ANALYSIS OF DAM BREACH PARAMETERS AND INUNDATION MAPPING (THE CASE OF KALID-DIJO EMBANCKMENT DAM)

Location: Silte Zone, SNNPR

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A THESIS SUBMITTED TO THE DEPARTMENT OF HYDRAULIC ENGINEERING, COLLEGE OF BIO SYSTEMS AND ENVIRONMENTAL ENGINEERING, SCHOOL OF GRADUATE STUDIES HAWASSA UNIVERSITY, ETHIOPIA

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN HYDRAULIC ENGINEERING

January, 2023

ADVISORS' APPROVAL SHEET

SCHOOL OF GRADUATE STUDIES

HAWASSA UNIVERSITY ADVISORS' APPROVAL SHEET This is to certify that the thesis prepared by Mr. Lemeneh Mersha entitled "ANALYSIS OF DAM BREACH PARAMETERS AND INUNDATION MAPPING (THE CASE OF KHALID-DIJO EMBANKMENT DAM)" submitted in partial fulfillment of the requirements for the degree of Master's with specialization in Hydraulic Engineering, the Graduate Program of the Department Hydraulic and Water Resources Engineering, and has been carried out by Lemeneh Mersha Bisetegn ID. No. GPHhydrW/0013/12, under my/our supervision. Therefore I/we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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APPROVAL PAGE

We, the undersigned, members of the Board of Examiners of the final open defense by Mr. Lemeneh Mersha have read and evaluated his thesis entitled **"ANALYSIS OF DAM BREACH PARAMETERS AND INUNDATION MAPPING (THE CASE OF KHALID-DIJO EMBANCKMENT DAM)"** and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree.

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ACKNOWLEDGEMENTS

First and foremost, I would like to thank the almighty God for being my support in all my journey of life. Second, to my adviser Dr. Moltot Zewdie and Co-advisor Girma Gadissa (MSc.) who guided me in doing this thesis. They provided me with invaluable advice and helped me in difficult periods. Their motivation and help contributed tremendously to the successful completion of the thesis.

Besides, I would like to thank Ministry of Water, Irrigation and Electricity of Ethiopia (MoWIE-Ethiopia) and SNNPRS Irrigation Development and Schemes Administration Agency who helped me by giving advice and providing the materials which I needed.

Also, I would like to thank my family and friends especially Mr. Gashaw Tamirat and Mr. Tewodros Lakew for their support. Without that support I could not have succeeded in completing this thesis.

At last, but not in least, I would like to thank everyone who helped and motivated me to work on this project.

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DECLARATION

I hereby declare that this MSc Specialty or equivalent thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis/dissertation have been duly acknowledged.

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This MSc Specialty or equivalent thesis has been submitted for examination with my approval as thesis advisor.

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Abbreviations and Acronyms

1D:	One-Dimensional
2D:	Two-Dimensional
DEM:	Digital Elevation Model
DSM:	Digital Surface Models
DTM:	Digital Terrain Models
EMA:	Ethiopian Map Agency
ECDSWC	Ethiopian Construction Design and Supervision Works Corporation
FEMA:	Federal Emergency Management (US. government)
GIS:	Geographic Information System
HECHMS:	Hydrologic Engineering Center Hydrology Modeling System
HEC- RAS:	Hydrologic Engineering Center River Analysis System
ICOLD:	International Committee of Large Dams
IDF:	Inflow Design Flood
Masl:	Meter above sea level
MODFLOW:	Modular Finite difference flow model
MoWIE:	Ministry of Water, Irrigation & Electricity
no:	Manning Coefficient
NDRSB:	National Disaster and Response Bureau US.
NOAA:	National Ocean and Atmospheric Administration US.
SDF:	Spill Way Design Flood
SRTM:	Shuttle Radar Topography Mission
SS:	Side Slope
TFH:	Time to Failure Hour
USACE:	United States Army Corps of Engineers
USBR:	United States Bureau of Reclamation
USDOI	United States Department of Interior
USGS:	United States Geological Survey

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Abstract

Embankment dam risk assessment is a method used to evaluate the catastrophic impact of dam break flooding. This study was conducted to analyze dam breach parameters of Kalid Dijo dam and inundation mapping. The dam breaching outflow hydrographs, inundation mapping and downstream flood propagations were simulated by applying HEC-RAS 5.0.6 model to study the possible relationships among the peak flows, dam breach parameters, and map the inundation area. The initial parameters are taken from literature based on the dam type and its silent features. HEC-RAS model 2D reservoir and river routing were performed and resulted in multiple breaching outflow hydrographs for specified conditions at the dam site and at specified downstream reach stations. The maximum breach discharge resulted from the model was 852.18 m^3/sec which results in overtopping the dam by 0.6m. Four Time to Failure Hour and four Breach Side Slope values are used for the sensitivity analysis at the dam site. Accordingly, the change of Time to Failure Hour resulted 403% change of the maximum discharge at the dam site which is high compared to the change in Breach Side Slope which produced 100% reduction in maximum discharge. This implied that the peak discharge was highly sensitive to changes in Time to Failure Hour more than five times than Breach Side Slope; and Time to Failure Hour is a controlling parameter at the dam site. At two randomly selected downstream observation stations 01 and 02, an increase in TFH and increase in Manning coefficient resulted in decrease in peak flow at reach station-01 and 02. The total inundated area is 454.19 sq.km. It covers 1100ha of command area and some parts of Dalocha town.

Key Words: Dam Breach, Breach Parameters, Sensitivity analysis, Inundation Mapping

1. Introduction

1.1. Background

From the early period of civilization, construction of dams has been a long-established practice. Dams are one of the infrastructures that play a vital role in meeting water demand of the society by providing many benefits like flood control, water supply, irrigation, hydropower, navigation, and recreation. These benefits provided by dams come at a risk due to their potential to fail and cause catastrophic flooding. For this reason, mitigation of this risk is essential by identifying potential failure modes, simulate the potential failure and protect against them for the benefit of the society as floods resulting from the failure of dams produce devastating disasters (Wahl, 2010). This dam failure caused for a number of reasons and its consequence leads to requirement for preparation of dam breach inundation modeling and mapping to identify the flood risk and mitigate the consequences. As the report of (FEMA, 2013) Dam breach inundation mapping received attention following two significant failure incidents in California (Baldwin Hills Dam in 1964 and Lover Van Norman Dam in 1971). This prompted the state of California to prepare dam failure inundation maps.

Therefore, embankment dam risk assessment is a measure that is commonly adopted to help mitigate the catastrophic impact of dam break flooding. While a very unlikely occurrence, extreme weather events, critical construction deficiencies and other unforeseen circumstances have led to many major dam failures in the last two centuries. These have resulted in significant loss of life, as well as economic, social and environmental damage. The physical breaching of a dam is a difficult phenomenon to predict and model, due to the complex interaction of hydraulic, structural and geotechnical properties. As such, many different methods with varying complexities have been developed to allow the ability to estimate the geometry and outflow of an embankment breach (Morris, M. and Hassan, M. , 2018).

The modeling of potential dam failures is an important activity for those who work to maintain the safety of dams, and for those who must plan for the potential downstream consequences of a failure. Methods for modeling dam failure vary in scope and complexity, depending on analysis needs, funding, availability of input data, dam type, failure mode and other factors. Embankment dams failing due to erosion caused by overtopping flow, seepage through the embankment, or seepage through the foundation or at embankment-foundation interfaces comprise a large segment of the inventory of dams for which failure analyses are required. Today, the most sophisticated modeling tools for these erosive failures of embankment dams utilize physically-based models that simulate erosion and dam failure on a time-step basis. A simpler approach taken for many studies is to model the process of breach development in a parametric way, defining the starting and ending points in the breach development process and simulating intermediate conditions using simple functional relationships that mimic the characteristics of breach development, but do not specifically simulate physical erosion processes (Wahl T. L., June 2014).

One of the biggest advances in the analysis of dam safety during the last decade has been the evolution of flood routing, both upstream and downstream, for dam break scenarios which are overtopping and piping. The availability of free and accurate topographic data and satellite imaging of high resolution, along with improvement of the software and methods of modeling, improved the dam break analysis techniques for dam safety studies. Currently, the use of two-dimensional depth-averaged (2DH) surface water flow models has facilitated production of flood inundation maps that are more realistic than those obtained using one dimensional cross-section averaged (1D) models, because of the capacity of these new models to propagate the flow in many directions.

The numerical solution schemes of these models make use of a computational mesh created on a digital elevation/surface/terrain model (DEM, DSM or DTM), provides, which yields reliable results throughout a floodplain, not only at stream cross-sections. This evolution has allowed more accurate flood hazard area estimation by supplying information such as velocity vectors, water depths, and flood wave arrival time, which in a allows analysis allow estimates of loss of life and property damage (Gonzalez, 2017).

HEC-RAS v-5.0 (Brunner G. W., 2014) is one of the newest models developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. It includes 2DH modeling capabilities that differ from those provided in other 2DH models, mainly because of how it processes the terrain data, and in how this feature can decrease computation time and increase accuracy of results. A description of the main features of how to develop a dam breach study with HEC-RAS and its main capabilities to carry out a flood hazard mapping is given here. Performing a sensitivity analysis of dam break parameters helps to study the relationships among the parameters that are involved in dam breach and flood propagation processes. The methods applied in conducting the sensitivity analysis of dam break parameters include hydrologic and hydraulic routing techniques.

These techniques help to predict dam-break breaching outflow hydrographs and flood wave propagation and provide the information regarding the wave front arrival time, inundated area, and flow depth.

Khalid-Dijo embankment dam which is located in Dijo River, the Southern Ethiopia is the tributary of Lake Shala. This dam is planned to store surface-runoff during the rainy season and used for supplementary irrigation during the wet season to fulfill the shortage of rain and for dry season irrigation purposes for 1100-hectares land (ECDSWC, 2020).

This study, dam break and sensitivity analysis of dam breach parameters using Khalid-Dijo dam as a testing basis, involved estimating the key parameters, dam break outflow hydrograph, time to dam failure, side slope of breach and Manning coefficients. The influences of parameters on peak flows were analyzed at the specified reach stations and dam site.

1.2. Statement of the Problem

Ethiopia is recently developing large dams for different purposes like water supply, irrigation & hydropower. It is obvious that constructing a dam has a huge contribution in changing the people's way of life but sometimes we need to ask ourselves "What if the dam fails? What causes the dam to fail? What will be the peak flow? What influences the breach peak outflow?"

In fact, in our country for most of the developed and under-development dams, pre-event analyses are 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 area for various infrastructures, alert concerned bodies to take a precaution on dam safety plans and formulate a hazard management system.

In this study area/Khalid-Dijo dam, most of the catchment area for the dam consists Worabe town, which is currently the town goes under development, so there is an increase in urbanization through the catchment. When urbanization increase, at the same time impervious area increase, which also increases the runoff throughout the catchment. Because of an increasing in runoff, the inflow to the dam becomes greater than the expected design flood. Such an increase in the inflow through the dam cause an overflow in the dam body. Consequently, dam failure may happen. Sudden release of impounded water from a dam has probability of interfering the activity of people living at the downstream as well as mechanized irrigation farms and associated infrastructure developed for Kalid-Dijo Irrigation Project and damage mechanized farms.

Therefore, the pre-event analysis aimed to model the possible breaching process of the embankment dam and to delineate the area that can be flooded out due to the hazardous wave front. The problems that might predict are; damage of the embankment and its appurtenant structures which are constructed with high investment, Loss of irrigable land and Inundation of downstream area causing loss of human life and property.

Efforts were made to assess gaps of previous experiences in breach modelling and flood inundation mapping and different considerations with detail study was carried out to fill some gaps as much as possible.

1.3. Objectives

1.3.1. General Objective

To undertake the analysis of dam breach parameters case of Kalid-Dijo Dam.

1.3.2. Specific Objectives

- To determine the maximum breach outflow at the dam site and route the outflow through the downstream reaches.
- To investigate the sensitive breach parameters to maximum discharges and determine the most sensitive parameter at the dam site.
- To examine the sensitivity of peak flows for specified Manning coefficients and time to failure
- Develop the flood map of the inundation area and classify them based on flood hazard levels

1.4. Research Questions

The research questions that were addressed in this paper were:

- What will happen if a probable maximum flood appears?
- What will be the sensitive parameter for maximum breach outflow?
- How does the manning coefficient/ roughness coefficient affect the breach outflow in the downstream of the dam?
- What will be the extent of the inundation area and its consequences?

1.5. Significance of the study

Preparing dam safety plans and hazard management strategies are unquestionably vital, since lots of human lives have been lost and tremendous amount of economic crisis have been recorded from dam failure events throughout the world in history. Setting out risk management, emergency action plans or evacuation planning system to protect both lives and materials during sudden dam failure phenomena and resulting flood waves is highly essential.

Hence, conducting pre-event analysis of Kalid Dijo dam break and its consequences and forwarding the hazard extent to public sectors have great significances among which some are listed below:

- Flood early warning system can be developed after the disastrous dam breaching flood has been identified.
- Downstream settlement of inhabitants can be planned in such a way that evacuation of the people during dam breach can be done rapidly saving as much lives as possible.
- Protection dikes can be provided at both downstream banks with their top level considerably higher than the propagating flood level, especially on the stretches along residence areas and road and irrigation infrastructures.
- The thesis paper can be used as a reference for further detailed researches on the study area.

2. Literature Review

2.1. Introduction

In Ethiopia, there was no dam breach event recorded so far but some of the major dams are suffering from piping and structural stability problems which may lead to breach and considerable loss. To mention some among this dam, Kesem is observed to have a $1m^3$ /s flow along its left side abutment and Tendaho have a total leakage of nearly 0.5m3 /s (WWDSE, March 2011).

