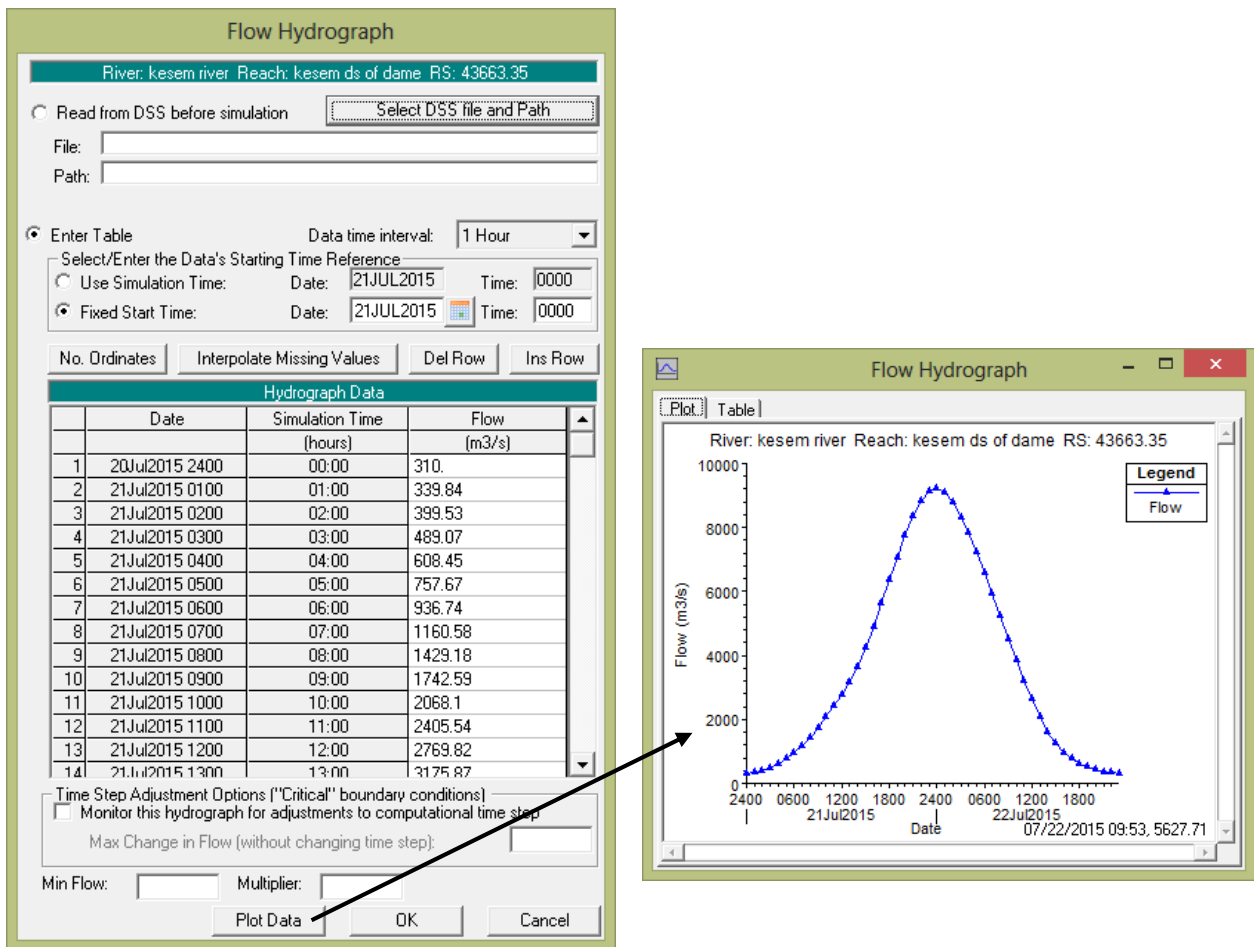


Figure 3-11PMF Inflow Hydrograph of Kesem River as an upstream boundary condition in HEC-RAS

In addition to the boundary condition, initial condition should be established at the beginning of the unsteady flow simulation. Initial condition consists of flow and stage information at each of the cross sections, as well as elevations for any storage areas defined in the system (HEC, 2010).

Once all the geometric and unsteady flow data have been entered, unsteady flow calculations can be performed. Unsteady flow analysis window where unsteady flow analysis can be computed in HEC-RAS is shown in Figure 3.12.

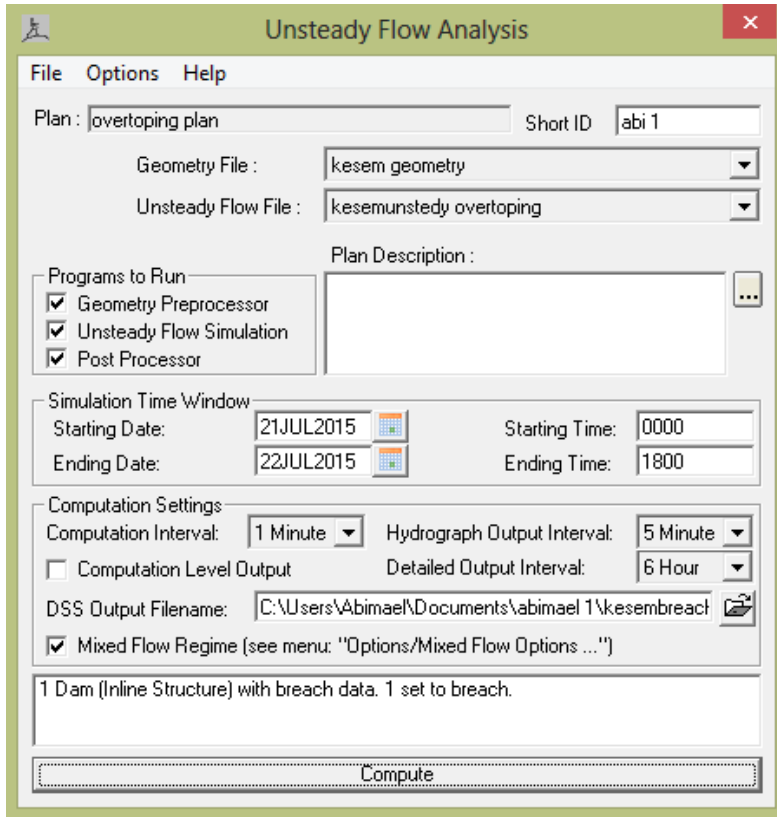


Figure 3-12 unsteady flow analysis window

3.6. Flood Plain Mapping

After unsteady flow analysis in HEC-RAS where water surface elevation at locations from upstream boundary to downstream boundary are obtained, the next step is to use this water surface elevations for flood mapping. Floodplain mapping is accomplished in the GIS using HEC-GeoRAS. HEC-GeoRAS is a set of ArcGIS tool which is specially designed to serve as a bridge between HEC-RAS and ArcGIS. It prepares geometric files on ArcGIS and also it export and import geometric files from and to HEC-RAS and ArcGIS.

To prepare the map in ArcGIS GIS information is exported from HEC-RAS and read into the GIS with GeoRAS. The geo-referenced cross sections are imported and water surface elevations attached to the cross sections are used to create a continuous water surface. The water surface is then compared with the terrain model and the floodplain is identified where the water surface is higher than the terrain. HEC-GeoRAS produces inundation maps for flood extent and depth.

4. RESULTS AND DISCUSSION

Based on the methodology and input requirement of the model selected, all the necessary steps are undertaken and the Dam Breach Analysis is simulated. So in this part all the necessary results will be shown and discussed towards the objective of the research.

4.1. Dam Breach Parameter

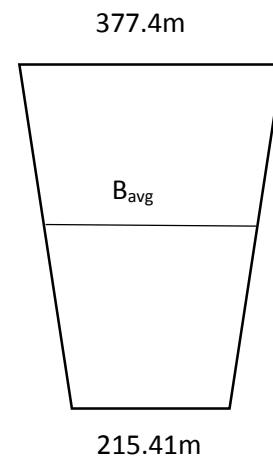
Estimating the dam breach parameters is one of the most important things that have to be done before dam breach analysis is simulated. Both MacDonald and Langridge-Monopolis (1984) and Froehlich (2008) are used to estimate breach parameters. The modes of failure for this dam are assumed to be overtopping and piping type of failure.

Breach parameters are estimated for both overtopping and piping and are used as an input for HEC-RAS. This dam breach parameters breach width, breach side slope and breach formation time are used as a geometric data during unsteady flow analysis. Results of dam breach parameter calculations for overtopping and piping failure mode for both MacDonald and Langridge-Monopolis (1984) and Froehlich (2008) methods are as follows.

Overtopping: - Froehlich (2008):

$$\text{Average breach width } B_{avg} = 0.27 K_o V_w^{0.32} H_b^{0.04}$$

$$B_{avg} = 215.14\text{m}$$



$$\text{Breach formation time } t_f = 63.2 \sqrt{\frac{V_w}{gH_b^2}} = 1.982 \text{ hrs}$$

Overtopping: - MacDonald and Langridge-Monopolis (1984)

Volume of material eroded from the dam embankment $V_{eroded} = 0.00348 (V_{out} * H_w)^{0.852}$

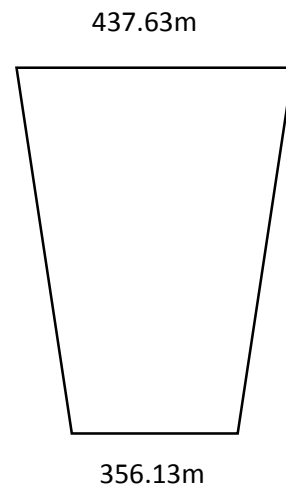
$$V_{eroded} = 3,664,595.25m^3$$

Bottom Width of the breach $W_b = \frac{V_{eroded} - H_b^2 (CZ_b + \frac{H_b Z_b Z_3}{3})}{H_b (C + \frac{H_b Z_3}{2})}$

$$W_b = 356.13m$$

Breach formation time $t_f = 0.0178 (V_{eroded})^{0.364}$

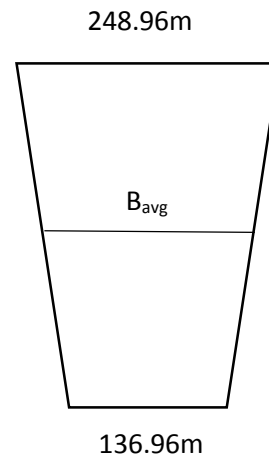
$$t_f = 4.36 \text{ hrs}$$



Piping: - Froehlich (2008):

Average breach width $B_{avg} = 0.27 K_o V_w^{0.32} H_b^{0.04}$

$$B_{avg} = 192.96m$$



Breach formation time $t_f = 63.2 \sqrt{\frac{V_w}{gH_b^2}} = 1.535 \text{ hrs}$

Piping: - MacDonald and Langridge-Monopolis (1984)

Volume of material eroded from the dam embankment $V_{eroded} = 0.00348 (V_{out} * H_w)^{0.852}$

$$V_{eroded} = 2,536,087.89m^3$$

Bottom Width of the breach $W_b = \frac{V_{eroded} - H_b^2 (CZ_b + \frac{H_b Z_b Z_3}{3})}{H_b (C + \frac{H_b Z_3}{2})}$

$W_b = 148.7 \text{ m}$

Breach formation time $t_f = 0.0178 (V_{eroded})^{0.364}$

$t_f = 3.81 \text{ hrs}$

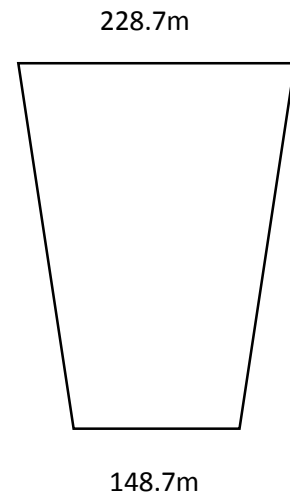


Table 4-1 summary of estimated breach parameters

Table Overtopping		
Dam breach parameters	Froehlich (2008)	Macdonald and Langridge-Monopolis (1984)
Breach bottom width	215.14m	356.13m
Breach side slope	1.0:1	0.5:1
Breach formation time	1.982hrs	4.36hrs
Piping		
Breach bottom width	136.95m	148.7m
Breach side slope	0.7:1	0.5:1
Breach formation time	1.535hrs	3.81 hrs

Based on the results from unsteady flow analysis, envelop curve and peak outflow regression equations, breach parameters from one of the method are selected.

4.2. Unsteady flow analysis

Unsteady flow analysis is the basic part of dam breach analysis where flood from the dam to the downstream boundary routed. With all the necessary data HEC-RAS can perform unsteady flow analysis. After entering boundary conditions for the farthest upstream and downstream cross sections and initial conditions in to HEC-RAS unsteady flow simulation can be initiated.

In unsteady flow analysis of this study PMF inflow hydrograph of Kesem River and normal depth of the farthest downstream vicinity are used as a boundary condition and initial flow and elevation for the storage area are used as an initial condition.

4.2.1. Unsteady flow analysis of overtopping

Unsteady flow simulation of overtopping failure in HEC-RAS requires PMF inflow hydrograph as an upstream boundary condition. Overtopping failure occurs when the flood due to the PMF inflow passes over the embankment.

Flood resulting from the PMF of Kesem River did not overtop the dam during unsteady flow simulation. The PMF rise the reservoir water surface elevation only to 939m which is 2m below the dam crest. Figure 4.1 shows the maximum water surface elevation on the dam profile during unsteady flow simulation.

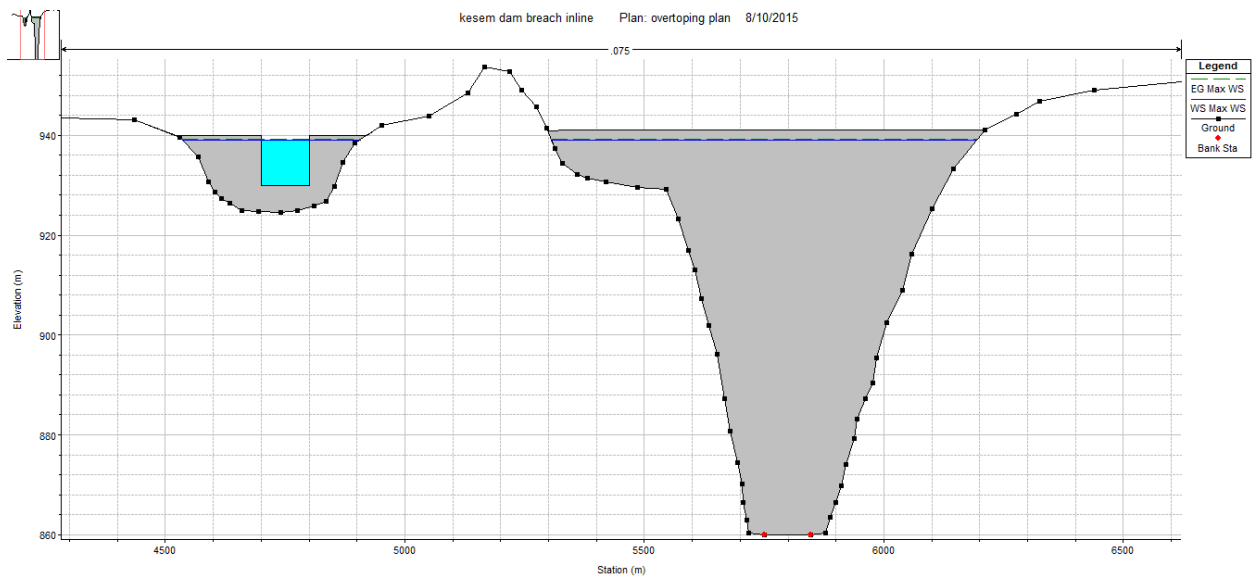


Figure 4-1 Kesem dam profile with maximum water surface

4.2.2. Unsteady flow analysis of piping

Unsteady flow analysis due to piping of Kesem dam in HEC-RAS is done after entering the necessary data for the simulation to begin. Dam breach parameters and boundary conditions are the necessary data that are inputted in HEC-RAS. For comparison two empirical formulas MacDonald and Langridge-Monopolis (1984) and Froehlich (2008) are used to estimate breach parameters. Unsteady flow analysis of piping in HEC-RAS is done for both methods.

The starting water surface elevation for piping is taken at the crest of the spillway, since the spill way is only used during flood events. Figure 4.2 shows water surface elevation before piping begins.

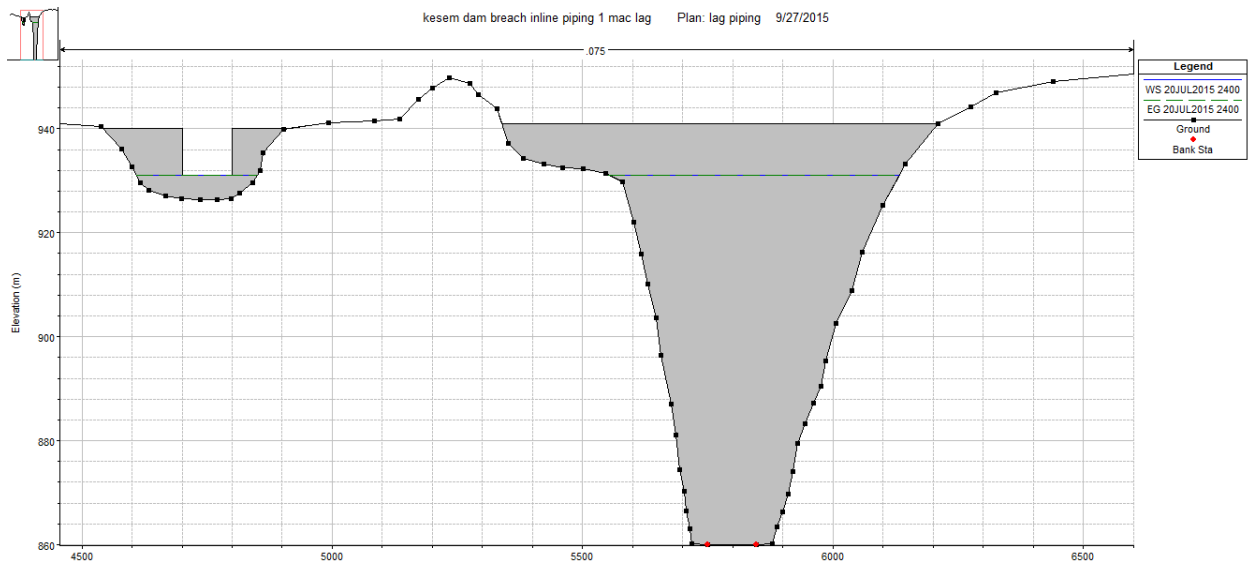


Figure 4-2 Kesem dam profile with water surface profile at the spillway crest level

Using breach parameters from Froehlich (2008) and MacDonald and Langridge-Monopolis (1984) for unsteady flow analysis in HEC-RAS out flow hydrograph from the breached dam and hydrograph at every cross section are obtained after the unsteady flow simulation. Figure 4.3 and Figure 4.4 shows hydrographs at the inline structure and at 20km, 40km and 60km from the dam for both Froehlich (2008) and MacDonald and Langridge-Monopolis (1984) respectively.