The most important component of dam break analysis is the definition of the reasonable breach parameters, which are highly difficult to be accurately predicted. In dam safety programs and flood forecast, dam break modeling is essential to evaluate dam induced risk and to support emergency plan, and directing first response teams for the area that may be damaged due to the consequence if the dam will break. Breach width and breach time have a great influence on the forecast of the outflow and the flooded area downstream of the dam (Natale.E., 2009). The dam is represented as a structure in the river setup when the dam break, the momentum equation is replaced by the broad crested weir flow equation which describes the flow through the structure (Sachin, 2014). Using the standard dam breach methods, the breach is initiated either as a trapezoidal breach or if the erosion-based method is used as a circular piping failure (Sachin, 2014).

2.2. Dam Breach Analysis

Dam breach analysis usually relates to the process of studying a dam failure phenomenon and analyzing the resulting potential flood hazards at the downstream region. This generally deals with simulation of assumed failure for dams and analyzing the resulting consequence (Pandya, P. H. & Jitaji, T. D., 2013).

Dam breach inundation analyses include the following four elements (FEMA, 2013).

- Estimation of the dam breach parameters,
- Estimation of the dam breach outflow hydrograph
- Routing of the dam breach hydrograph downstream and
- Estimation of downstream inundation extent and severity.

2.2.1. Dam Breach Parameters Definition

The following definitions are commonly accepted for use in evaluating and selecting dam breach parameters.

• **Breach formation time (also time-to-failure)** – The duration of time between the first breaching of the upstream face of the dam (breach initiation) and when the breach has reached it full geometry.

• **Breach depth (also breach height) –** The breach depth is the vertical extent of the breach measured from a specific elevation to the invert of the dam breach.

• **Breach width** – The breach width is the average of the final breach width, typically measured at the vertical center of the breach.

• **Breach side slope factor** – The breach side slope is a measure of the angle of the breach sides represented as X horizontal to 1 vertical (XH: 1V).

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

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

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

Breach formation (defined above) begins when the flow through the dam has increased and progressed from the upstream face to the downstream face of the dam. It is uncontrolled, and will result in the failure of the dam (FEMA, 2013).

2.2.2. Dam Breach Parameters Estimation

The geometric description of a dam breach must be estimated to simulate the resultant flood wave and downstream consequences. There are four critical elements of any breach analysis:

i. Breach parameter estimation (breach size/shape and time of failure)
ii. Breach peak discharge and breach hydrograph estimation
iii. Breach flood routing and
iv. Estimation of the hydraulic conditions at critical locations.

The most commonly used approaches for the required elements of the analysis are as described below by California Department of Water Resources (CDWR, 2010).

2.2.2.1. Comparative Analysis

The simplest approach to dam breach flood estimation is comparative analysis. This method compares a given dam of interest with those in a database of well documented dam failure case histories. A given dam geometry, height, slope angles, and reservoir areas and volumes are compared with a list of similar sized dams that have failed. Dam breach parameters and peak discharges reported from the failure case histories of similarly configured dams are then directly applied to the dam being analyzed.

2.2.2.2. Parametric Regression/Empirical/ Equations

Parametric regression equations are empirically derived using case study information to estimate the time-to-failure and ultimate breach geometry, then simulate breach growth as a time dependent linear process to compute breach outflows using principles of hydraulics. Numerous equations to predict breach parameters have been developed based on analyses of case studies. The equations stated in Table 2: Published Parametric Regression Equations for Predicting Breach Parameters (FEMA, 2013).

These empirical regression equations were developed to predict the average breach width, breach depth, and time-of-failure or formation time depending actual failure of dams. MacDonald and Langridge Monopolis (1984), Froehlich (1995), Froehlich (2008) and Von Thun and Gillette (1990) were among empirical regression equations which to calculate the breach parameters with the help of HEC-RAS software for this study.

(Wahl T. L., 2010) suggests that one of the main advantages of using empirical parametric regression equations is that the user can exhibit some control over the breach parameters used in the model, and thus account for site - specific factors.

2.2.2.3. Predictor Regression Equations

Predictor regression equations are empirically developed equations used to estimate peak discharge based on actual case study data. These equations are used as a prediction method to determine a reasonable outflow hydrograph shape. The equations listed in Table 3: The empirical relationships developed by various authors for predicting peak breach discharge (FEMA, 2013). MacDonald and Langridge Monopolis (1984), Froehlich (1995), Froehlich (2008) and Von Thun and Gillette (1990) were among empirical regression equations which to calculate the peak discharge for the given dam breach analysis with the help of HEC-RAS model.

2.2.2.4. Physically Based Erosion Models

A physically-based model (also referred to as a “process” or “causal” model) utilizes generally accepted relationships based on physical principles to establish the framework of a model. The model then attempts to solve those relationships for a given input. This is a relatively simple concept, but it can become very complex when the input is changing with time. In the case of dam breach analysis, both the input and physical constraints are changing with time as the dam erodes and the reservoir evacuates.

Although several physically-based models have been reported as being in the development stage for research purposes, the National Weather Service’s BREACH program (NWS BREACH or BREACH) is currently the only widely available model. BREACH predicts the development of a breach and the resulting outflow using an erosion model based on principles of hydraulics, sediment transport and soil mechanics. It was initially developed in 1987, but has had several upgrades in 1988, 1991, and 2005. The model takes into account several components of a dam and reservoir that are not considered in the empirical methods, such as area versus elevation, dam dimensions, soil properties of the dam, and tail water effects downstream. It is relatively simple to run and is widely used within the United States (FEMA, 2013).

Unfortunately, BREACH is no longer supported by the National Weather Service and significant advances in the understanding of the complex mechanics of a dam failure have not been incorporated (Wahl T. , 1998). Also, the model has only been calibrated with a very limited number of cases.

2.2.3. Breach Mechanisms for Embankment Dams

Although breaching in embankment dams may occur for a variety of reasons, breaches in embankment dams are most often modeled as overtopping or piping failures.

2.2.3.1. Overtopping Failures

Overtopping failures can occur very differently depending on the composition of the dam. Perhaps the simplest overtopping failure to discuss is failure of a cohesive soil embankment. According to a study by Ralston (1987), a small head cut typically forms on the downstream face of a cohesive soil embankment and progresses upstream. The breach is considered to begin when erosion occurs across the width of the dam crest. After the breach initiates at the top of the dam crest, it enlarges to its ultimate extent. If there is no physical reason to believe the embankment would fail at a certain location, the breach should be modeled as initiating at the maximum section typically located at the centerline of the downstream main channel.

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

• **Linear progression:** rate of erosion remains the same for the duration of erosion development.

• **Sine wave progression:** breach grows very slowly at the beginning and end of development and rapidly in between

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

2.2.3.2. Piping / Internal Erosion Failures

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

Piping failures are typically modeled in two phases, before and after the dam crest collapses. Water flow through the piping hole is modeled as orifice flow before the dam crest collapses and as weir flow after the dam crest collapses. For small dams constructed from cohesive soils, it is possible for the reservoir to completely empty before the dam crest collapses (State of Colorado Department of Natural Resources, 2011).

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

2.3. Dam Breach Outflow Routing

Routing of the breach outflow hydrograph downstream of the flood plain has a great advantage to evaluate the potential consequence of dam failure in the dam breach analysis and downstream risk area identification studies. Routing of the breach hydrograph is performed to evaluate the attenuated or reduced peak discharge at critical locations downstream of the dam. In addition to calculating the attenuation, determining the flood wave arrival time and the depth/velocity of flow at those critical locations are also very important parts of the analysis (Garcia-Martinez, r., Gonzalez-Ramirez, n. & O'brien, J, 2009).

The propagation of flood waves in a natural channel is a gradually varied unsteady flow process, which is governed by conservation of mass and momentum equations. The solution of these equations in a distributed manner is referred to as distributed routing of flood waves. Routing by distributed system methods is called hydraulic routing and the flow is calculated as a function of both space and time. When no spatial variability is taken into account and when the channel reach or reservoir is considered as a black box, the corresponding routing procedure is referred to as lumped routing. Routing by lumped system methods is called hydrological routing. These methods calculate the flow as function of time alone (Chow, 1964)

2.3.1. The Inflow Flood through the Reservoir

HEC-RAS can use three methods to route inflow hydrographs through reservoirs, based on which equations are solved:

- One-dimensional cross section averaged unsteady flow routing (full St Venant equations)
- Two-dimensional depth averaged unsteady flow routing (full St. Venant equations or diffusion wave equations)
- Level-pool routing

In practice, one of the most important items that must be taken into account before choosing any routing method is the availability of data to describe the reservoir. For an unsteady flow routing, detailed bathymetric/cross section data inside the reservoir, which may be difficult to acquire, are needed. For this reason, level-pool routing may be best approach because only the reservoir elevation -volume relationship is needed. (Gonzalez, 2017).

2.3.1.1. Hydrologic Routing Techniques

Besides the techniques incorporating both St. Venant equations, there are several methods that use the continuity equation only. These hydrological flood-routing methods are very popular. In these techniques, only the wave propagation is studied by considering the increases and decreases of storage in a reach lying between two measuring points. The hydrological model is much simpler, and it is easier to take account of additional inflows from a variety of sources. However, because the relationship between storage and flow is determined empirically by these methods, they cannot be used directly when flow data or levels are required for design purposes (Fread, D.L, 1991).

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

2.3.1.2. Hydraulic Routing Techniques

In hydraulic routing, the flow is described through a set of hydrodynamic differential equations of unsteady-state flow and simultaneous solutions of those equations lead to determination of the outflow hydrograph. Hydraulic routing is based on the principles of hydraulics in which flow is computed as a function of time at several locations along the conveyance system.

The Continuity and Momentum equations are considered to be the most accurate and comprehensive solution to 1-D unsteady flow problems in open channels. Nonetheless, these equations are based on specific assumptions, and therefore have limitations. The assumptions used in deriving the 1-D unsteady flow equations are as follows (FEMA, 2008):

a. Velocity is constant and the water surface is horizontal across any channel section.
b. All flows are gradually varied with hydrostatic pressure prevailing at all points in the flow, such that vertical accelerations can be neglected.
c. No lateral secondary circulation occurs.
d. Channel boundaries are treated as fixed; therefore, no erosion or deposition occurs.
e. Water is of uniform density, and resistance to flow can be described by empirical formulas, such as Manning’s and Chezy’s equation.

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

2.3.2. Dam-Break maximum breaching outflow verification

After the analysis is completed and the hydrograph developed, it is necessary to check the reasonableness of the maximum breaching outflow Qmax. There are a few commonly known techniques to check Qmax: historical predictor equations, parametric models, physically based erosion methods, direct comparison techniques, customized prediction equations, classical equations, FERC recommended equations and current OES recommended equations. The rule of the thumb is to check Qmax obtained in one method with the result of the other techniques. In addition to the peak flow equations, one can also compare computed model peak outflows to envelope curves of historic failures. When comparing computed results to the envelop curve shown below keep in mind that this envelope curve was developed from only fourteen data sets, and may not be a true upper bound of peak flow versus hydraulic depth (USACE, 2014).

2.4. Dam Breach Analysis Study Approaches

The two-primary dam breach study approaches used by State governments and Federal agencies in USA are an event-based approach and a risk-based approach. The event-based approach has been traditionally the most widely used for dam breach analysis. The event-based approach is a deterministic method based on specific precipitation and non-precipitation events for the dam breach analysis and downstream inundation mapping. For the event-based approach, both a non-hydrologic “fair weather failure,” also referred to as a “sunny day failure,” and a specific hydrologic failure event, such as the Probable Maximum Flood (PMF), are usually established based on a dam’s hazard potential classification (FEMA, 2013).

2.4.1. Event-Based Approach

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

2.4.1.1. Fair Weather (Non-Hydrologic) Failure

As defined by FEMA (2013) a fair weather (Sunny Day) breach is a dam failure that occurs during fair weather (i.e., non-hydrologic or non-precipitation) conditions. A fair-weather breach is analyzed by establishing an initial reservoir water level and commencing a breach analysis without additional inflow from a storm event. A fair-weather breach is typically used to model piping failures for hydrologic, geologic, structural, seismic, and human-influenced failure modes. Base flow conditions for a fair-weather failure are typically ignored because of the small discharge and volume compared to that of a dam breach. As a general guidance, base flow can be ignored if the dam breach flow is two times greater than the base flow. Where base flow is considered, the discharge is typically estimated based on reported base flows through the dam's outlet works or from stream gage records. The three most common initial water level elevations for fair weather breach analyses are as follows:

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

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

b) Invert of Auxiliary Spillway (lowest uncontrolled spillway)

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

c) Top of Dam / Maximum Pool Level

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

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

2.4.2. Risk-Based (Consequences-Based) Approach

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

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

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

2.4.2.1. Inflow Design Flood and the Incremental Hazard Evaluation

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

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

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

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

2.4.2.2. Loss of Life / Population at Risk

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

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

2.4.3. Tiered Dam Breach Analysis

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

A tiered study approach was developed by the USDOI and is presented in their report titled Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report (USDOI, March 2011). The tiered dam breach analysis approach presented in this document adapts the USDOI approach and provides additional detail.

2.5. Potential Uses of Inundation Mapping

The primary purpose of an inundation map is to show the areas that would be flooded and travel times for wave front and flood peaks at critical locations if a dam failure occurs or there are operational releases during flooding conditions.

Inundation maps have a variety of uses including EAPs, mitigation planning, emergency response, and consequence assessment. Each use has unique information requirements and may be used in different manners (FEMA, 2013).

2.5.1 Emergency Action Plans

An EAP is a formal document that identifies potential emergency conditions at a dam and specifies preplanned actions to be followed to minimize property damage and loss of life. The EAP specifies actions the dam owner, in coordination with emergency management authorities, should take to respond to incidents or emergencies related to the dam. It presents procedures and information to assist the dam owner in issuing early warning and notification messages to responsible downstream emergency management authorities.

Depending on the inundation map developed, the EAP helps to assist the dam owner and emergency management authorities with identifying critical infrastructure and population-at-risk sites that may require protective measures and warning and evacuation planning. The EAP must clearly delineate the responsibilities of all those involved in managing the incident and how those responsibilities should be coordinated.

2.5.2. Emergency Response

Emergency response embodies the actions taken in the immediate aftermath of an incident to save and sustain lives, meet basic human needs, and reduce the loss of property and the effect on critical infrastructure and the environment. In the case of dam failures and incidents, this would be the response by the dam owner, local community emergency management, and first responders such as fire and police departments to minimize the consequences of an imminent or actual dam failure or incident.

Given the short warning times typically encountered with dam failures and incidents, dam emergency evacuation plans should be developed before the occurrence of an incident. It is recommended that plans be based on a worst-case scenario and address the following elements, including identifying the roles and responsibilities for all action items:

- Identification of critical facilities and sheltering
- Initiating emergency warning systems (who is responsible and what is the method)
- Specific evacuation procedures, including flood wave travel time considerations (for example, evacuation of special needs populations and lifting evacuation orders)
- Distance and routes to high ground
- Traffic control measures and traffic routes
- Potential effect of weather or dam releases on evacuation routes (for example, identify whether portions of the evacuation route may be flooded before the dam incident occurs)
- Vertical evacuation/sheltering-in-place
- Emergency transportation
- Safety and security measures for the dam perimeter and affected areas Re-entry into affected areas

Although the EAP does not need to include the actual evacuation plan, it should indicate who is responsible for an evacuation, and what plan will be followed.

2.5.3. Hazard Mitigation Planning

Mitigation is the proactive effort to reduce loss of life and property by lessening the effect of disasters. This is achieved through identifying potential hazards and the risks they pose in a given area, identifying mitigation alternatives to reduce the risk, and risk analysis of mitigation alternatives. In the case of dam failures and incidents, hazard mitigation planning involves identifying the population at risk and identifying actions to reduce their vulnerability.

Hazard mitigation planners need digital data that defines the dam breach hazard. Information needed includes the breach inundation zone boundary, depth of flooding, velocity, and timing.

2.5.4. Dam Breach Consequence Assessment

Dam breach consequence assessment includes identifying and quantifying the potential consequences of a dam failure or incident. While hazard mitigation planning focuses on specific projects to reduce flood risk, consequence assessment focuses on the economic and social impacts of a potential disaster and the organizational and government actions needed in the aftermath of a dam breach to respond and recover. Data compiled for a consequence assessment can also be used in risk assessments.

2.6. Classification of Hazard

According to FEMA 2013 (Federal emergency management agency), hazard caused by dam failures is classified as;

i. High hazard dam

High Hazard Dam is a dam for which loss of human life is expected to result from failure of the dam. Designated settlements located downstream within the bounds of possible inundation should also be evaluated for potential loss of human life. It is important to note that the potential of loss of a single life is sufficient to classify a dam as high hazard (Claudia C Hoeft & Mark Locke, 2010).

ii. Significant hazard dam

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

iii. Low hazard dam

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

iv. No public hazard dam (NPH)

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

The hazard classification also triggers the requirement to prepare an emergency action plan, requiring preparation of inundation maps which accurately predict dam breach flood depths and arrival times (flood warning time) at critical locations. Adverse effects of a dam failure can be classified in to four categories.

1. **Damage to people:** in principle, apart from loss of life, damage to people could also consider other aspects such as people injured with different degrees of gravity. However, due to the difficulty of quantification of wounded numbers, quantitative analysis usually focuses only on the first aspect.
2. **Direct economic damage:** damage caused directly by the impact of the flood and is the most visible type. It includes the cost associated with the damage suffered by the dam itself.
3. **Indirect economic damage**: damage happened after the event as a result of the interruption of the economy and other activities in the area.
4. **Other damages:** these are related to environmental damage, social disturbing, loss of reputation, damages to historical or cultural heritage, etc. However, all of these aspects are difficult to quantify and so are usually treated in a qualitative way. FEMA guidelines (US Dep LS, 2013) recommends a three-step rating system that defines low hazard, significant-hazard and high-hazard potential classifications depending on the potential for loss of life, economic loss, and environmental damage resulting from a hypothetical dam failure.

2.7. Dam Breach Modeling Techniques

Numerical models used to carry out the dam break study and flood routing which may be one-dimensional, two-dimensional or combination of both. When the channel width increases or the flow becomes non-channelized, accuracy of one-dimensional models decreases hence two-dimensional models are more reliable while carrying out floodplain study. Most widely used hydrodynamic models used to determine dam breach hydrograph and flood routing are parametric models. Parametric models can either be hydrologic or hydraulic models. Hydrologic models like HECHMS solve equation of continuity and these models have the advantage of simplicity, easy to use and efficiency of computation. These models provide attenuated flood hydrograph at required location but they do not provide accurate result over water surface elevation and flow velocity. Because of the constraints of the hydrologic models, for most dam break and flood mapping study, unsteady flow and dynamic routing method is adopted. This method is called as transient flow or hydraulic routing and these models solve equation of continuity and equation of momentum and hence, they are adopted to predict the dam breach flood formation and its progression in downstream reaches. The hydrodynamic modelling involves solution of two partial differential equations derived by Barre De Saint Venant in 1871. The equations are a) Conservation of mass equation or Continuity equation and b) Conservation of momentum equation. Some of the models used for dam break study and flood inundation mapping are described below:

DAMBRK MODEL was developed by NWS, USA in 1984 and it was updated by BOSS International. This model predicts dam breach wave formation and its progression in downstream. The important three features of DAMBRK are ability to describe dam failure mode temporarily and geometrically, computation of the outflow hydrograph through the breach section and its ability to route the outflow through a downstream channel. This model is good in determining potential influenced area allowing users to input geometric and temporal data to accurately predict the initial breach wave. This includes both piping and overtopping failures. The updated BOSS DAMBRK has many improvements including faster calculations and it has graphic interface.

FLO-2D model is developed based on MUDFLOW model in 1989. This predicts the flood hazard, mudflow and debris flow over alluvial river and this uses grid system to determine floodplain based on elevation, roughness coefficient and it is good in predicting flow path and area. In this model, sediment laden and without sediment flow can modelled. In this model, discharge is estimated based on depth of flow over each sector and summing up all sector on all the four sides of grid. Accuracy of the model is dependent on the density of grid system and the data available.

FLDWAV model was developed by the National Weather Services, USA as a replacement to DAMBRK model. This model has wave front tracking for more accuracy and computational time required is less. This allows dam breach prediction and potential concerned area are calculated. This model is designed to model rapid flood events from large precipitation event or dam break occurrences predicting flow through single stream or network of streams using real forecasting technology and considers terrain and properties of material at different time interval and adjust flow pattern. This model also has secondary function to predict flow through hydrological structures and river basins and it performs dam break analysis, flood predictions, pumping stations and other rapid flow scenarios. It allows flow to change from subcritical to supercritical vice versa based on location and time and can model one dimensional unsteady debris flow also.

SMPDBK model is developed by National Weather Services (NWS), USA as simplified version of DAMBRK. This is good for obtaining dam classification and potential dam break risk. Unlike DAMBRK this model is quick and easy to use and does not require many parameters but provides similar results compared to DAMBRK when performed on simpler cases. The model produces the information needed to find out the downstream inundation areas. The SMPDBK model does not require a computer with high specification to run the model and it is efficient model to simulate the dam break scenario and for preparing emergency action plan.

Danish Hydraulic Institute (DHI) MIKE 11 is a new generation software comprising of fully windows integrated graphical user interface. It is 32 bit and 64-bit application having fast computation speed compared to earlier versions of MIKE and other software. It is fully dynamic, user-friendly, one-dimensional tool for the break analysis which simulates flood waves, sediment transportation, water quality in channels or water bodies, etc.., The main feature of MIKE 11 is that it is an integrated modular structure with various add on modules like hydrology, advection – dispersion, water quality model, cohesive sediment transport, non-cohesive sediment transport, hydrodynamic model, rainfall – runoff model and flood forecasting model. Hydrodynamic module is one of the core modules of MIKE 11 package simulating unsteady flow in open channel. It uses implicit finite difference method for simulation.

Danish Hydraulic Institute (DHI) MIKE 2 is a two-dimensional modeling software developed based on MIKE 11 for 2D free surface flows and the model perform study on Newtonian fluids over initial dry terrain. It is used for the two-dimensional simulation of the flow in rivers and estuaries and predict a wide range of floodplain situation in two-dimension. This allows sub-critical, super-critical and mixed flow condition of flow and it is applicable to the designing of the coastal and offshore structure, inland flooding and overland flood modeling. Care should be taken when modelling heavy steeply rising floods and shallow wave fronts. It uses a finite difference method to solve the numerical solution of the two-dimensional shallow water equations, the depth integrated incompressible Reynolds averaged Navier Stokes equations subject to Boussinesq assumption and the presence of hydrostatic pressure.

Hydraulic Engineering CENTRE'S River Analysis System (HEC-RAS) is developed by USACE's Hydrologic Engineering Center. In the previous versions of HEC-RAS, it was possible to simulate the flow as one-dimensional flow only using full Saint Venant equation to simulate the flow. In February 2016, RAS model with ability to perform 2D hydrodynamic unsteady flow routing using Saint Venant equation or Diffusion wave equation was introduced. HEC_RAS model simulate the flow in river channels and floodplains and the model was considered as efficient model for predicting downstream flooding effects from an upstream event i.e., by dam break study. HEC-RAS model uses the breach information and breach geometry as input data to simulate the dam break model. The model simulates the resulting flood wave generated based on the consequences of an upstream event and models downstream effects based on the results of dam break studies. It works with both off channel and on channel reservoir storages, dam break modeling and mixed flow analysis.

2.8. Recommendations for Selecting Modeling Software

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

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

Summarizes guidance on a balanced tiered approach to dam breach modeling cross referenced to the most commonly used one- and two-dimensional model used for dam breach studies. Regulatory authorities should be consulted for allowable use of propriety non-Federal and two-dimensional models not listed in Table 2.1.

Table 2. 1 Recommended Model Types for Various Levels of Dam Breach Modeling (FEMA, 2013).

Tier Level	Applicable to	Peak Breach Discharge Prediction	Downstream Routing of Breach Hydrograph
Tier 1-Screening and Simple Analysis (basic method)	Low-hazard potential / small-size or first-level screening for significant or high-hazard dams	Regression equations, NWS SMPDBK, GeoDamBREACH or TR-66, HEC-HMS or DSAT	GeoDamBREACH, SMPDBK, HEC-RAS Steady State, HEC-HMS Hydrologic Routing, or DSAT
Tier 2-Intermediate	Significant-hazard potential / intermediate-size or high-hazard dams with limited population at risk	HEC-HMS, HEC-RAS Unsteady Model, DSAT or WinDAM	HEC-RAS (Steady or Unsteady Modeling) or Two-Dimensional Model for unconfined floodplains
Tier 3-Advanced	High-hazard potential/ large-size dams with sufficient population at risk to justify advanced analyses	HEC-HMS, HEC-RAS Unsteady Model, or WinDAM	HEC-RAS Unsteady Model or Two-Dimensional Model

2.9. Hydraulic Model

2.9.1 The HEC-RAS model

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) is a simulation software developed by the US Army Corps of Engineers and has been developed to manage rivers and other public works under their jurisdiction. The HEC-RAS software has found wide acceptance among hydraulic engineers and researchers due to its robust channel flow analysis capabilities and its ability to simulates unsteady flood wave propagation and identifies flood prone areas as the areas where the ground is lower than the computed water elevation and allows the user to visualize the flood propagation in real time, thus making the software ideal for dam breach modeling.

The HEC-RAS system contains four one-dimensional river analysis components for: (1) steady flow water surface profile computations; (2) unsteady flow simulation; (3) movable boundary sediment transport computations; and (4) water quality analysis. A key element is that all four components use a common geometric data representation and common geometric and hydraulic computation routines (Brunner G. W., 2010).

The HEC-RAS system is comprised of a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, and graphing and reporting facilities. HEC-RAS is able to take into consideration hydraulic effects of bridges, culverts, weirs, and other structures in the river and floodplain on water surface calculations (Brunner G. W., 2010). The HEC-RAS modeling system was developed as part of the Hydrologic Engineering Centers Next

Generation software and replaces several existing Corps of Engineers programs, including the HEC-2 water surface profile program (ASCE, 2019).

The dam break tool in HEC-RAS can simulate the breach of an inline structure such as dam, or a lateral structure such as a levee. The latest versions of the HEC-RAS model include algorithms to model both overtopping and piping breaches (Brunner G. W., 2010). HEC-RAS uses hydraulic principles through cross sections upstream and downstream of the dam to define how the reservoir drains during the formation of a dam breach.

The dam crest is modeled as an inline weir and either a piping failure or overtopping failure is simulated with enlargement of the breach occurring over time as defined by a specified breach progression. Flow through the piping hole is calculated as orifice flow and flow through the breach is calculated as weir flow. The water surface profile upstream of the dam is back-calculated using unsteady momentum and hydraulic principles for each time step and the resulting drawdown through the hole and/or breach produces an outflow hydrograph. Resulting water levels for each time step downstream of the dam are used to model potential backwater effects and the weir and orifice coefficients are automatically adjusted for submergence, if necessary.