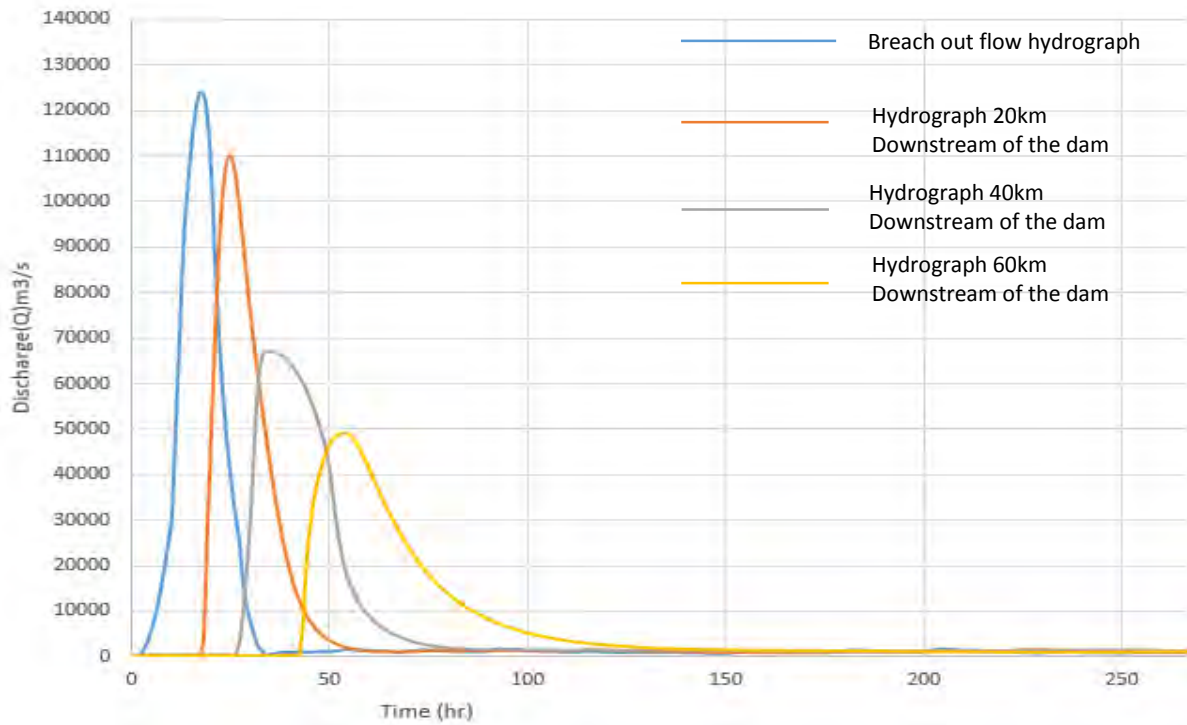


Figure 4-3 Hydrographs after unsteady flow analysis using Froehlich (2008)

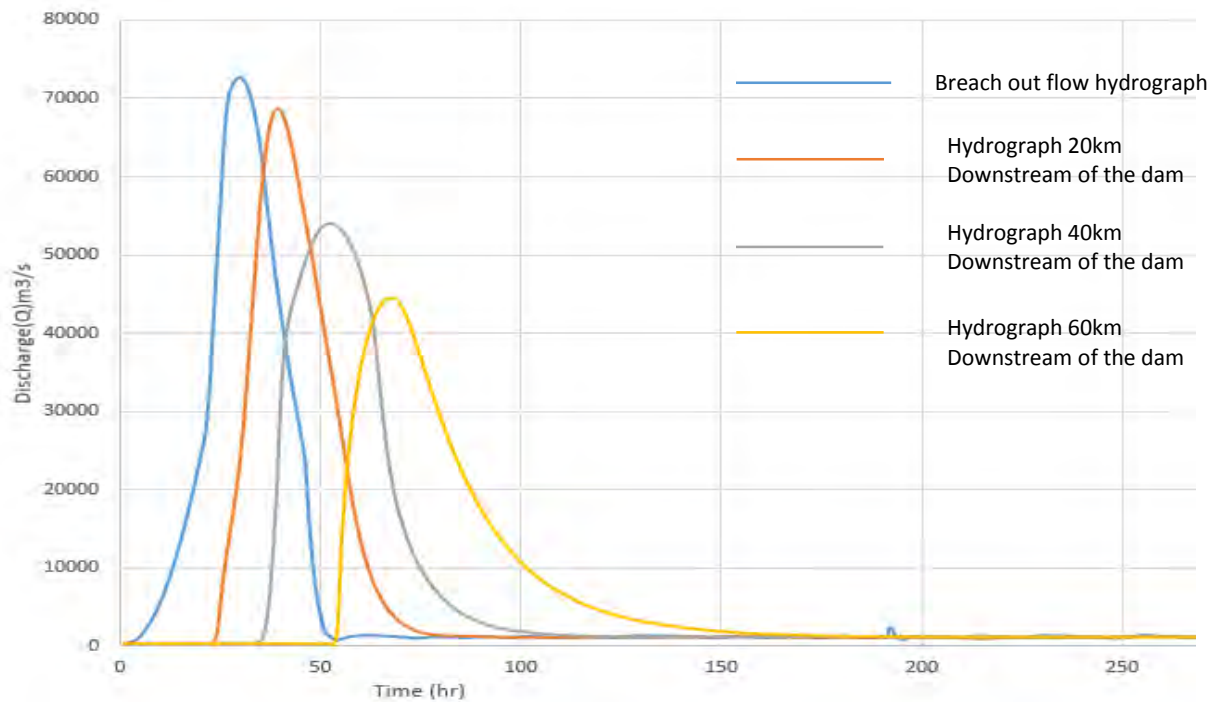


Figure 4-4 Hydrographs after unsteady flow analysis using MacDonald and Langridge-Monopolis (1984)

Both Froehlich (2008) and MacDonald and Langridge-Monopolis (1984) have their own importance, Froehlich (2008) differentiate between piping and overtopping and MacDonald and Langridge-Monopolis (1984) differentiate between earth fill dam and rock fill dam. Breach parameters from both equations are more or less similar but breach formation time which is one of the parameters is 1.535 hrs for Froehlich (2008) and 3.81hrs for MacDonald and Langridge-Monopolis (1984). This makes the magnitude of the peak outflow to diminish and the out flow flood to take longer time to pass through the breach in case of MacDonald and Langridge-Monopolis (1984). Figure 4.5 shows the difference between breach out flow hydrographs using Froehlich (2008) and MacDonald and Langridge-Monopolis (1984).

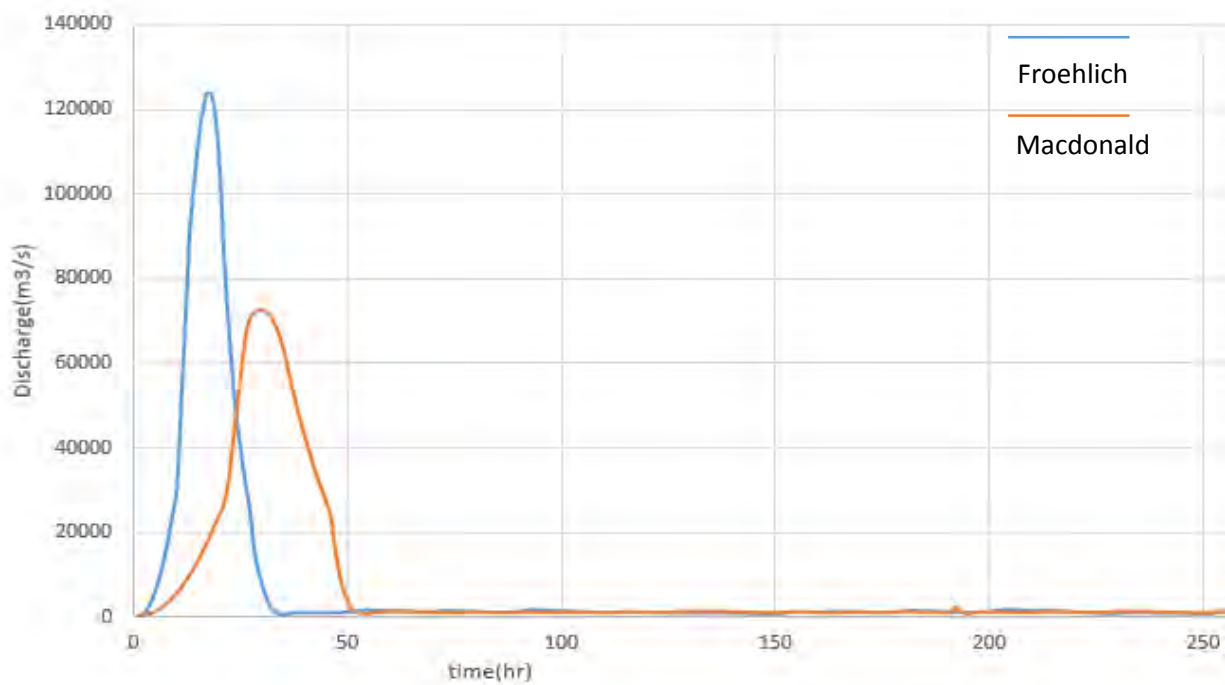


Figure 4-5 Breach out flow hydrograph of Froehlich (2008) and MacDonald and Langridge-Monopolis (1984)

After unsteady flow analysis due to piping is simulated in HEC-RAS, HEC-RAS can show the breach on the inline structure and water surface profile of Kesem River. Figure 4.6 and Figure 4.7 shows breach on Kesem dam and water surface profile of Kesem River.