HEC-RAS can also model a piping failure that does not progress to the point of collapsing the crest. In this scenario, the piping hole is simulated as a sluice gate (Brunner G. W., 2010). The simplest case that can be used for testing the coupling of breach process models with HEC-RAS is that of a single reach bounded upstream by a dam with a reservoir described by a storage-elevation function, and the next refinement that can be simulated is the same system with dynamic routing being used for flow in the reservoir.

The HEC-RAS computational program has the ability to model extreme flow dynamics in the downstream of the reach due to a dam break flood waves and produce water surface profiles over the length of the modeled area (Asburry and Goodell, 2009). HEC-RAS is the most widely used hydraulic model for dam safety analyses in the United States and can be utilized for steady and unsteady flow analyses (State of Colorado, 2020). Moreover, the HEC-RAS outputs like water surface profiles can easily be converted to flood inundation maps by its companion program called HEC-GeoRAS which is an Arc-GIS extension, (Sredojevic and Simonovic, 2009).

Because of complexity of solving routines, the output solutions were thoroughly reviewed for stability and correctness. Furthermore, many studies indicate that the solution found by HEC-RAS is stable and trustworthy (Ackerman and Brunner, 2006). Hence, due to its extensive capabilities and freely availability in USACE website; its' outputs compatibility with Arc-GIS (HEC-

GeoRAS) software packages and for the said advantages, HEC-RAS model is selected by the Author to simulate the Kalid Dijo dam break.

2.9.2 HEC-GeoRAS

HEC-GeoRAS is an ArcGIS extension developed by the HEC. This model contains a set of tools specifically designed to process geospatial data to support hydraulic model development and analysis of water surface profile results. It assists in creating data sets in GIS to extract information essential for hydraulic modeling. After steady or unsteady flow simulation, HEC-RAS results can be exported for processing in the GIS by GeoRAS. The user can read the HEC-RAS results into the HEC-GeoRAS and perform the flood inundation mapping.

2.9.3 ArcGIS

The geographical information system, GIS is a system capable of capturing, storing, analyzing and displaying geographically referenced information. Hence, in our context, this model with its HEC-GeoRAS extension shall be used to study the areal distribution of the flood on downstream reach and to delineate the boundary of inundation.

2.9.4 The Breach model

BREACH: Fread (1988) developed the physically-based BREACH model to more realistically simulate breaches initiated by overtopping or piping. The model uses the Meyer-Peter and Muller sediment transport equation as modified by Smart (1984) for steep channels. The model permits specification of three different embankment materials: an inner core, an outer portion (downstream shell), and a vegetated cover or riprap protective layer on the downstream face of the dam. Flow through the breach section is determined by orifice or weir equations, and flow down the face of the dam is modeled as a quasi-steady uniform flow with roughness determined from the Strickler equation for Manning’s n. The model uses a much simpler computational algorithm than that of Ponce and Tsivoglou (1981). The model accounts for spillway flows around man-made dams and includes the effect of tail water depth on breach outflows. The model introduces two structural mechanisms that may contribute to breach formation: the breach shape may be impacted by slope stability of the breach side slopes, and possible collapse of the upper portion of the dam by shear and sliding is analyzed.

Today, the BREACH model is probably the best known physically based model. The most commonly applied breach prediction method in practice is probably the use of the uniform breach formation rate routines contained in DAMBRK and other models.

2.10 Modeling the propagation of flood

Having determined the breach parameters and peak outflow the next task is to route the most catastrophic outflow hydrograph over the downstream channel and flood plain and prepare an inundation map.

Inundation maps show the flood contour for natural floods of certain return periods, in most cases up to PMF, and flood contours of potential dam break floods caused by a "sunny day failure" and a failure superimposed over certain natural "base flood conditions". (ICOLD, 2012) These inundation maps are used to control building activities, develop necessary warning and evacuation plans and conduct consequence assessments.

Various models have been developed to execute the flood inundation mapping process. HECRAS is an example of one-dimensional models analyzing downstream flow using unsteady flow equations. The FESWMS (Finite Element Surface Water Modeling System) is a two-dimensional model developed by the Federal Emergency Management Agency (FEMA) to simulate movement of water in rivers, estuaries and coastal waters. While the 1-D models require cross sectional data of flow channels with sufficient extension on both left and right direction, the 2-D models use topographical data in the form of a continuous surface represented by finite element mesh. Case studies comparing 1D and 2D could show that the 2D models represent more accurate flow in the flood plain due to both lateral and longitudinal movement of water (Cook, 2008).

In spite of the timely improvements of flood routing models, the predictive accuracy of such models can be subject to significant error (2 feet or more in the crest profile) due to inaccuracy in the computed reservoir inflow, breach parameter estimation and dynamics, downstream cross section properties, flow resistance coefficients both in channels and flood plains, effect of sediment transport and flow constrictions in channel and the complex flow patterns that are not adequately described by one-dimensional flow equations (Fread, 1981).

To correctly estimate the consequences derived from a structural failure modeling of flood propagation should be highly accurate. Identification of the inundated areas, inundation depth, speed and duration, as well as the impact that flood water characteristics (salt, freshwater, contaminated water, etc.) can have on the inundated areas, are very important for decision making, emergency evacuation and early warning. The De Saint-Venant equations or shallow water equations are used for modeling dam break flood propagation. These equations consist of the mass and momentum conservation equations, and assume that the vertical velocities are much smaller than the horizontal velocities, which leads to hydrostatic pressure distribution in a channel cross section (Zagonjolli, 2007).

3. Materials and Methods

3.1. Description of Study Area

3.1.1. Location of the Project

Kalid-Dijo Dam project area is found in Southern Nation Nationalities and Peoples Region, Silite Zone, near to Worabe Town, specifically located between Bureka & Germma Kebeles. Basin wise, the project is located in Rift Valley Basin with geographic coordinates of 38^o 12” 46’ E longitude and 7^o 51”12’ N latitude. Moreover, it is located at a distance of 170 km from the capital of Ethiopia, Addis Ababa and 3km from the Zonal town of Worabe in the North – East direction. The gross command area of the proposed Kalid-Dijo Dam and Irrigation Project is about 1100 ha of gross area. The catchment area and mean annual flow of the dam site is 141.9km^2 1.44m^3/sec respectively.

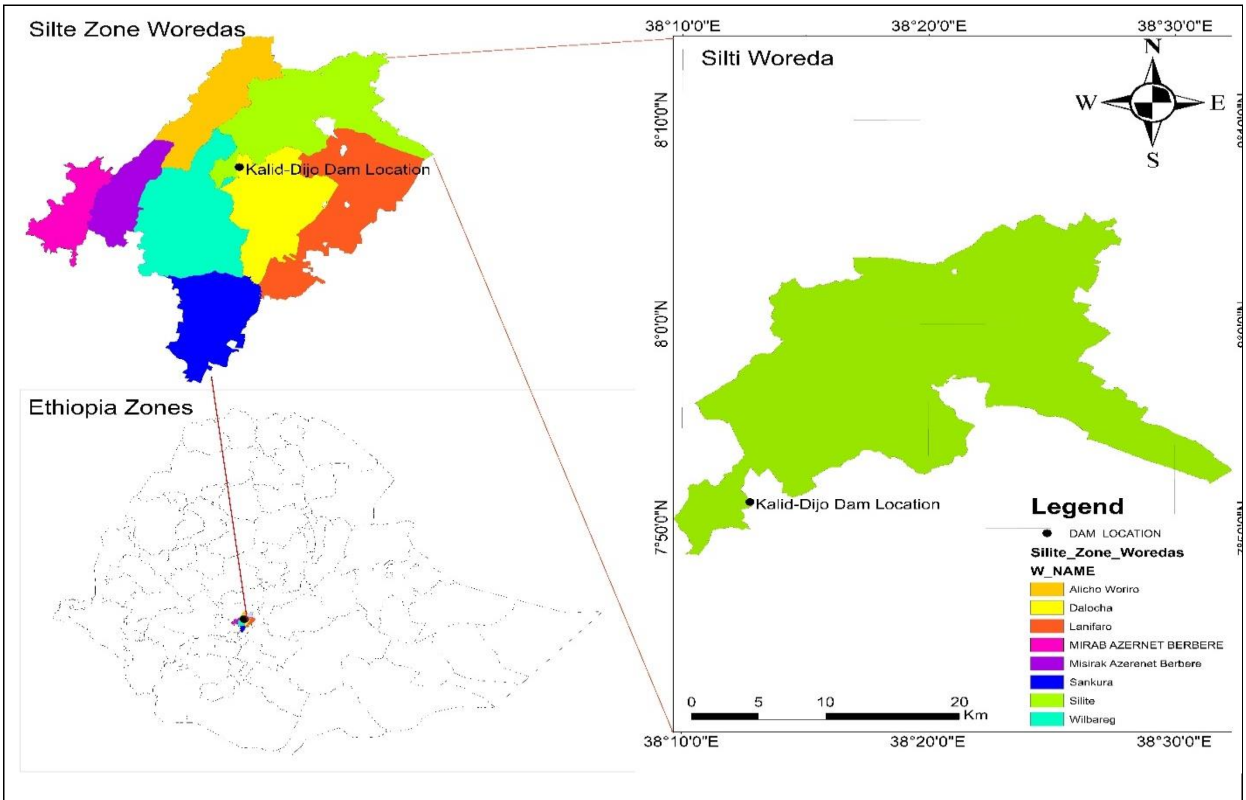

Figure 3. 1 Location Map of Study Area

3.1.2. Climate

The mean annual rainfall of the catchment based on three weighted metrological station data around Dijo watershed, that are Butajira, Wulbarag and Tora metrological stations. The mean annual rainfall of the study area ranges from 1010.74 mm to 1131.37 mm. Where, the rainfall distribution in the watershed is characterized bimodal type of rainfall, the first season rainfall occurs between April to June and the second rainfall occur between July to September having 132.2mm and 144.9 mm pick rainfall in the first and second rainfall season respectively. Regarding its temperature, the maximum mean monthly temperature in the watershed is 29.8^0C which occurs in February and April and the minimum temperature is 6.1^0C occurs in the month of December. In general, Dijo Woreda in particular is one of sub-tropical/ Weyina Dega zone in Ethiopia, well maintained steeply landscape and has two and longer rainy seasons for agriculture and other water demands. High climate variability, rainfall unpredictability and low agricultural productivity and high population density has led to poverty, malnutrition and food security problem in the area.

3.1.3. Hydrology

Hydrological location of the site is in north-western direction of Rift valley. The watershed is raised from the highlands of Alicho Woriro Woreda known as Limaza River, which is a tributary of Dijo River. Dijido River flows to the lake Shalla sub-basin and finally joins the rift valley Basin emerging from Alicho Woriro Woreda Highlands. The project site is selected on the Limaza River at coordinates of 413,783.19m Easting and 865,301.85m Northing, which is the seasonal tributary of Dijo River. The Dijo River has a flow of water for about 10-11 months per year.

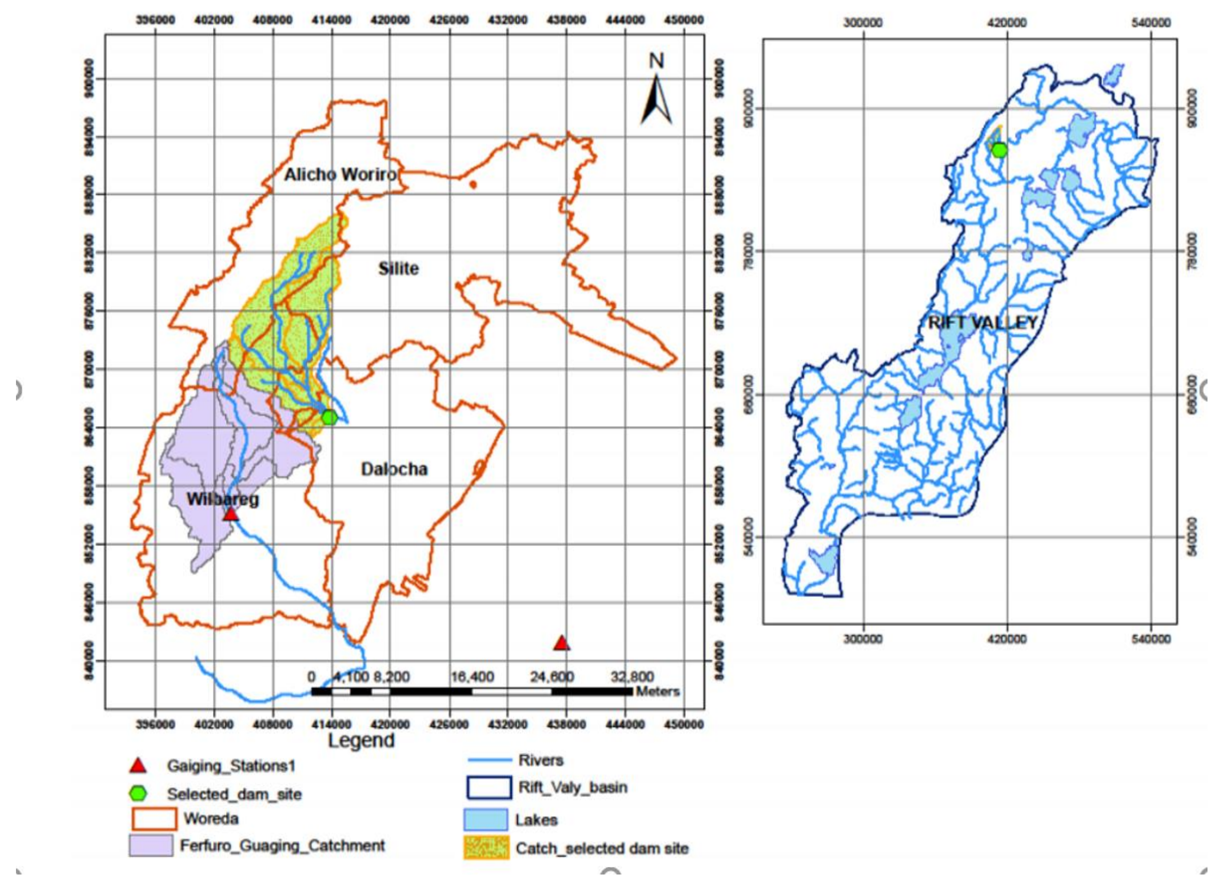

Figure 3. 2 Catchment area of the project in rift valley (ECDSWC, 2020)

3.1.4. Watershed

The watershed is located within SNNP regional state specifically in the Selte Zone. The watershed covers a total area of 14,202.2 ha; geographically it is located between 864000- 888000 UTM Northing and 404000- 416000 UTM Easting. It is about 170 km from Addis Ababa through Worabe town. The watershed is situated at an altitude of about 1945 – 3179 meters above sea level. It is to be found in the Rift Valley basin and embodies four woredas and fourteen kebele's named as; Dalocha woreda (Burqa-Dipo and Garmama Gale kebeles), Worabe town (town kebeles 1, 2, and 3, Albazer Zemu Shidges and Date Wazir-6 kebeles), Alicho wariro woreda (Date Wazir-1, Date Wazir-2, Kechemo, Edonaletazer, Gone, and Qewagoto kebeles), and Siliti woreda (Ansilobeso kebele).