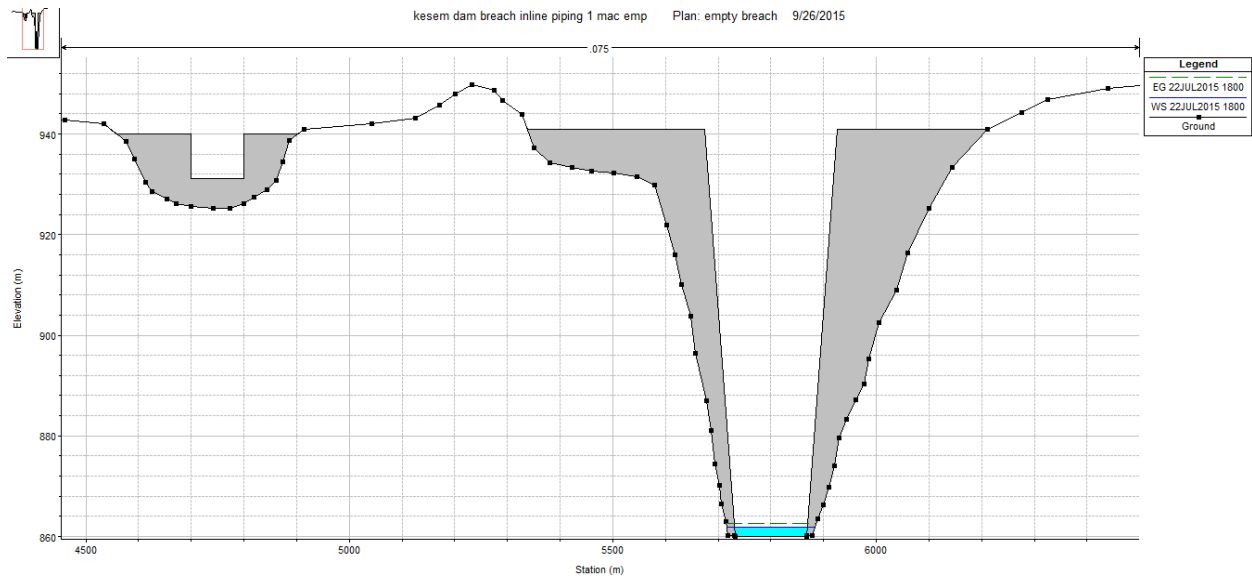


Figure 4-6 Kesem dam profile after the breach

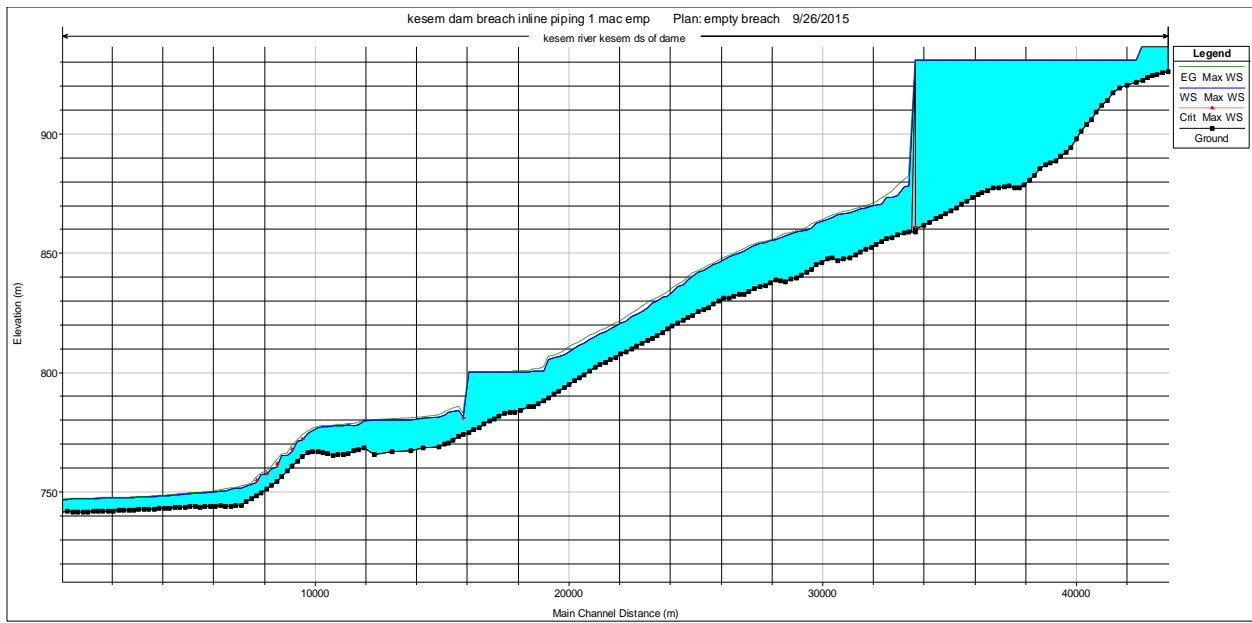


Figure 4-7 Water surface profile of Kesem river

4.3. Peak flow equations and Envelop curve

The computed peak outflow from the HEC-RAS model for both Froehlich (2008) ($Q_p = 123,685.8m^3/s$) and MacDonald and Langridge Monopolis (1984) ($Q_p = 72,670.8m^3/s$) are compared to Peak outflow regression equations as a test for reasonableness. Several researchers have developed equations from historical dam failure data. The equations developed are only used for comparison purpose.

Shown below is peak outflow equations and there resulting peak outflow for Kesem dam breach.

- USBR (1982): $Q = 19.1(h_w)^{1.85} = 49,483m^3$
 h_w -Depth of water above the breach invert at time of breach (70m)
- MacDonald and Langridge Monopolis (1984): $Q = 3.85(V_w h_w)^{0.411} = 81,618.04m^3$
 V_w -volume of water above the breach invert (480Mm³)
- Soil conservation service (SCS,1981): $Q = 16.6h_w^{1.85} = 43,007.03m^3$
- Hagen (1982): $Q = 0.54(S h_d)^{0.5} = 106,477.265m^3$
 S - Reservoir storage for water surface elevation at breach time (480M³)
 h_d – height of the dam (81m)
- Singh and Snorrason (1984): $Q = 13.4(h_d)^{1.89} = 54,217.794m^3$
- Costa (1985): $Q = 1.122(S)^{0.57} = 99,609.41m^3$

Peak outflow obtained from the HEC-RAS model using breach parameters from Macdonald and Langridge Monopolis (1984) is closer to peak outflow from regression equations, than peak outflow obtained from the model using breach parameters from Froehlich (2008). Peak outflow from the model using breach parameters from Macdonald and Langridge Monopolis (1984) have an average error of 0.3 when compared with results from peak outflow regression equations and peak outflow from the model using breach parameters from Froehlich (2008) have an average error of 0.4 when compared with results from peak outflow regression equations.

In addition to the peak flow equations, the model peak outflow can also be compared to envelop curve of historical failure. The flowing figure shows the envelop curve and where peak outflows from the model lay on the envelop curve using breach parameters from both Macdonald and Langridge Monopolis (1984) and Froehlich (2008).

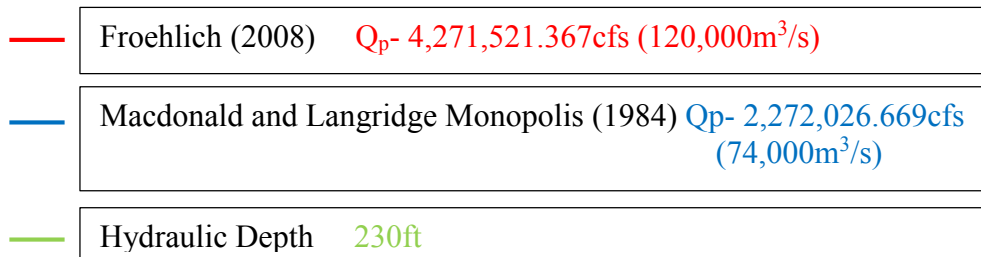
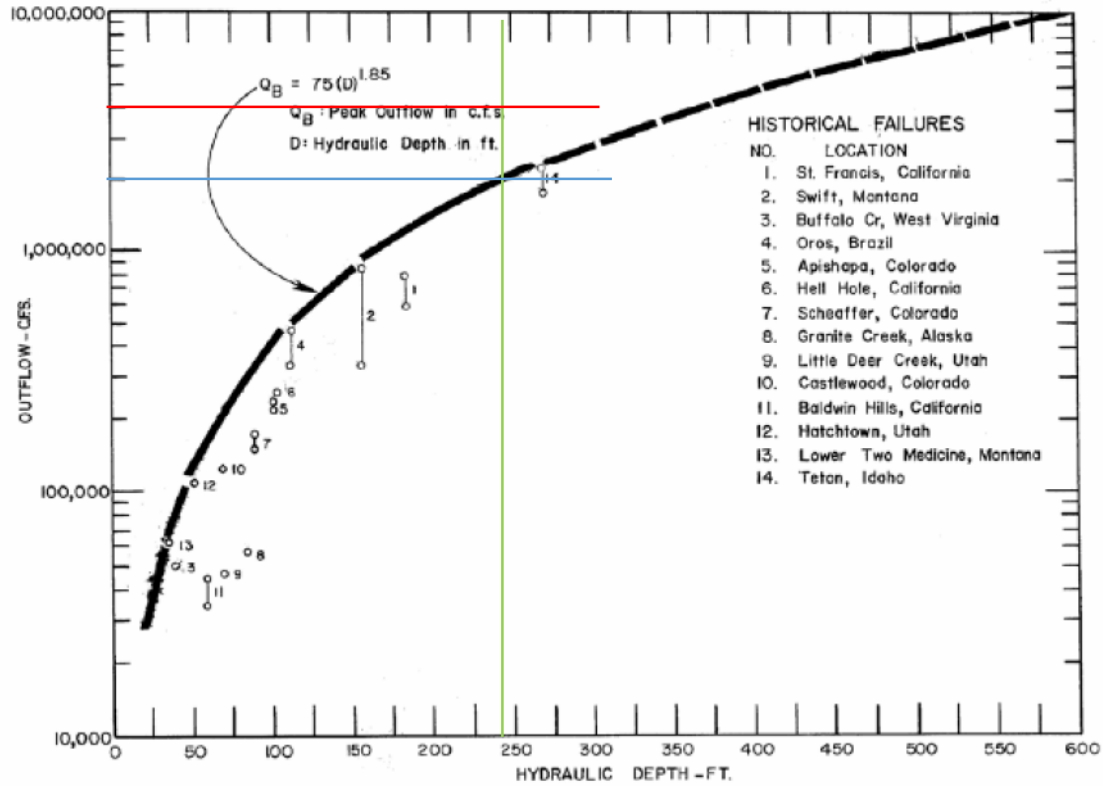


Figure 4-8 Peak outflow envelop curve of Historical dam failures

Peak outflow obtained from the HEC-RAS model using breach parameters from Macdonald and Langridge Monopolis (1984) lays in the envelop curve but peak outflow obtained from HEC-RAS using breach parameters from Froehlich (2008) lays outside of the curve. This shows that breach parameters obtained from Macdonald and Langridge Monopolis (1984) is more accurate than breach parameters obtained from Froehlich (2008).

4.4. Flood Mapping

Flood mapping is the final step in dam breach analysis. In this study the HEC-GeoRAS tool in GIS performs the flood mapping process in GIS. The flood map shows the maximum water surface and up to where this maximum water surface extends on the flood plain. The flood map is created on a type of DEM called TIN which is derived from a raster DEM. Once the flood map is created it can be shown on the TIN itself or on an aerial map of the study area. Since peak outflow from HEC-RAS using breach parameters from Macdonald and Langridge Monopolis (1984) is more accurate when compared with results of peak outflow regression equations and lays inside the envelop curve, water surface elevations obtained using breach parameters from Macdonald and Langridge Monopolis (1984) are used for the mapping process. Figure 4.8 shows map of the flood due to piping of Kesem Dam on a TIN and figure 4.10 shows map of flood due to piping of Kesem dam on an aerial map. Figure 4.9 shows the map of the study area.

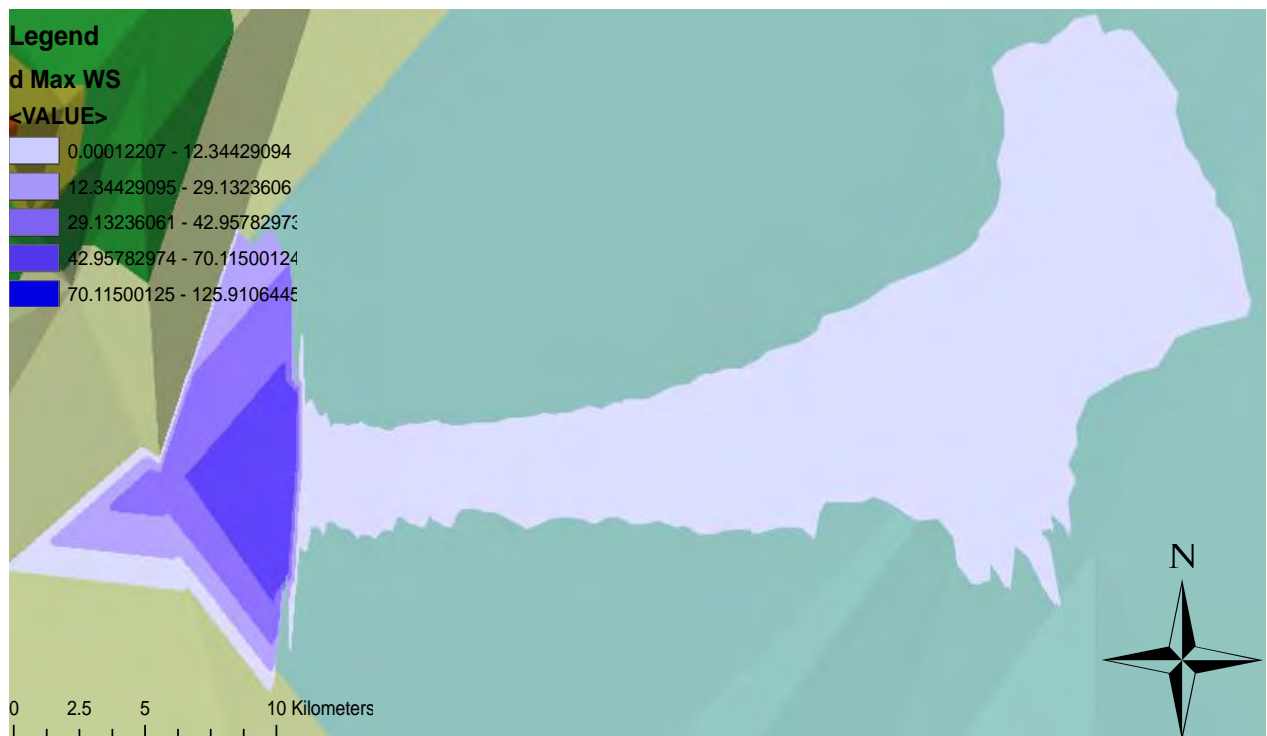


Figure 4-9 Flood map on a TIN derived from DEM

The flood map covers 183km² of land which is under water. The map shows different water surface elevations, differentiating them with color.



Figure 4-10 Aerial map of the study area

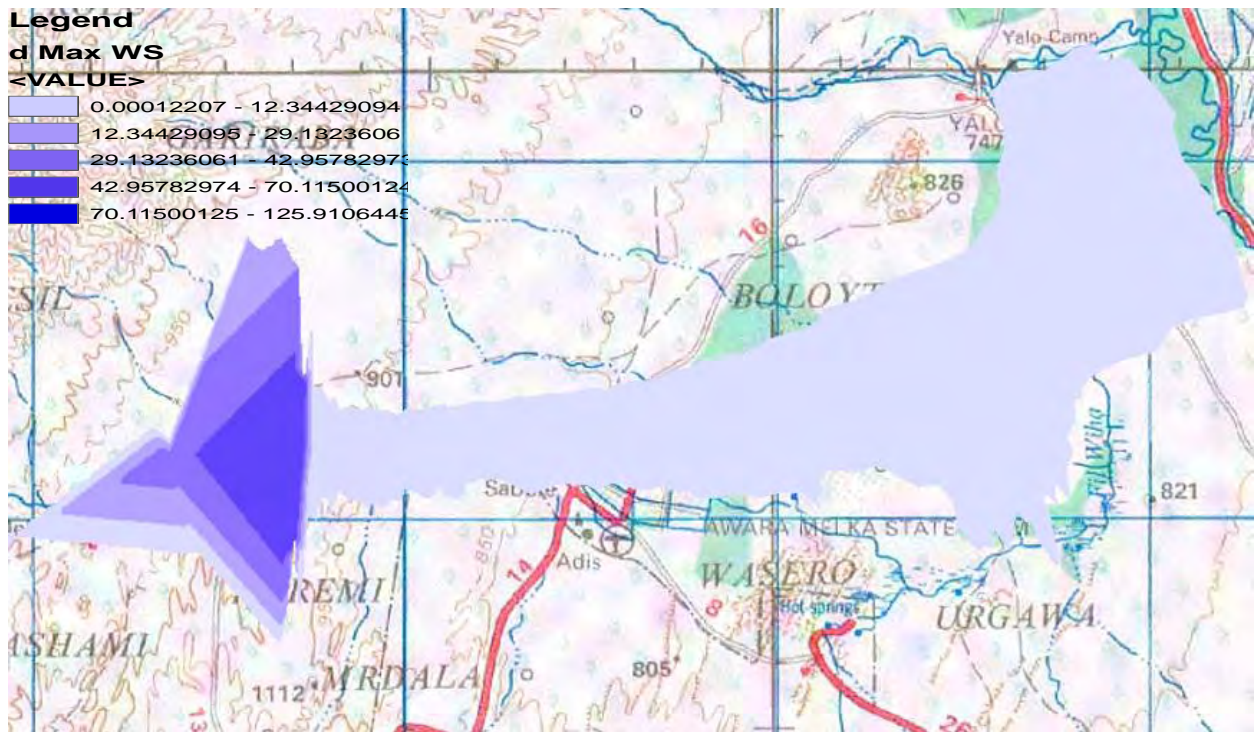


Figure 4-11 Flood map on an aerial map of the study area

As seen from the flood map that is overlain on areal map of the study area Sabure Town and Alibete village are affected by the flood.

The water surface profile can also be displayed on the XYZ perspective plot in HEC-RAS. Figure 4.11 shows the XYZ perspective plot for unsteady flow analysis of Kesem River.

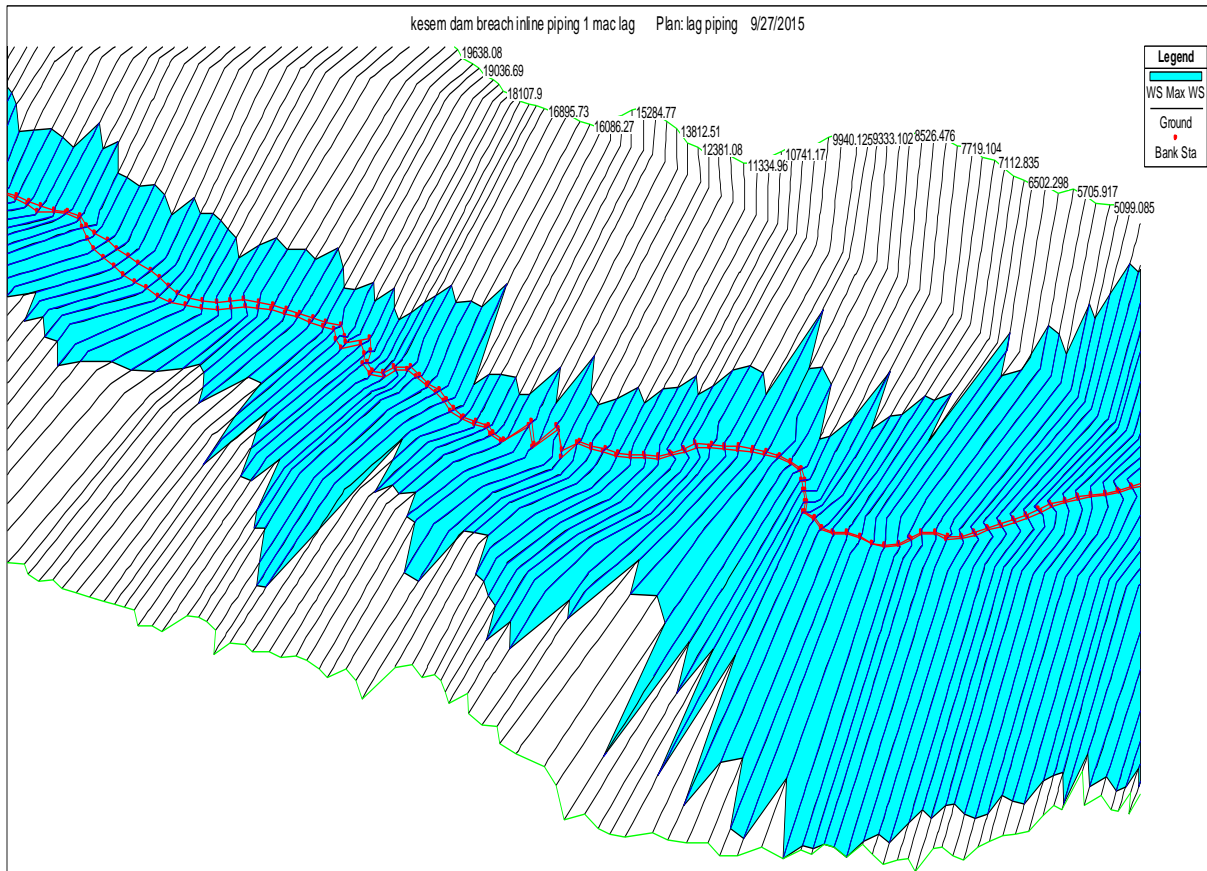


Figure 4-12 Perspective plot of water surface profile in HEC-RAS

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

For correct management of flood due to dam breach analysis and its consequence, Models and Empirical Formulas that are most widely used and can give more similar result when compared to historical failures. The river hydraulics model HEC-RAS have been used for computing flood wave propagation resulting from a dam failure scenario in different countries worldwide and is generally adequate for unsteady flow water surface computation.

Dam breach is modeled after selecting a failure scenario. Failure scenarios selected for this study are overtopping and piping, because most historical dam failures are due to those types of failure scenario. In this study Kesem dam can safely pass the PMF inflow of Kesem River without overtopping the embankment, this is because the dam have adequate spillway capacity and free board.