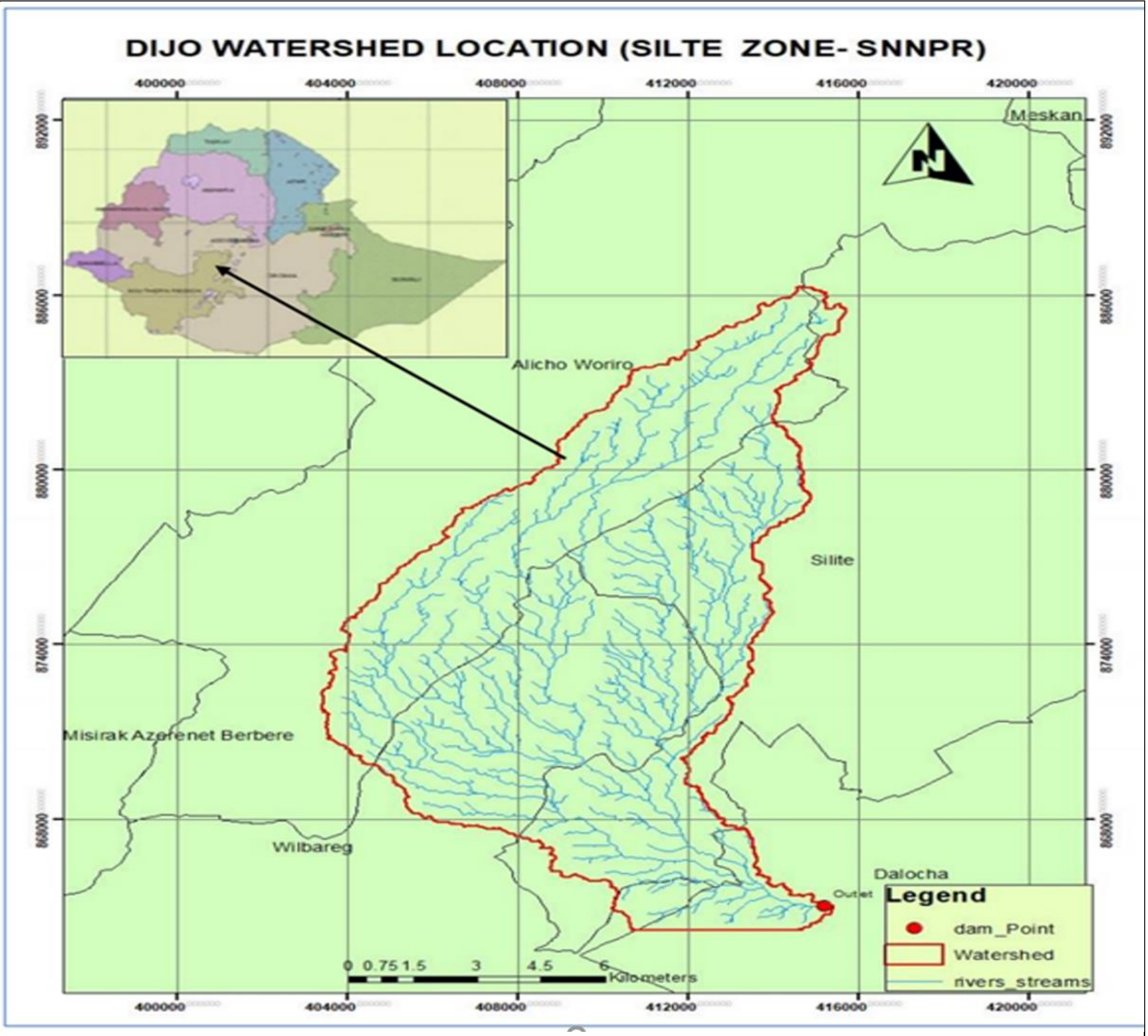

Figure 3. 3 Location map of Dijo Watershed (ECDSWC, 2020).

3.1.4.1. Soil of The Watershed

There are four major soil types found in the watershed, namely: Vertisols, Nitosols and Luvisols, and Lithosols. These soil types are found in different landform within the watershed. They occur with two horizons which are Dystric and Eutric for the former and Eutic and Pelic for the later. Vertisols, occur over 76 percent, while Nitosols and Luvisols, and Lithosols cover 16 percent, 5.52 percent, and 1.52 percent of the watershed, respectively. Luvisols, (with the exception of Letic, Gleyic, Vitric, Albic, Ferric and Dystric soil units) explained by FAO (2001) are fertile soils and suitable for a wide range of agricultural uses; and require erosion control measures if occur on steep slopes.

Table 3. 1 Soils of Dijo Watershed

Soil Type	**Area(ha)**	**%**
Lithosols	216.00	1.52
Luvisols	783.30	5.52
Nitosols	2,272.04	16.00
Vertisols	10,930.82	76.97
Total	**14,202.16**	**100**

Source: ECDSWC Main Report 2020

3.1.5. Regional Geology

The project area lies on western escarpment of Main Ethiopian Rift (MER), particularly in its central segment. Thus, discussing the regional geological set up of the MER and particularly the central Segment (CMER, Central Main Ethiopian Rift) is worthy in order to grab clues about the project site.

The Central MER (where Kalid Dijo Dam Site is located) is bounded by the Yerer-Tullu-Wellel volcano-tectonic lineament (YTVL) to the north and the Goba-Bonga lineament to the south. It is also bounded to the east and west by fault escarpments (some of them with offset more than 1500m) such as the Munesa and Guraghe rift margins. With the formation of the Wonji Fault Belt, tectonic movements and volcanic activity produced step-like structures and associated volcanic activity, represented by ignimbrites, basalts and unwelded pyroclastics. According to Abebe et al, 2005, the regional geology of the study area is composed of:

- Pre-Tertiary sediments and crystalline basement,
- Oligocene (32-29 Ma) & lower Miocene (12-8) plateau volcanics,
- Miocene-Pliocene rift-shoulder trachytic-rhyolitic volcanics & pyroclastics layers,
- Plio-Pleistocene rift floor,
- Quaternary central volcanics & basaltic lava flows, associated scoria cones and phreato-magmatic deposits, and
- Quaternary lacustrine sediments & interbedded pyroclastics,

In general, with the formation of the Wonji Fault Belt, tectonic movements and volcanic activity step-like structures and associated volcanic activity, represented by ignimbrites, basalts and unwelded pyroclastics are produced in the area (Mohr, 1967).

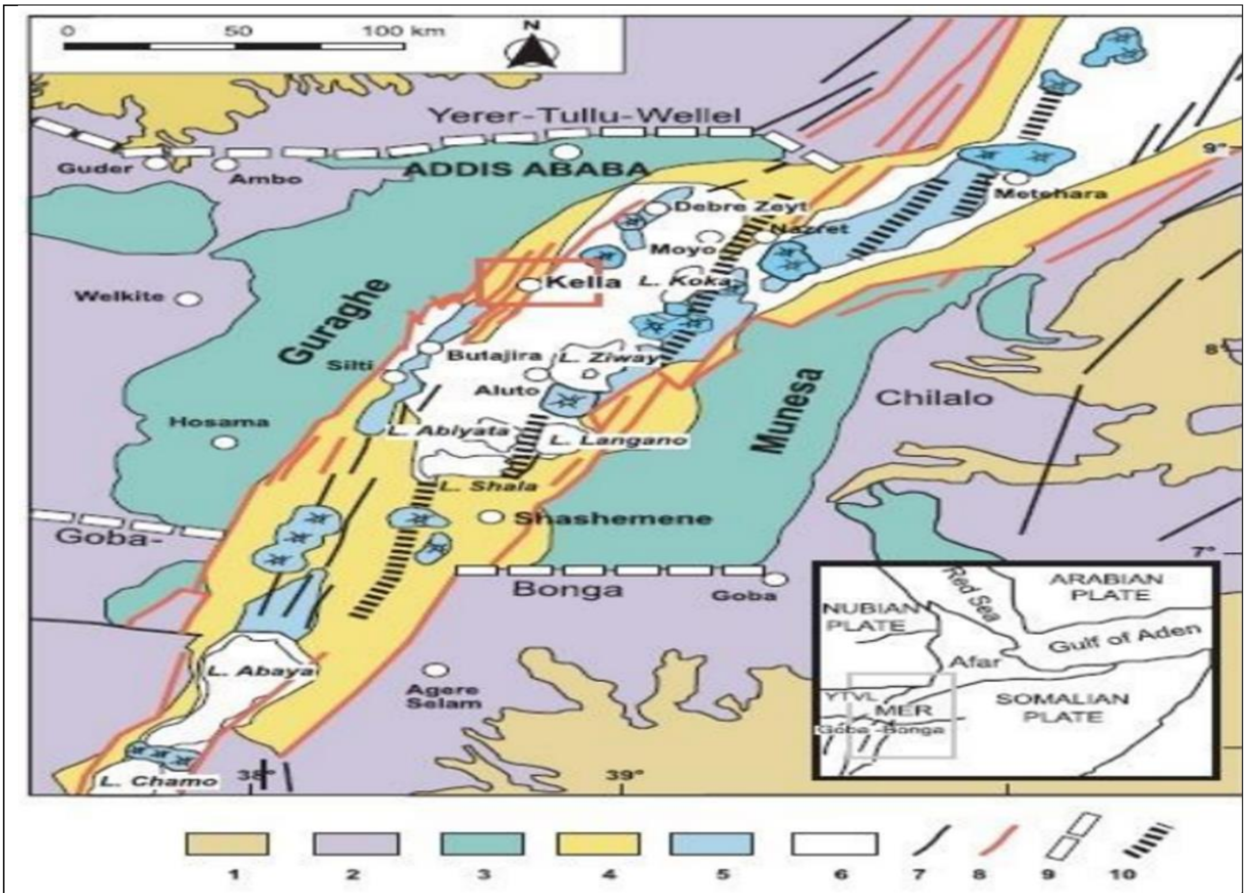

Figure 3. 4 Simplified Geological Map of Central Ethiopia, (Tsegaye Abebe, Balestrieri, M.L. and Bigazzi, G., 2010)

(1) Pre-Tertiary sediments and crystalline basement, (2) Oligocene (32-29 Ma) & lower Miocene (12-8) plateau volcanics, (3) Miocene-Pliocene rift-shoulder trachytic-rhyolitic volcanics & pyroclastics layers, (4) Plio-Pleistocene rift floor, (5) Quaternary central volcanics & basaltic lava flows, associated scoria cones and phreato-magmatic deposits, (6) Quaternary lacustrine sediments & interbedded pyroclastics, (7) faults, (8) major rift boarder faults, (9) major transversal tectonic lineaments in the basement, (10) Wonji Fault Belt segments. YTVL=Yerer-Tullu-Wellel volcano-tectonic lineament.

3.1.5.1. Dam Site Geology

Field mapping at the dam axis option sites at a scale of 1:2000 outlined major stratigraphic successions of pyroclastic deposits (unwelded Tuff, welded Tuff), Ignimbrite flows and volcanic ash falls. In the Dijo dam site area, the following lithologic units are identified during the current investigation at captioned scale and their detail is presented below:

- Volcanic Ash

- Unwelded Tuff
- Welded Tuff

A) Left Abutments

The left abutment is formed by volcanic ash which in turn is covered by thick black cotton residual soil; this unit covers unconsolidated pyroclastic deposits (tuff). The top part is relatively thick (1-2m) black cotton residual soil. Below this residual soil, 2-3m thick volcanic ash layers made up of brownish to dark grayish colored fragments of pulverized fine to medium grained rock and volcanic glasses can be traced in vertical sections of streams.

B) Channel area

Major portion of the channel of Dijo River is narrow and is outcropped by welded tuff. At the crossing site of the dam axis, 13m width of the channel is covered by river channel deposits comprising of pebble to cobble size rocks of various composition. Projections/interpretation favors welded tuff at this river bed profile.

C) Right Abutment

Except the variation in coverage, the right abutment of the dam axis is outcropped by lithologies described at the left abutment. Traversing to west from Dijo River channel, relatively more consolidated welded tuff unit covers the remaining portion of dam axis. Similar to the left abutment, this unit is weak, soft, fractured by systematic tectonic joints and thus is considered as soft rock. Compared with unwelded tuff, this unit looks more competent and formed steep cliffs exposing fresh rocks at Dijo River banks. The E-W running tight tectonic joints transverse the dam axis and the NE-SW trending joints parallel the dam axis.

Figure 3. 5 Lithostratigraphy of Dijo River Channel and Banks, Upstream of dam axis photo looking north (ECDSWC, 2020).

Table 3. 2 Summarized geological units at Dijo River dam axis profile

	Dijo River Dam Axis Option
Left Abutment	Ash/Soil
	Unwelded Tuff
	Welded Tuff
Channel Area (Presumable)	River Channel Deposit
	Welded Tuff
Right Abutment	Welded Tuff
	Unwelded Tuff
	Ash/Soil

3.1.6. Land Use

According to the data from respective Kebele offices, about 53 percent of the Woreda's total area is taken up by annual crops followed by infrastructure for social services. Forests and communal grazing lands take up less than 15 percent of the remaining lands other nonagricultural land, and bodies of water account for the rest. There has been an increase in the area of land cultivated and a consequent decrease in the area of land open for grazing associated with high population pressure from time to time. This has been accompanied by a move from communally to individually held land except burial areas. People have household level grazing lands around homesteads to keep their livestock. Dalocha command area is covered dominantly by cultivated land (1478.2 ha) with some grass /fallow lands. There is no and free grazing in the target area related with intensive farming.

Table 3. 3 Land use and area of the targeted project Kebeles (Source: ECDSWC Main Report 2020)

S/No	Kebele	Cultivated Land	Grazing Land	Social Services (Schools, Burial Places, Mosques, etc.)	Total Kebele Area
1	Chimo Qoro	1,076.8	20.16	728.16	1,825.12
2	Germama Gale	986.6	22.75	720.1	1,729.45
3	Gete Kuiyo	1,088	62.7	496.3	1,647
4	Gude Godebamo	819.9	77.8	696.1	15,93.8
5	Dalocha Telikasa	807.65	980	476.35	2,264
6	Burka Dilapa	882.01	482.79	598.2	1,963
	Total target Kebele area in hectares				**11,022.37**

3.2. Silent Feature of The Dam

The principal components of the works are:

- An earth-rock fill dam 1828.25m long, 28 m high above the riverbed level with crest width 8m at (El. 1978m a.s.l); it has the fill volume of (about 1,400,000 m3)
- The storage capacity of the reservoir is certainly remarkable. The gross storage capacity of the dam is 8M m3 and dead storage is 1.36M m3.
- Ogee side channel spillway, design discharge 127.8m3/s, 30m crest length, crest elevation at 1974.5m a.s.l.

- Downstream control inlet and outlet, with conduit diameter 1m, conduit capacity 1.8m3/sec, design discharge 1.5m3/sec and downstream release of water 0.3m3/sec .

Source: Ethiopian Construction Design and Supervision Works Corporation, Main Report 2020

3.3. Data Collection and Analysis

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

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

Dam characteristics: This category includes data about name of dam, dam type, dam size, location of the dam, elevation of downstream toe of dam, design water storage pool elevation, maximum flood surcharge elevation, spillway crest elevation, crest of dam elevation, and height of the dam measured from downstream toe to the crest, and category of the dam.

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

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

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

3.4. Sensitivity Analysis Steps

The analysis of dam breach involves using different computer application software. Depending on the level of study and extent of expected accuracy, modelers can select appropriate type of modeling software. In this study, HEC-RAS 5.0.6 -2D have been used to perform the dam break and sensitivity analysis at the dam site and its downstream reaches.

Dam-break analysis assumed in this study causes severe breaching outflow hydrograph at the dam site, this study also incorporated the breaching outflow hydrograph routing through its downstream channel. In order to meet this scope of study, the Kalid Dijo Dam and its downstream river. A sensitivity analysis flow chart using the Kalid Dijo Dam and its downstream river is presented in Figure 3-3. The flow diagram describes and presents the overall modeling approaches and assumed scenarios. Also, it showed the matrices used to analyze the relative importance of one or more of the parameters which are thought to have significant influences on the dam-break phenomenon.

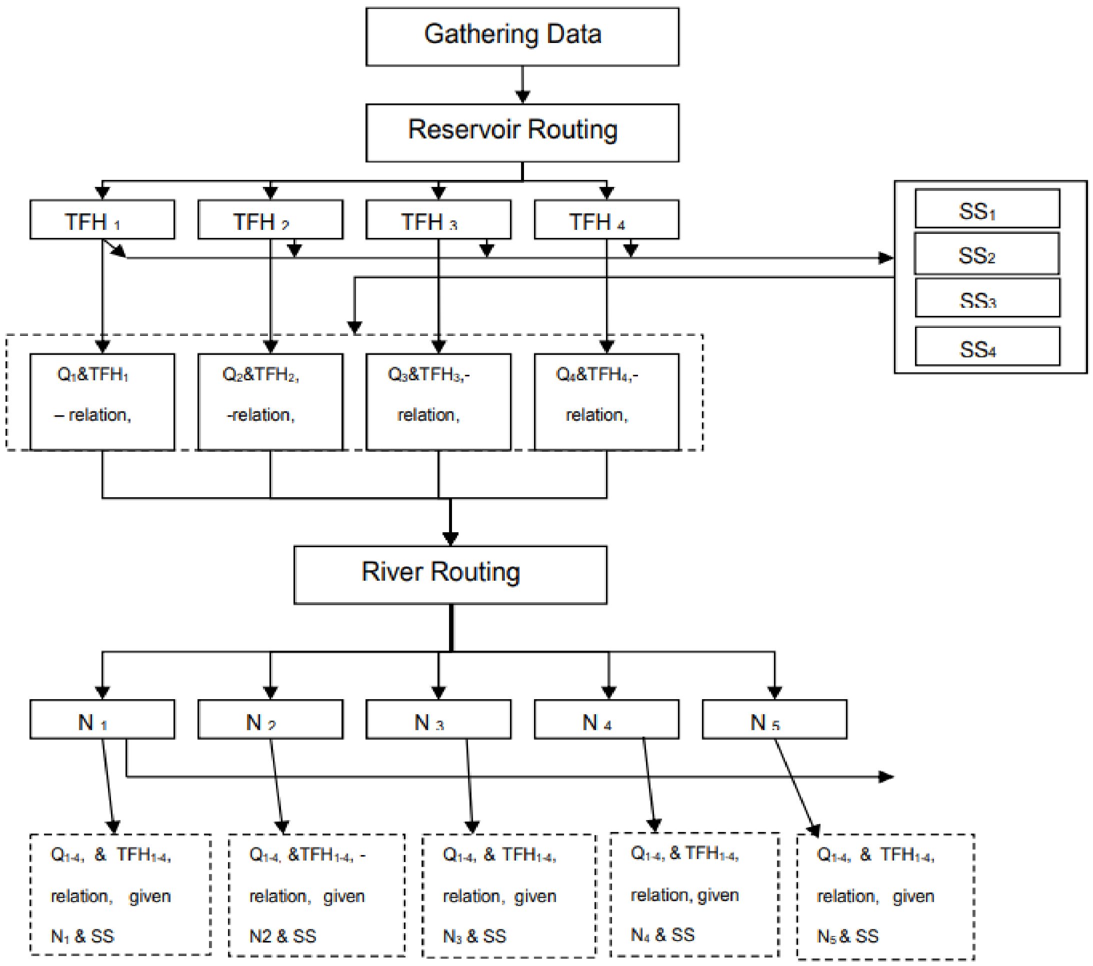

Figure 3. 6 The flow diagram describes and presents the overall modeling approaches and assumed scenarios.

Notes:

Four different hydrographs produced based on a given relation, i.e., outflow hydrograph (Q1, Q2, Q3 and Q4)

Four different Time to Dam Failure Hours used (TFH 1, TFH2, TFH3, TFH4)

Four different breach side slope values used (SS1, SS2, SS3 and SS4)

Five different manning coefficient values used (N1, N2, N3, N4 and N5)

3.4.1. Breach Outflow Hydrograph Modeling

As shown in Figure 3.6, HEC-RAS 5.0.6 computer program was used for the sensitivity analysis for Kalid Dijo Dam. The model assumes a single inflow to the reservoir of the Kalid Dijo Dam in addition to defining the physical characteristics of the dam and reservoir. As indicated on the flow diagram, four different scenarios of time to dam failure in hours (TFH) are assumed to evaluate the sensitivity of outflow hydrographs to these changes.

Additionally, four side slopes of dam breach are assumed to examine their responses of peak outflows at dam for the aforementioned changes. The combined effects of side slope and time to dam failure in hours' changes on peak outflows at the dam are also evaluated using HEC-RAS modeling software. Moreover, the outputs of HEC-RAS modeling, combinations of the four time to dam failure in hours' scenarios and a single side slope of breach resulted in four dam-break outflows hydrographs, are used as an input hydrograph for studying the relationships among river flow parameters.

3.4.2. River Modeling

As shown in the second half of the flow chart, the HEC-RAS hydraulic modeling software execute unsteady flow river routing of dam breach hydrographs through the downstream channel. The four dam-break outflow hydrographs generated were the prime inputs in this modeling. In HEC-RAS modeling, each routing hydrographs are varied with five different Manning's n values and resulted in twenty computer runs. The outputs are analyzed to evaluate sensitivity of downstream peak flows to change in Manning's n values for a given TFH.

3.5. Dam Breach Analysis Procedures

The parameters of dam breach depend on type of the dam and mode of failure. The shape and duration of the breach, together with the size of the dam and the reservoir, would determine, to a great extent, the characteristics of the breach outflow hydrographs. Overtopping mode of failure is assumed in this study because of the following points;

- Most of dams have failed due to this mode >70% of failures around worldwide (NPDM 2011)

- Kalid Dijo dam is, zoned rock fill with central clay core and composite dam (embankment with concrete structure overflow section). Therefore, low or no seepage is expected
- In the design seepage control measures such as foundation cutoffs, grouting, adequate and non-brittle impervious zones, transition zones, and drainage blankets are incorporated.

Hydraulic analysis of dam breach includes two primary tasks, the prediction of the reservoir outflow hydrograph and the routing of that hydrograph through the downstream valley.

3.5.1. Predicting the Outflow Hydrographs

For flood hydrograph estimation, the breach modeled by defining acceptable dam breach parameters is the core of this project. Predicting the outflow hydrographs at the dam location is done using HEC-RAS 5.0.6 under different scenarios. The process of predicting the outflow hydrograph is a multi-steps approach and began with defining the 2D river geometry, the reservoir characteristics, physical description of Kalid Dijo Dam, and its detailing breach characteristics.

3.5.1.1. Defining the River Geometry

When using HEC-RAS the first task is creating a project with 12.5m DEM file which is available on https://vertex.daac.asf.alaska.edu/#, and by changing all the units into metric system and inserting the projection file of the project area. The next step after creating the project is to create the geometry of the river. This can be done by RAS Mapper, which is GIS tool of HEC-RAS. The geometry of the Dijo River generated on RAS Mapper is shown in the figure below.

3.5.1.2. Identifying Physical Descriptions of Dam

This step includes the identification of dam height, dam crest width, spillway elevation and width and coefficient of discharge. In this study, data about the physical characterizes of the Kalid Dijo dam is imported from the dam drawings are used as an input in HEC-RAS modeling.

- Dam crest elevation =1978.1m
- Spill way elevation=1974.5m
- Mid-Level Outlet Elevation=1963.5m

Table 3. 4 Kalid Dijo Dam Relevant Reservoir Volumes

Mid-Level Outlet	EL.1963.5	3.52Mm^3
Minimum Operating Water Level	EL.1963	1.95Mm^3
Full Reservoir Level	EL. 1973	7.87M m3
Maximum Water Level	EL.1975.1	9.04Mm^3
Dam Crest	EL.1978.1	10.8Mm^3

Source: ECDSWC   Main Report (2020)

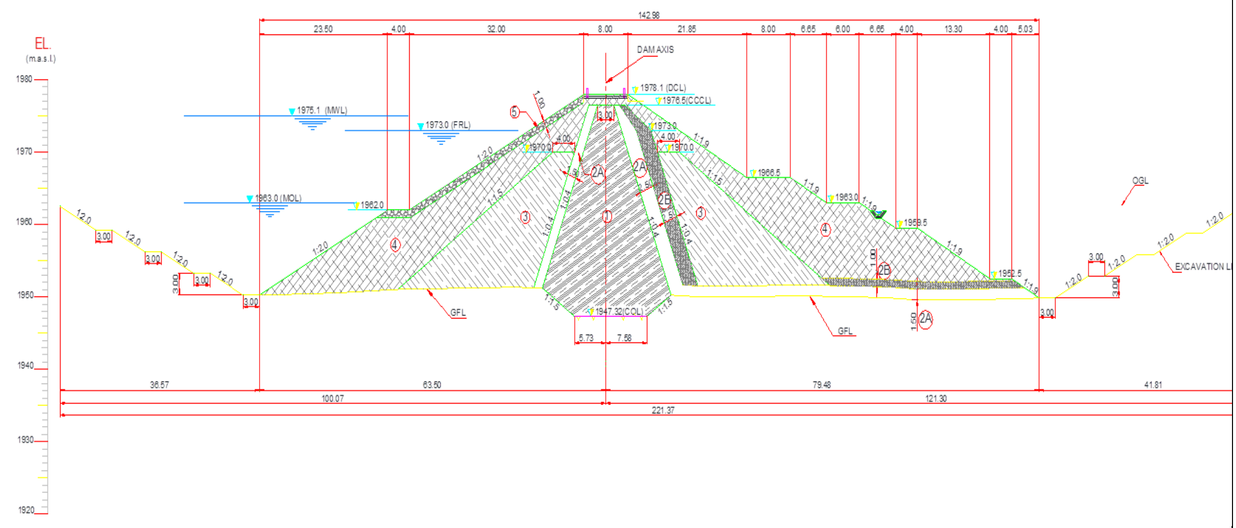

Figure 3. 7 Kalid Dijo Dam (ECDSWC, 2020).