Empirical formulas are used to predict the breach parameters of Kesem dam. Froehlich (2008) and MacDonald and Langridge-Monopolis (1984) are used to estimate the breach parameter, where in turn the breach parameters are used for unsteady flow calculations in HEC-RAS. When peak outflow obtained with this empirical equations was compared to peak outflow empirical equations and peak out flow envelop curve from historical dam failure, MacDonald and Langridge-Monopolis (1984) was found to be more accurate as can be seen in the result. Peak outflow obtained using MacDonald and Langridge-Monopolis (1984) have an error of 0.3 when compared to peak outflow equations and lays inside the peak outflow envelop curve. Peak outflow obtained using Froehlich (2008) have an error of 0.4 when compared to peak outflow empirical equations from historical failure and lays outside the peak outflow envelop curve. Hence, Peak outflow using MacDonald and Langridge-Monopolis (1984) was chosen for unsteady flow analysis in HEC-RAS and Inundation Mapping in ArcGIS.

Previous dam breach analysis on Kesem dam used two models BREACH and HEC-RAS. BREACH was used to estimate the breach parameters and calculate the breach outflow hydrograph and HEC-RAS was used to perform the flood routing. Whereas, the present study calculates the breach parameters using empirical equations and performs both outflow hydrograph calculation and flood routing in HEC-RAS. When comparing the peak outflow and

breach parameters of the previous study and the present study it can be concluded that they are more or less similar other than the fact that they are performed with different methodology.

From the plotted flood map on ARCGIS TIN and aerial map, it can be seen that the flood affects 18300 hectares of area. The XYZ plot also show the extent of the water surface on the cross sections. The XYZ perspective plot in HEC-RAS and the flood map on ARCGIS are somewhat different, this is because HEC-RAS only sees elevation deferens on the cross sections not in between the cross sections but it is a different case for ARCGIS it can see elevation difference everywhere depending on the quality of DEM. From the map it can be concluded the flood from the dam breach covers Sabure town, Alibete village and irrigation farms.

5.2. Recommendation

The study benefited from the hydrological data obtained from different sources, mainly from the development of the area and availability of DEM of the study area. However, the work also faced limitations with regard to both data quantity and quality. To better understand and increase the knowledge on dam breach analysis in Ethiopia, row elevation data of the flood plain and there proper utilization is very crucial. New elevation data should be collected for a better information about the flood plain.

Although there are number of dams for hydropower, irrigation and water supply in Ethiopia, dam breach analysis have been given very little and/or no attention in the country. But it is very essential towards mitigating loss of life and property due to the flood from the dam breach. In the future more studies on dam breach analysis have to be done in this country.

The current study uses regression equations to predict the breach parameters, but physically based dam breach models give more detailed results even though they are more difficult. Future dam breach studies should consider to use physically based model to predict dam breach parameters.

When any unusual seeping of water is observed from the dam body any personnel who constantly observes the dam must inform the concerned body. The concerned body must try to save the dam before irreversible damage is done. If the actions taken to save the dam don't work, the authorities around the flood prone area have to evacuate peoples that are in danger because of the flood.

6. REFERENCES

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APPENDIX 1 PMF inflow hydrograph of the entire catchment

Direct surface runoff Hydrograph (m3/s)	Base Flow (m3/s)	PMF Hydrograph (m3/s)
0	310	310
29.8448	310	339.8448
89.5344	310	399.5344
179.0688	310	489.0688
298.448	310	608.448
447.672	310	757.672
626.7408	310	936.7408
850.5768	310	1160.5768
1119.18	310	1429.18
1432.5872	310	1742.5872
1758.1016	310	2068.1016
2095.5392	310	2405.5392
2459.8224	310	2769.8224
2865.8736	310	3175.8736
3343.5376	310	3653.5376
3922.6776	310	4232.6776
4594.4064	310	4904.4064
5328.7872	310	5638.7872
6066.1488	310	6376.1488
6767.7592	310	7077.7592
7433.5448	310	7743.5448
8024.792	310	8334.792
8502.7136	310	8812.7136
8819.6352	310	9129.6352
8927.772	310	9237.772
8791.1704	310	9101.1704
8457.284	310	8767.284
8021.5168	310	8331.5168
7508.0464	310	7818.0464
6916.9464	310	7226.9464
6272.008	310	6582.008
5606.0936	310	5916.0936
4919.2952	310	5229.2952
4211.576	310	4521.576
3539.516	310	3849.516
2903.2992	310	3213.2992
2311.8864	310	2621.8864
1774.2384	310	2084.2384
1308.2768	310	1618.2768
931.9232	310	1241.9232
654.1384	310	964.1384
457.0008	310	767.0008

304.6672	310	614.6672
188.1768	310	498.1768
107.5296	310	417.5296
53.7648	310	363.7648
17.9216	310	327.9216
0	310	310

APPENDIX 2 Elevation, area and capacity relationship of Kesem Reservoir

Elevation (m)	Original Area (km ²)	Original Capacity (MCM)
930	28.66	480
928	26.38	410
926	24.5	365
924	21.29	324
922	19	285
920	16.5	248
918	13.86	216
916	12.7	190
914	11.29	165
912	10.1	143
910	9.45	124.7
909	8.9	113.6
908	8.34	103
906	7.76	89
904	6.73	74.68
902	5.68	61.8
900	5.1	52
898	4.18	40.4
896	3.78	35.72
894	2.98	27.26
892	2.29	20.67
890	2	17.8
888	1.62	13.3
886	1.38	11.56
884	1.06	8.51
882	0.84	6.13
880	0.68	5.1
878	0.5	3.6
876	0.42	3.07
874	0.32	2.4
872	0.28	2
870	0.22	1.7
868	0.18	1.4
866	0.14	0.9
864	0.09	0.5
862	0.05	0.2
860	0	0

APPENDIX 3 Breach outflow hydrograph using Dam breach parameters from Macdonald and Langridge-Monopolis

River: kesem river Reach: kesem ds of dame RS:
33702.87

	Date	Stage INST-VAL METERS	Flow INST-VAL M3/S
1	20Jul2015 2400	931.11	310.38
2	21Jul2015 0005	931.1	374.29
3	21Jul2015 0010	931.1	578.74
4	21Jul2015 0015	930.96	869.59
5	21Jul2015 0020	930.94	1356.96
6	21Jul2015 0025	930.91	1976.62
7	21Jul2015 0030	930.8	2710.48
8	21Jul2015 0035	930.72	3596.91
9	21Jul2015 0040	930.63	4621.29
10	21Jul2015 0045	930.51	5775.41
11	21Jul2015 0050	930.37	7062.89
12	21Jul2015 0055	930.22	8489.24
13	21Jul2015 0100	930.05	10052.16
14	21Jul2015 0105	929.84	11761.06
15	21Jul2015 0110	929.62	13597.78
16	21Jul2015 0115	929.38	15555.28
17	21Jul2015 0120	929.11	17626.51
18	21Jul2015 0125	928.79	19799.97
19	21Jul2015 0130	928.43	22069.26
20	21Jul2015 0135	928.03	24426.03
21	21Jul2015 0140	927.56	26855.79
22	21Jul2015 0145	926.94	31987.12
23	21Jul2015 0150	926.15	40679.11
24	21Jul2015 0155	925.18	49383.71
25	21Jul2015 0200	923.94	58048.04
26	21Jul2015 0205	922.39	66360.09
27	21Jul2015 0210	920.77	70518.41
28	21Jul2015 0215	919	71827.07
29	21Jul2015 0220	917.05	72507.2
30	21Jul2015 0225	914.98	72670.8
31	21Jul2015 0230	912.69	72060.66
32	21Jul2015 0235	910.31	70956.92
33	21Jul2015 0240	907.7	69005.88
34	21Jul2015 0245	904.94	66326.91

35	21Jul2015	0250	902.12	63007.54
36	21Jul2015	0255	898.97	58404.44
37	21Jul2015	0300	895.99	54035
38	21Jul2015	0305	893.06	49642.19
39	21Jul2015	0310	890.35	45789.22
40	21Jul2015	0315	887.76	42212.87
41	21Jul2015	0320	885.31	38952.82
42	21Jul2015	0325	882.79	35372.93
43	21Jul2015	0330	880.47	32440.87
44	21Jul2015	0335	878.17	29578.76
45	21Jul2015	0340	875.86	26768.61
46	21Jul2015	0345	873.4	23476.48
47	21Jul2015	0350	869.89	15720.35
48	21Jul2015	0355	867.88	10014.62
49	21Jul2015	0400	866.07	6045.1
50	21Jul2015	0405	864.64	3535.43
51	21Jul2015	0410	863.21	1721.92
52	21Jul2015	0415	862.64	1327.31
53	21Jul2015	0420	862.09	898.59
54	21Jul2015	0425	861.67	694.05
55	21Jul2015	0430	861.89	886.06
56	21Jul2015	0435	862.09	1047.58
57	21Jul2015	0440	862.3	1205.06
58	21Jul2015	0445	862.39	1251.99
59	21Jul2015	0450	862.47	1306.7
60	21Jul2015	0455	862.53	1359.26
61	21Jul2015	0500	862.52	1356.58
62	21Jul2015	0505	862.51	1332.41
63	21Jul2015	0510	862.53	1352.1
64	21Jul2015	0515	862.52	1347.25
65	21Jul2015	0520	862.48	1308.87
66	21Jul2015	0525	862.46	1287.09
67	21Jul2015	0530	862.43	1261.57
68	21Jul2015	0535	862.38	1220.29
69	21Jul2015	0540	862.34	1184.29
70	21Jul2015	0545	862.3	1153.84
71	21Jul2015	0550	862.24	1105.64
72	21Jul2015	0555	862.2	1070.1
73	21Jul2015	0600	862.15	1031.57
74	21Jul2015	0605	862.13	1028.98
75	21Jul2015	0610	862.14	1043.06
76	21Jul2015	0615	862.18	1070.46
77	21Jul2015	0620	862.23	1109.82