5.1.3. Determining Inflow Hydrograph to the Reservoir

The inflow design flood is expected to cause the dam to breach in order to analyze the worst case of dam breach analysis. Since the dam is categorized as large dam and there are communities, irrigation infrastructures and command areas at the downstream of the dam, the inflow hydrographs generated from PMF was used for the breach analysis.

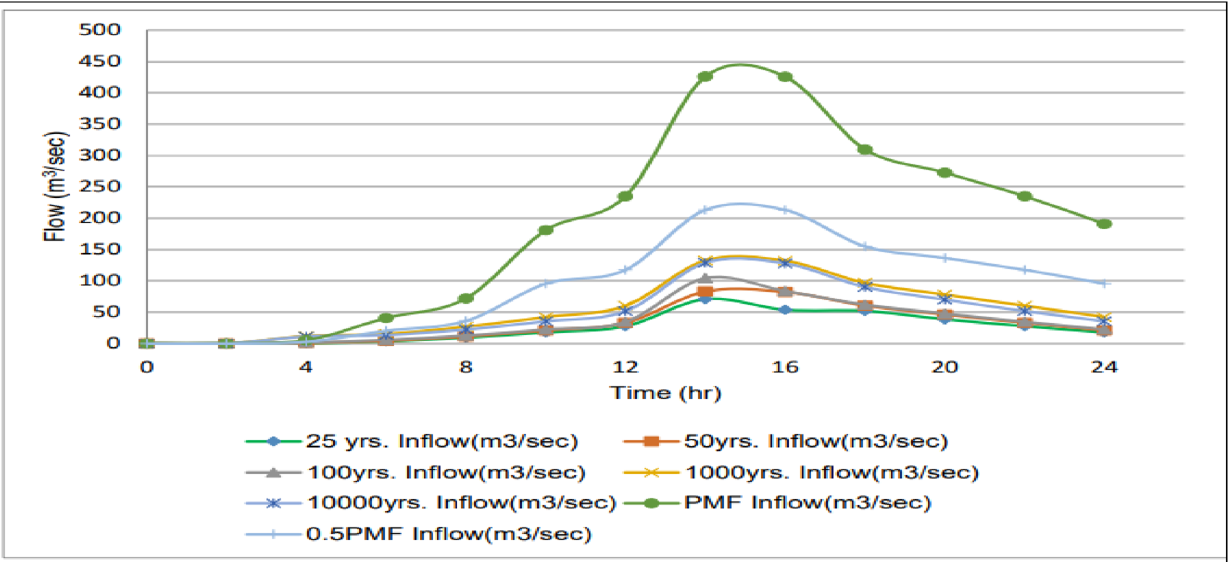

Figure 3. 8 Kalid-Dijo Dam and Irrigation Project Hydrograph for different return periods (ECDSWC, 2020)

3.5.1.4. Estimating Dam Breach Characteristics

The dam breach core characteristics are:

a. Shape of the Breach: Since Kalid Dijo Dam is an earth-rock fill dam, a trapezoidal shape of breach is assumed in this study.
b. Breach Bottom Elevation (H_b): By assuming the possible worst case, the breach bottom elevation is set to be equal to 1950m amsl.
c. Average Breach Bottom Width ($\overline{B}$): In this modeling, the breach bottom width is estimated based on empirical formula provided by Froehlich 2008.

$$\overline{B} = 0.27 K_O V_W^{0.32} h_b^{0.04} \qquad \text{............................ (0-1)}$$

Where: - $\overline{B}$= average of width of final breach

$$K_O = \begin{cases} 1.3\,OT \\ 1.0\,P \end{cases}$$ .... (3-2)

OT = Over Topping

P = Pipinig

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

h_b=height of the breach (=1978.1-1950=28.1)

Where: dam crest=1978.1

Minimum WL= 1950

By inserting the above values in the empirical formulas,

$B_{ave} = 0.27 * 1.3 * 10{,}800{,}000^{0.32} * 28.1^{0.04}$=71.44m

d. Side Slope of Breach (SS): the slope parameter Z or SS identifies the side slope of the breach, i.e., 1 vertical: Z horizontal. Based on Froehlich 2008 the value of SS is taken as 1:1 for embankment dams. A trapezoidal shape was specified with various combinations of Z values. The four Z values, 0.125, 0.25, 0.5 and 1 are used to simulate reservoir routing and examining the relationships between peak breaching outflows and change in Z values at the dam site. For multi-variables or parameters of dam breach sensitivity analysis, the analysis flow chart was presented in Figure 3-6.

e. Time to Failure in Hours (TFH): Based up on Froehlich 2008:

$$T_f = 60\,x \left(\frac{V_w}{g h_b^2} \right)^{0.5}$$ .... (3-3)

Where: T_f=Breach formation time (seconds)

V_w=Reservoir Volume at time of failure (cubic meter)

h_b=Height of the final breach at failure (meter)

$T_f = 60 * (\frac{10{,}800{,}000}{9.81*28.12})^{0.5} = \left(\frac{2240.38\ sec}{3600\ sec}\right) = 0.62hr$

In this study, four different TFH are used to analyze the sensitivity of peak breaching outflows at the dam site due to change in TFH.

Additionally, these four TFH are simulated for each Z values and resulted in sixteen different peak breaching outflows. The relationship among these three parameters, peak flow, TFH and Z are further analyzed to compare the result with existing condition.

3.5.1.5. Breach Outflow and Verification

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

First, using Froehlich 2008 method the maximum breach outflow should be calculated because the breach parameters are estimated using this method. According to this method,

$$Q_p = 3.1 * Bave.* hw^{1.5} * \left(\frac{\Upsilon}{\Upsilon + T_{f} * hw^{0.5}}\right)^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(3-4)}$$

Where: γ =instantaneous flow reduction factor $= \frac{23.4\, A_s}{B_{ave}}$

$$\Upsilon = \left(\frac{23.4 As}{B_{ave.}}\right) = \frac{23.4 * 129}{71.44} = 42$$

A=Surface Area of the reservoir corresponding to h_w ($sq.km$).

A_s= 129

$$Q_p = 3.1 * Bave.* hw^{1.5} * \left(\frac{\Upsilon}{\Upsilon + T_{f} * hw^{0.5}}\right)^3$$

$$Q_p = 3.1 * 71.44 * 28.1^{1.5} * \left(\frac{42}{42 + 0.62 * 28.1^{0.5}}\right)^3 = 865.95 \text{m3/sec}$$

For verification of the reasonableness of the value of breach out flow obtained by the analysis we have to compare it with the value obtained by empirical formula as shown above or with the envelope. But the envelope as discussed in the literature review will not be the true upper bound because it only taken in to account fourteen historical dam failure incidents. Therefore, the value obtained using the empirical relationship suggested by Froehlich 2008 as show above will be used as an upper peak breach outflow.

3.5.2. Routing Breach Outflow Hydrographs through Downstream Reaches

Dam-break flood hydrograph is a dynamic and unsteady phenomenon. Therefore, the preferred approach is to utilize a fully developed Unsteady State flow routing model. In order to accurately model the flows, the unsteady flow computer program, HEC-RAS 5.0.6 was used to route breaching outflow hydrographs through natural waterways.

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

3.5.2.1. Defining Channel Geometry and Boundary Conditions

During modeling of the downstream channel of the Kalid Dijo Dam using HEC-RAS, the first step is to establish the external boundary conditions. The upstream boundary is selected at a location such that it is independent of the downstream conditions. The downstream boundary was selected at a location that is independent of flow conditions below the boundary. The last downstream cross section is set at a reasonable distance and a normal depth is chosen to define the downstream boundary conditions. After the routing reach is established by the boundary locations, cross sections are obtained to represent the reaches.

3.5.2.2. Selecting Manning Coefficients, “n” values

Manning’s coefficient n is used to describe the resistance to flow due to channel roughness caused by sand/gravel bed-forms, bank vegetation and obstructions, bend effects, and circulation-eddy losses and so on.

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

By referring the Dijo River bed and bank materials), Manning n values 0.03 was taken. In this study five “n” values are used to study the relationships among peak flow, time to failure (TFH), and Manning n at two specified locations along the downstream river.

3.5.2.3. Dam-break Outflow Hydrographs

Four different outflow hydrographs derived from four TFH during HEC-RAS analysis are used as inflow hydrographs for the downstream river routing. As shown on flow chart Figure 3-3, each breaching outflow hydrographs was used in conjunction with five different Manning (n) values to show the change in the breach outflow in different stations and as a result, twenty independent computer runs have been conducted.

3.6. Flood Hydrograph Routing

Flood routing can predict the temporal and spatial variations of a flood wave through a river reach and/or reservoir. This sensitivity analysis consisted of reservoir and downstream channel. The hydrologic routing and hydraulic routing are used to conduct flood routing through the reservoir and downstream channel.

3.6.1. Hydrologic Routing

The hydrologic routing involved the balancing of inflow, outflow, and storage-discharge relation through use of the continuity equation. This application of hydrologic routing is used for reservoir routing. The reservoir component initiated by receiving upstream inflows and routed these inflows through a reservoir using Storage Routing Methods.

The PMF inflow hydrograph to the reservoir is taken as an inflow design flood in order to carry out the analysis. In addition to IDF, the shape of breach, average width of breach, side slope of breach, breaching time, and other dam breach parameters were identified and predefined for generating the breaching outflow hydrographs by HEC-RAS 5.0.6 computer software.

3.6.2. Hydraulic Routing

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

In order to carry out the hydraulic routing through the Dijo, the downstream cross sections data is processed in RAS MAPPER that included all 2D geometries, Manning coefficients, reach lengths and boundary conditions. The boundary conditions included all of the external boundaries of the system, as well as the internal locations and set the initial flow and storage area conditions at the beginning of the simulation. Since the Dijo downstream channel is modeled as an open-ended reach, the downstream boundary condition is set as a normal depth. As recommended in HEC-RAS user manual for this option of boundary condition, the last cross section is placed far enough such that any errors it produced would not affect the results at the study reach.

4. Result and Discussion

4.1. Probable Maximum Flood Data Validation

In this study, the Gumbel frequency distribution method was applied to validate the design PMF that provided by the dam owner. Accordingly, the PMF of the design is approximately 450 m3/sec; however, the verified PMF is approximately 428 m3/sec which is in the range of 5%. Therefore; the design flow is accepted and adopted for further analysis.

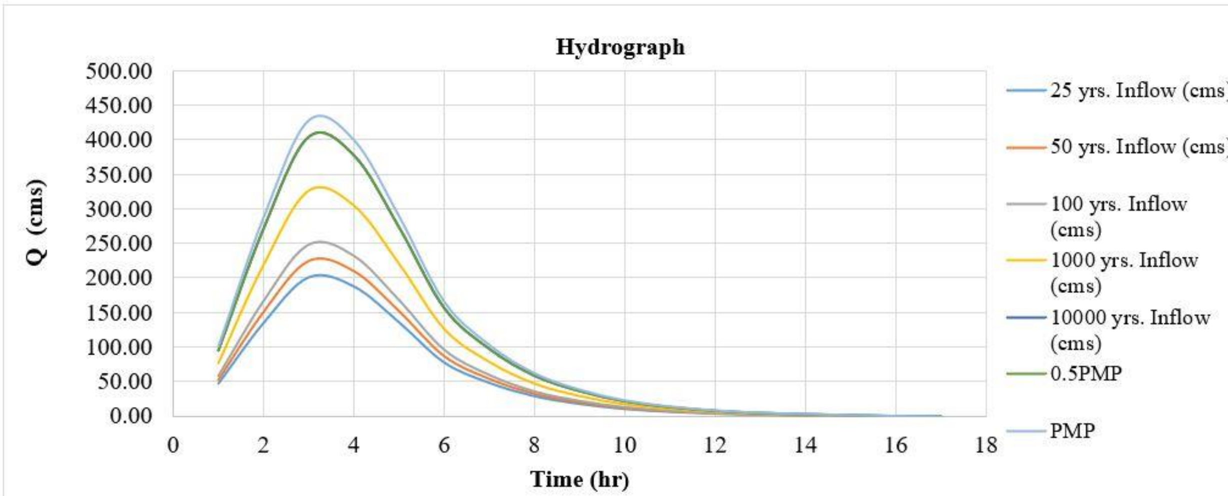

Figure 4. 1 Kalid_Dijo Dam Hydrograph for different return periods

4.2. Model output at Dam location

The dam break and sensitivity analysis of Kalid Dijo Dam involved testing a number of dam breach parameters. The findings were discussed in the following sub-sections.

4.2.1. Dam Breach Outflow Hydrograph

Maximum breach discharge (Q_{max}), breach development time in hours (TFH), and side slope of breach (SS or 1: Z) were the three principal parameters analyzed at the dam site in the first half of this study, reservoir component.