78	21Jul2015	0625	862.28	1152.61
79	21Jul2015	0630	862.33	1200.05
80	21Jul2015	0635	862.32	1169.85
81	21Jul2015	0640	862.33	1180.37
82	21Jul2015	0645	862.28	1145.93
83	21Jul2015	0650	862.24	1107.26
84	21Jul2015	0655	862.21	1083.91
85	21Jul2015	0700	862.17	1056.29
86	21Jul2015	0705	862.16	1048.39
87	21Jul2015	0710	862.18	1063.81
88	21Jul2015	0715	862.21	1093.6
89	21Jul2015	0720	862.26	1135.72
90	21Jul2015	0725	862.32	1184.81
91	21Jul2015	0730	862.38	1237.38
92	21Jul2015	0735	862.33	1186.78
93	21Jul2015	0740	862.31	1159.45
94	21Jul2015	0745	862.28	1138.83
95	21Jul2015	0750	862.21	1081.49
96	21Jul2015	0755	862.19	1065.51
97	21Jul2015	0800	862.22	1098.28
98	21Jul2015	0805	862.24	1125.08
99	21Jul2015	0810	862.27	1154.2
100	21Jul2015	0815	862.32	1189.79
101	21Jul2015	0820	862.36	1203.24
102	21Jul2015	0825	862.39	1239.8
103	21Jul2015	0830	862.34	1189.02
104	21Jul2015	0835	862.34	1185.65
105	21Jul2015	0840	862.32	1180.21
106	21Jul2015	0845	862.26	1115.59
107	21Jul2015	0850	862.25	1116.69
108	21Jul2015	0855	862.2	1077.62
109	21Jul2015	0900	862.16	1045.45
110	21Jul2015	0905	862.16	1047.28
111	21Jul2015	0910	862.17	1062.06
112	21Jul2015	0915	862.2	1091.1
113	21Jul2015	0920	862.26	1135.14
114	21Jul2015	0925	862.33	1193.92
115	21Jul2015	0930	862.41	1262.85
116	21Jul2015	0935	862.4	1239.02
117	21Jul2015	0940	862.39	1234.2
118	21Jul2015	0945	862.38	1228.04
119	21Jul2015	0950	862.33	1179.25
120	21Jul2015	0955	862.32	1168.09

121	21Jul2015	1000	862.28	1142.63
122	21Jul2015	1005	862.23	1090.23
123	21Jul2015	1010	862.2	1074.96
124	21Jul2015	1015	862.21	1087.06
125	21Jul2015	1020	862.25	1124.84
126	21Jul2015	1025	862.31	1179.77
127	21Jul2015	1030	862.39	1257.12
128	21Jul2015	1035	862.5	1343.31
129	21Jul2015	1040	862.49	1329.45
130	21Jul2015	1045	862.48	1303.25
131	21Jul2015	1050	862.51	1332.42
132	21Jul2015	1055	862.5	1334.69
133	21Jul2015	1100	862.48	1303.78
134	21Jul2015	1105	862.48	1308.06
135	21Jul2015	1110	862.48	1314.37
136	21Jul2015	1115	862.46	1292.16
137	21Jul2015	1120	862.45	1280.61
138	21Jul2015	1125	862.45	1283.41
139	21Jul2015	1130	862.43	1264.07
140	21Jul2015	1135	862.41	1244.88
141	21Jul2015	1140	862.4	1240.62
142	21Jul2015	1145	862.37	1214.39
143	21Jul2015	1150	862.34	1188.82
144	21Jul2015	1155	862.31	1169.88
145	21Jul2015	1200	862.27	1129.77
146	21Jul2015	1205	862.23	1100.87
147	21Jul2015	1210	862.19	1064.1
148	21Jul2015	1215	862.14	1029.75
149	21Jul2015	1220	862.13	1027.37
150	21Jul2015	1225	862.15	1043.77
151	21Jul2015	1230	862.19	1076.16
152	21Jul2015	1235	862.24	1122.85
153	21Jul2015	1240	862.3	1175.61
154	21Jul2015	1245	862.37	1235.23
155	21Jul2015	1250	862.35	1196.42
156	21Jul2015	1255	862.35	1200.81
157	21Jul2015	1300	862.32	1179.43
158	21Jul2015	1305	862.27	1123.38
159	21Jul2015	1310	862.25	1119.95
160	21Jul2015	1315	862.21	1076.61
161	21Jul2015	1320	862.17	1047.88
162	21Jul2015	1325	862.15	1046.01
163	21Jul2015	1330	862.13	1028.99

164	21Jul2015	1335	862.15	1044.51
165	21Jul2015	1340	862.21	1091.86
166	21Jul2015	1345	862.27	1148.86
167	21Jul2015	1350	862.34	1209.6
168	21Jul2015	1355	862.38	1226.66
169	21Jul2015	1400	862.36	1210.76
170	21Jul2015	1405	862.33	1177.06
171	21Jul2015	1410	862.31	1166.62
172	21Jul2015	1415	862.26	1127.03
173	21Jul2015	1420	862.23	1093.34
174	21Jul2015	1425	862.18	1062.21
175	21Jul2015	1430	862.17	1058.16
176	21Jul2015	1435	862.18	1067.52
177	21Jul2015	1440	862.22	1099.26
178	21Jul2015	1445	862.27	1148.97
179	21Jul2015	1450	862.35	1213.32
180	21Jul2015	1455	862.45	1299.9
181	21Jul2015	1500	862.42	1268.58
182	21Jul2015	1505	862.39	1226.67
183	21Jul2015	1510	862.37	1214.51
184	21Jul2015	1515	862.3	1150.46
185	21Jul2015	1520	862.25	1108.2
186	21Jul2015	1525	862.2	1075.34
187	21Jul2015	1530	862.19	1073.29
188	21Jul2015	1535	862.2	1080.63
189	21Jul2015	1540	862.22	1096.19
190	21Jul2015	1545	862.24	1112.64
191	21Jul2015	1550	862.25	1123.59
192	21Jul2015	1555	863.37	2381.78
193	21Jul2015	1600	862.82	1589
194	21Jul2015	1605	862.31	1099.21
195	21Jul2015	1610	861.8	728
196	21Jul2015	1615	862.05	1012.11
197	21Jul2015	1620	862.28	1194.19
198	21Jul2015	1625	862.31	1177.16
199	21Jul2015	1630	862.38	1226.16
200	21Jul2015	1635	862.41	1263.36
201	21Jul2015	1640	862.38	1224.19
202	21Jul2015	1645	862.38	1217.55
203	21Jul2015	1650	862.37	1223.18
204	21Jul2015	1655	862.32	1170.33
205	21Jul2015	1700	862.31	1161.47
206	21Jul2015	1705	862.27	1139.17

207	21Jul2015	1710	862.22	1086.68
208	21Jul2015	1715	862.19	1072.51
209	21Jul2015	1720	862.2	1081.97
210	21Jul2015	1725	862.23	1109.48
211	21Jul2015	1730	862.28	1156.86
212	21Jul2015	1735	862.35	1218.86
213	21Jul2015	1740	862.44	1294.82
214	21Jul2015	1745	862.43	1274.2
215	21Jul2015	1750	862.43	1262.98
216	21Jul2015	1755	862.43	1273.18
217	21Jul2015	1800	862.41	1248.9
218	21Jul2015	1805	862.38	1218.84
219	21Jul2015	1810	862.37	1213.7
220	21Jul2015	1815	862.33	1179.16
221	21Jul2015	1820	862.29	1147.73
222	21Jul2015	1825	862.26	1127.75
223	21Jul2015	1830	862.21	1076.81
224	21Jul2015	1835	862.17	1053.97
225	21Jul2015	1840	862.18	1066.78
226	21Jul2015	1845	862.22	1098.79
227	21Jul2015	1850	862.28	1155.3
228	21Jul2015	1855	862.36	1224.7
229	21Jul2015	1900	862.47	1322.62
230	21Jul2015	1905	862.5	1320.88
231	21Jul2015	1910	862.53	1354.27
232	21Jul2015	1915	862.53	1351.68
233	21Jul2015	1920	862.49	1319.85
234	21Jul2015	1925	862.48	1309.08
235	21Jul2015	1930	862.48	1313.05
236	21Jul2015	1935	862.46	1290.38
237	21Jul2015	1940	862.44	1272.48
238	21Jul2015	1945	862.43	1270.89
239	21Jul2015	1950	862.41	1249.49
240	21Jul2015	1955	862.38	1224.21
241	21Jul2015	2000	862.37	1213.65
242	21Jul2015	2005	862.33	1184.37
243	21Jul2015	2010	862.3	1154.22
244	21Jul2015	2015	862.27	1129.64
245	21Jul2015	2020	862.22	1086.4
246	21Jul2015	2025	862.18	1055.84
247	21Jul2015	2030	862.11	1005.7
248	21Jul2015	2035	862.09	997.52
249	21Jul2015	2040	862.1	1013.46

APPENDIX 4 Breach outflow hydrograph using breach parameters from Froehlich

River: kesem river Reach: kesem ds of dame
RS: 33702.87

	Date	Stage INST-VAL METERS	Flow INST-VAL M3/S
1	20Jul2015 2400	931.11	310.38
2	21Jul2015 0005	931.1	663.74
3	21Jul2015 0010	931.07	1798.64
4	21Jul2015 0015	930.89	3653.69
5	21Jul2015 0020	930.8	6335.69
6	21Jul2015 0025	930.64	9766.08
7	21Jul2015 0030	930.36	13910.82
8	21Jul2015 0035	930.04	18805.96
9	21Jul2015 0040	929.63	24439.88
10	21Jul2015 0045	929.08	30869.32
11	21Jul2015 0050	927.93	50922.57
12	21Jul2015 0055	926.5	71311.6
13	21Jul2015 0100	924.44	91442.68
14	21Jul2015 0105	921.94	102854.6
15	21Jul2015 0110	918.81	112619.2
16	21Jul2015 0115	915	119779.9
17	21Jul2015 0120	910.22	123661.5
18	21Jul2015 0125	904.27	123658.8
19	21Jul2015 0130	897.57	118268.3
20	21Jul2015 0135	890.92	105581.5
21	21Jul2015 0140	887.7	85327.77
22	21Jul2015 0145	884.76	69455.02
23	21Jul2015 0150	882.08	56598.68
24	21Jul2015 0155	879.79	46570.15
25	21Jul2015 0200	877.79	38570.9
26	21Jul2015 0205	875.79	31372
27	21Jul2015 0210	874	25569.22
28	21Jul2015 0215	870.74	16323.04
29	21Jul2015 0220	868.73	10810.77
30	21Jul2015 0225	866.97	7213.38
31	21Jul2015 0230	865.29	4302.72
32	21Jul2015 0235	863.67	1933.69
33	21Jul2015 0240	862.78	1337.01
34	21Jul2015 0245	861.18	113.28

35	21Jul2015	0250	861.71	684.97
36	21Jul2015	0255	861.72	704.52
37	21Jul2015	0300	862.1	969.11
38	21Jul2015	0305	862.2	1039.84
39	21Jul2015	0310	862.19	1023.51
40	21Jul2015	0315	862.19	1015.37
41	21Jul2015	0320	862.23	1050.92
42	21Jul2015	0325	862.25	1068.05
43	21Jul2015	0330	862.24	1061.44
44	21Jul2015	0335	862.23	1050.16
45	21Jul2015	0340	862.26	1074.05
46	21Jul2015	0345	862.3	1109.58
47	21Jul2015	0350	862.33	1134.25
48	21Jul2015	0355	862.36	1153.76
49	21Jul2015	0400	862.4	1186.67
50	21Jul2015	0405	862.47	1240.64
51	21Jul2015	0410	862.55	1313.05
52	21Jul2015	0415	862.67	1411.34
53	21Jul2015	0420	862.83	1549.9
54	21Jul2015	0425	862.97	1677.96
55	21Jul2015	0430	862.94	1645.81
56	21Jul2015	0435	862.87	1568.79
57	21Jul2015	0440	862.82	1530.23
58	21Jul2015	0445	862.76	1486.08
59	21Jul2015	0450	862.73	1456.26
60	21Jul2015	0455	862.7	1434.98
61	21Jul2015	0500	862.67	1404.03
62	21Jul2015	0505	862.64	1375.27
63	21Jul2015	0510	862.6	1345.51
64	21Jul2015	0515	862.55	1302.94
65	21Jul2015	0520	862.5	1260.13
66	21Jul2015	0525	862.44	1211.26
67	21Jul2015	0530	862.23	946.03
68	21Jul2015	0535	862.42	1192.32
69	21Jul2015	0540	862.46	1230.65
70	21Jul2015	0545	862.54	1297.88
71	21Jul2015	0550	862.63	1381.54
72	21Jul2015	0555	862.76	1491.87
73	21Jul2015	0600	862.88	1597.2
74	21Jul2015	0605	862.82	1542.84
75	21Jul2015	0610	862.77	1489.57
76	21Jul2015	0615	862.74	1459.4
77	21Jul2015	0620	862.69	1420.55

78	21Jul2015	0625	862.64	1375.11
79	21Jul2015	0630	862.6	1340.98
80	21Jul2015	0635	862.55	1300.59
81	21Jul2015	0640	862.49	1248.64
82	21Jul2015	0645	862.43	1202.68
83	21Jul2015	0650	862.36	1141.61
84	21Jul2015	0655	862.32	1119.38
85	21Jul2015	0700	862.31	1113.48
86	21Jul2015	0705	862.32	1125.59
87	21Jul2015	0710	862.36	1157.99
88	21Jul2015	0715	862.42	1201.41
89	21Jul2015	0720	862.49	1257.5
90	21Jul2015	0725	862.57	1329.38
91	21Jul2015	0730	862.69	1430.89
92	21Jul2015	0735	862.86	1581.24
93	21Jul2015	0740	863.1	1812.47
94	21Jul2015	0745	863.02	1720.12
95	21Jul2015	0750	863.01	1704.16
96	21Jul2015	0755	862.95	1652.13
97	21Jul2015	0800	862.87	1567.34
98	21Jul2015	0805	862.8	1516.97
99	21Jul2015	0810	862.75	1471.58
100	21Jul2015	0815	862.71	1440.7
101	21Jul2015	0820	862.69	1424.02
102	21Jul2015	0825	862.66	1390.86
103	21Jul2015	0830	862.62	1359.03
104	21Jul2015	0835	862.58	1328.12
105	21Jul2015	0840	862.53	1281.22
106	21Jul2015	0845	862.47	1235.34
107	21Jul2015	0850	862.41	1183.78
108	21Jul2015	0855	862.34	1123.18
109	21Jul2015	0900	862.31	1112.87
110	21Jul2015	0905	862.31	1113.66
111	21Jul2015	0910	862.3	1109.94
112	21Jul2015	0915	862.31	1113.17
113	21Jul2015	0920	862.33	1129.26
114	21Jul2015	0925	862.35	1147
115	21Jul2015	0930	862.36	1156.26
116	21Jul2015	0935	862.36	1156.45
117	21Jul2015	0940	862.35	1148.46
118	21Jul2015	0945	862.34	1134.55
119	21Jul2015	0950	862.32	1116.04
120	21Jul2015	0955	862.29	1092.37

121	21Jul2015	1000	862.25	1064.55
122	21Jul2015	1005	862.22	1041.16
123	21Jul2015	1010	862.19	1022.37
124	21Jul2015	1015	862.18	1016.18
125	21Jul2015	1020	862.17	1007.02
126	21Jul2015	1025	862.16	1002.41
127	21Jul2015	1030	862.16	1000.84
128	21Jul2015	1035	862.16	997.76
129	21Jul2015	1040	862.15	992.86
130	21Jul2015	1045	862.15	992.44
131	21Jul2015	1050	862.15	992.43
132	21Jul2015	1055	862.15	989.68
133	21Jul2015	1100	862.14	986.17
134	21Jul2015	1105	862.14	986.28
135	21Jul2015	1110	862.14	985.53
136	21Jul2015	1115	862.13	981.23
137	21Jul2015	1120	862.12	974.32
138	21Jul2015	1125	862.12	970.13
139	21Jul2015	1130	862.11	964.43
140	21Jul2015	1135	862.09	950.6
141	21Jul2015	1140	862.07	936.72
142	21Jul2015	1145	862.06	926.52
143	21Jul2015	1150	862.03	906.67
144	21Jul2015	1155	861.99	879.39
145	21Jul2015	1200	861.97	860.76
146	21Jul2015	1205	861.92	826.18
147	21Jul2015	1210	861.87	793.49
148	21Jul2015	1215	861.82	759.59
149	21Jul2015	1220	861.82	766.3
150	21Jul2015	1225	861.87	798.49
151	21Jul2015	1230	861.96	859.63
152	21Jul2015	1235	862.1	960.43
153	21Jul2015	1240	862.3	1119.02
154	21Jul2015	1245	862.54	1308.75
155	21Jul2015	1250	862.57	1325.43
156	21Jul2015	1255	862.52	1275.25
157	21Jul2015	1300	862.49	1252.88
158	21Jul2015	1305	862.43	1206.01
159	21Jul2015	1310	862.39	1170.34
160	21Jul2015	1315	862.38	1163.43
161	21Jul2015	1320	862.41	1193.19
162	21Jul2015	1325	862.46	1238.3
163	21Jul2015	1330	862.52	1284.62

164	21Jul2015	1335	862.58	1333.44
165	21Jul2015	1340	862.63	1379.79
166	21Jul2015	1345	862.56	1305.66
167	21Jul2015	1350	862.53	1284.34
168	21Jul2015	1355	862.56	1313.51
169	21Jul2015	1400	862.54	1294.21
170	21Jul2015	1405	862.49	1252.44
171	21Jul2015	1410	862.43	1203.84
172	21Jul2015	1415	862.37	1154.95
173	21Jul2015	1420	862.33	1126.17
174	21Jul2015	1425	862.3	1105.83
175	21Jul2015	1430	862.31	1115.69
176	21Jul2015	1435	862.34	1141.29
177	21Jul2015	1440	862.39	1177.87
178	21Jul2015	1445	862.44	1221.02
179	21Jul2015	1450	862.51	1276.61
180	21Jul2015	1455	862.6	1349.97
181	21Jul2015	1500	862.72	1452.29
182	21Jul2015	1505	862.86	1578.97
183	21Jul2015	1510	862.82	1537.89
184	21Jul2015	1515	862.77	1486.54
185	21Jul2015	1520	862.74	1464.01
186	21Jul2015	1525	862.71	1443.89
187	21Jul2015	1530	862.68	1413.01
188	21Jul2015	1535	862.65	1381.41
189	21Jul2015	1540	862.62	1357.75
190	21Jul2015	1545	862.57	1316.22
191	21Jul2015	1550	862.52	1273.37
192	21Jul2015	1555	862.46	1228.6
193	21Jul2015	1600	862.4	1173.09
194	21Jul2015	1605	862.33	1115.49
195	21Jul2015	1610	862.31	1110.67
196	21Jul2015	1615	862.34	1135.05
197	21Jul2015	1620	862.36	1157.14
198	21Jul2015	1625	862.4	1187.2
199	21Jul2015	1630	862.45	1230.55
200	21Jul2015	1635	862.53	1295.82
201	21Jul2015	1640	862.64	1388.15
202	21Jul2015	1645	862.79	1521.11
203	21Jul2015	1650	863.02	1736.06
204	21Jul2015	1655	863.09	1787.16
205	21Jul2015	1700	863.09	1788.89
206	21Jul2015	1705	863.01	1703.2

207	21Jul2015	1710	862.96	1661.29
208	21Jul2015	1715	862.89	1594.99
209	21Jul2015	1720	862.82	1529.03
210	21Jul2015	1725	862.79	1507.24
211	21Jul2015	1730	862.77	1495.91
212	21Jul2015	1735	862.75	1473.26
213	21Jul2015	1740	862.72	1451.65
214	21Jul2015	1745	862.71	1436.35
215	21Jul2015	1750	862.68	1413.16
216	21Jul2015	1755	862.65	1382.94
217	21Jul2015	1800	862.62	1357.09
218	21Jul2015	1805	862.57	1320.66
219	21Jul2015	1810	862.52	1275.81
220	21Jul2015	1815	862.47	1231.46
221	21Jul2015	1820	862.4	1175.38
222	21Jul2015	1825	862.33	1120.7
223	21Jul2015	1830	862.28	1077.07
224	21Jul2015	1835	862.21	1028.3
225	21Jul2015	1840	862.12	966.38
226	21Jul2015	1845	862.04	904.97
227	21Jul2015	1850	861.96	854.83
228	21Jul2015	1855	861.9	812.35
229	21Jul2015	1900	861.93	838.86
230	21Jul2015	1905	862.01	897.51
231	21Jul2015	1910	862.12	973.68
232	21Jul2015	1915	862.27	1094.68
233	21Jul2015	1920	862.27	1084.8
234	21Jul2015	1925	862.2	1024.89
235	21Jul2015	1930	862.19	1018.04
236	21Jul2015	1935	862.2	1028.03
237	21Jul2015	1940	862.17	1009.3
238	21Jul2015	1945	862.12	971.15
239	21Jul2015	1950	862.14	981.75
240	21Jul2015	1955	862.14	985.88
241	21Jul2015	2000	862.1	956.78
242	21Jul2015	2005	862.07	933.66
243	21Jul2015	2010	862.09	946.77
244	21Jul2015	2015	862.06	928.42
245	21Jul2015	2020	862	882.44
246	21Jul2015	2025	862.01	891.11
247	21Jul2015	2030	862.02	897.93
248	21Jul2015	2035	862.01	891.73
249	21Jul2015	2040	861.96	856.6