The dam breach outflow hydrograph for the Kalid Dijo embankment dam resulted from the HEC-RAS dam break modeling is shown in the figure 4-2. The peak breach outflow from the hydrograph is 852.18m3/sec which is less than the upper bound of breach outflow value (865.95m3/sec) that were calculated by using Froehlich 2008 equation. This **validates** the outflow hydrograph obtained from the model.

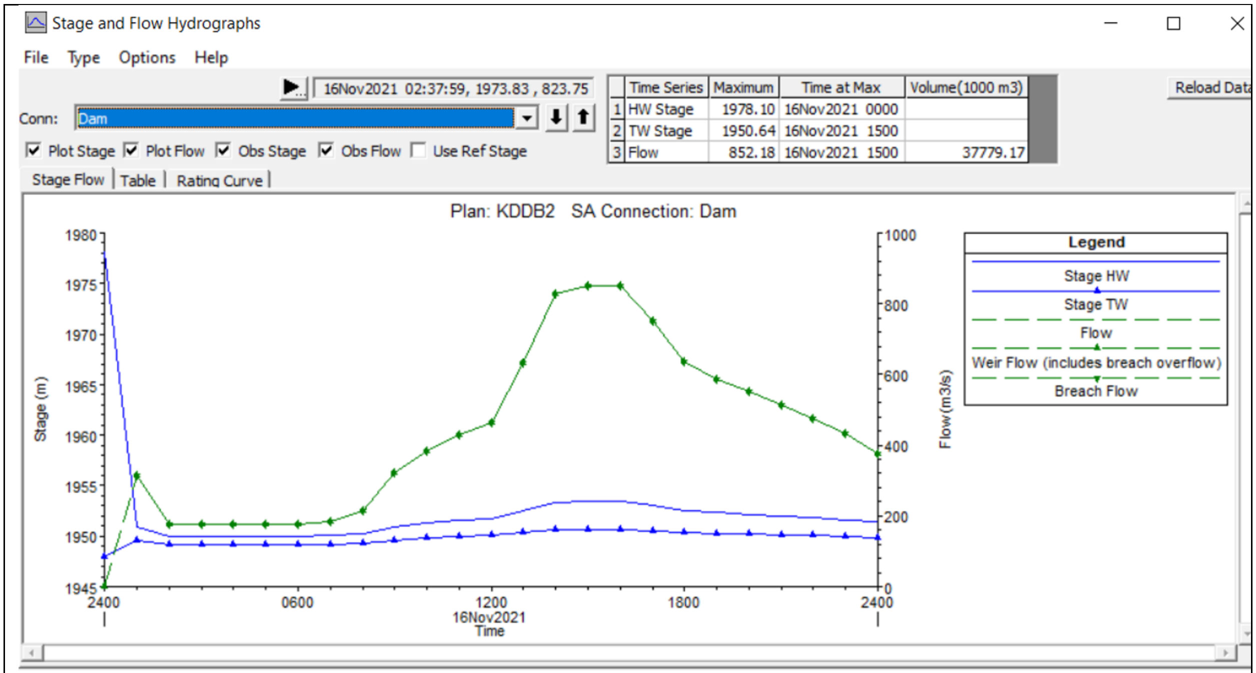

Figure 4. 2 Breach Outflow Hydrograph at the Dam

In addition, with the peak flow the model output was also further checked by envelope curve (hydraulic depth Vs breach outflow discharge) of historic failures as shown in figure 4-3.

Hydraulic Depth=1978.1m-1950m= 28.1m

Q_{max} =852.17 m3/se

Appendix Figure C. 1 showed that the values of breach outflow discharge versus hydraulic depth fall under the envelope curve developed by 14 dam failures data sets. As shown in the figure the intersection point of discharge Vs hydraulic depth is situated far from the curve due to the height of the dam. This again **validates** the breach outflow from the model.

Therefore, the analysis of different breach parameters and inundation mapping was done based on the same model setup used for the breach, by changing the values of selected different parameters of the breach.

The following subsequent section showed the details of the analysis done at the dam site and downstream river selected reach stations.

4.2.2. Time to Dam Failure versus Maximum Breach Discharges

Time to Dam Failure versus Maximum Breach Discharges for a Given Side Slope of Breach. The HEC-RAS dam-break subroutine was used for testing the sensitivity between TFH and Q_{max} for a given Side slope (SS) of breach. The results of testing are summarized in Appendix Table A. 1 for the detail outputs of HEC-RAS.

In this sensitivity analysis, four different TFH were used for HEC-RAS dam-break computer runs and resulted in four dam breaching outflow hydrographs. Figure 4-4 shows four dam breach hydrographs for a given side slope of breach (1:1) with four different TFH. Also, Appendices showed the detail of HEC-RAS dam-break outputs, and Figure 4-4 showed the maximum breach discharge varied with TFH with a given side slope Z. Based on results, the peak discharge increased when TFH increased. A 100 % increase in TFH (0.6Hrs to 1.2Hrs.) resulted in 358 % increment in peak discharge. Again, a 150 % increment in TFH (0.6 Hrs. to 1.5 Hrs.) resulted in 403% increase in peak discharge at the dam site at the same site. The variation of peak discharge with TFH is showed in the following figures and a best fitted equation.

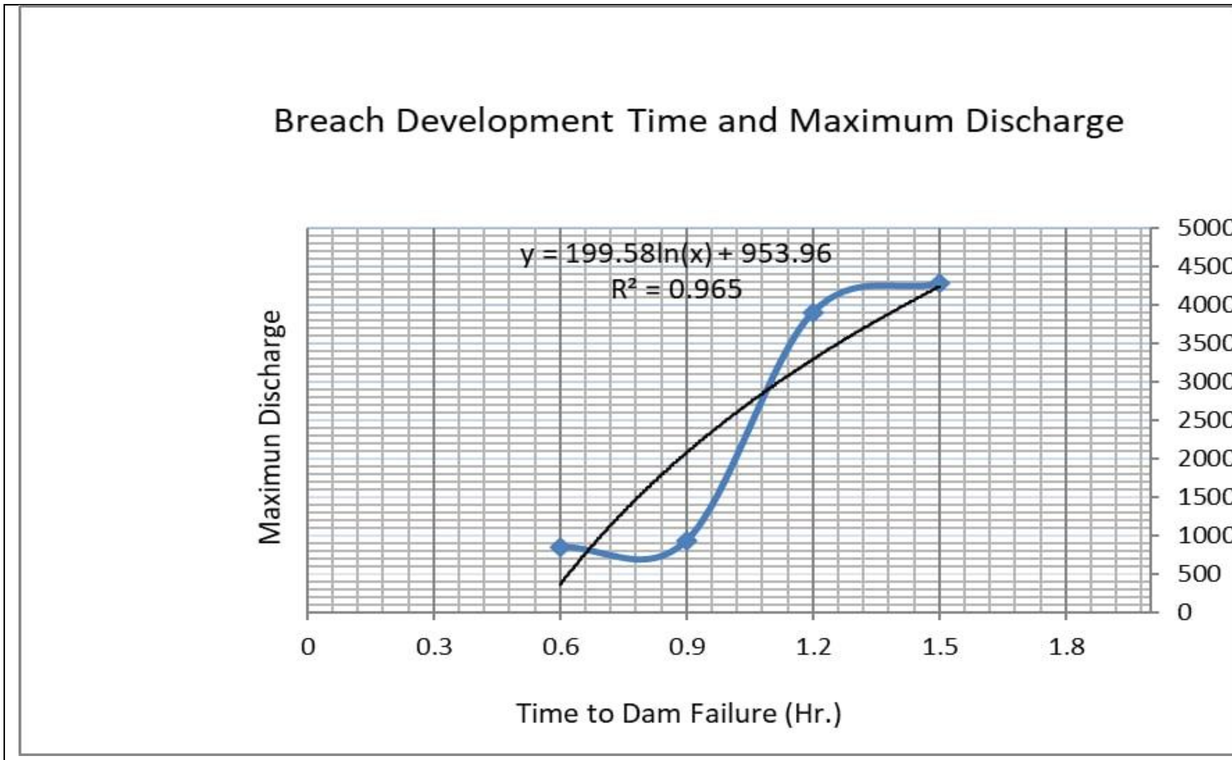

Figure 4. 3 Time to Dam failure and Maximum Discharge at Dam Site

Figure 4-4 shows graphical relationships between the two parameters. An equation developed based on best fitted trend line showed existence of non-linear function between TFH and Q_{max} for a given side slope of breach at dam site.

Q_{max} = 199.58ln (TFH) + 953.63 ………….............................. Equation 4-1

With $R^2 = 0.965$

Where:

Q_{max}: Maximum Discharge at the Dam

TFH: Time to Dam Failure in Hours

The equation revealed the qualitative influence of change in dam breach development time on the maximum discharge at dam site. As shown above and mentioned in the literature, breaching hydrograph was a multi-variable analysis that involved using empirical formula, historical data and a lot of personal experience. Despite all of these, the above results confirmed that predicting dam breach outflow hydrographs was dependent on and sensitive to a minor change in dam breach development time.

4.2.3. Side Slope Verses Maximum Breach Discharges

4.2.3.1. Side Slope Verses Maximum Breach Discharges for a Given Time to Dam Failure

The HEC-RAS dam-break subroutine was applied and resulted in the following summarized data for the two variables, Q_{max} and SS, at the dam provided that TFH remained the same. Appendix Table A. 2 presented the detail outputs of HEC-RAS reservoir routings.

The shape parameter (Z) identified the side slope of the breach, i.e., 1 vertical: Z horizontal. For this study a trapezoidal shape of breach was assumed, and breach bottom elevation was set to remain constant in all cases as discussed in methodology section 3.5.1.5. The sensitivity analysis between side slope of breach and maximum discharge was done based on the above-mentioned conditions.

Results indicated that the maximum discharge increased as the side slope of breach decreased at the dam site. A 75% decrease in side of slope of breach (1 to 0.25) resulted in 148 % increment in maximum discharge at the dam relative to original data. Again, an 88 % decrease in SS (1 to 0.125) produced 211% increment in maximum discharge at the dam site which shows the small change in side slope affects the flow significantly. The increments of percent change in peak flow were high compared to the corresponding percent changes in SS. This observation was further illustrated in Figure 4-5

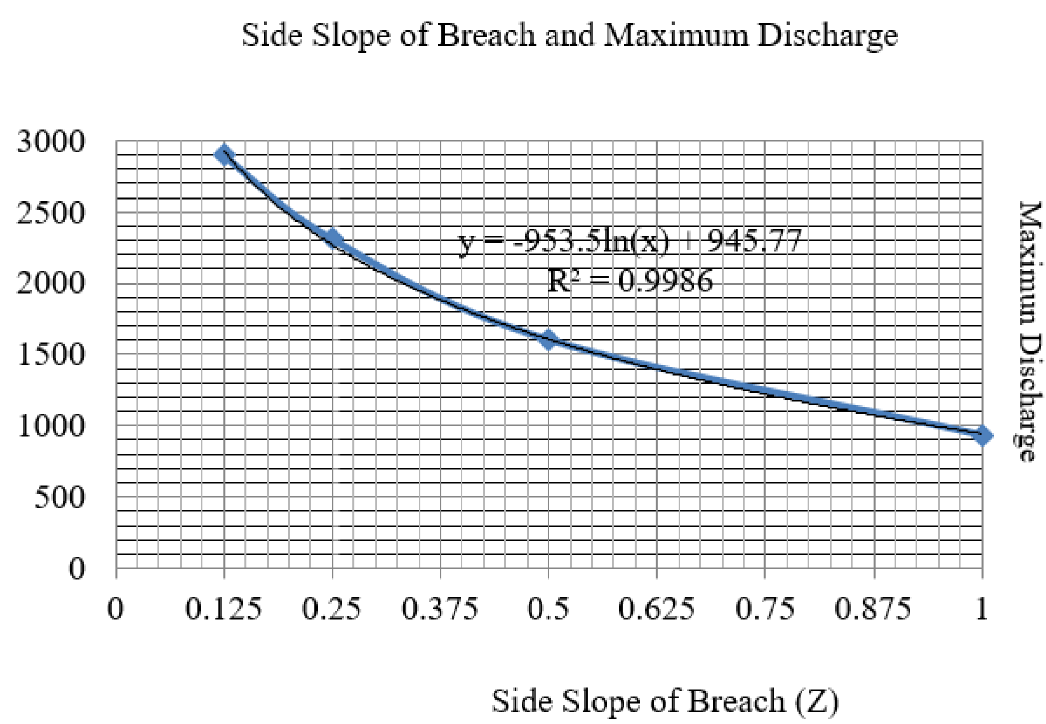

Figure 4. 4 Side Slope of Breach and Maximum Discharge at Dam Site

Figure 4-5 shows the influence of side slope of breach on maximum discharge at the dam location. As the Z value increased, the maximum discharge at the dam decreased at a steady rate as shown in the above figures. The results of this sensitivity analysis indicated that the maximum discharge at the dam site increased as the Z values decreased under the condition, TFH was remained constant.

Further qualitative analysis of the relationship between the two variables is shown in the following equation. The best fit equation developed from Figure 4-5, and it described the sensitivity of peak discharge to Z values and defined as follows:

$$Q_{max} = -953.5\ln(Z) + 945.77 \qquad \text{............................} \quad (4\text{-}2)$$

With $R^2 = 0.9986$

Where:

Q_{max}: maximum discharge at the dam

Z: side slope of dam breach

The best fit equation showed estimated non-linear relationship between the two parameters, and this evaluation agreed with the coefficient of determination R^2 which indicated the level of the equation's accuracy.

4.2.4. Relative Effects of Time to Dam Failure and Side Slope of Breach

The relative effects of Time to Dam Failure and Side Slope of Breach on Maximum Breach Discharges. The HEC-RAS dam-break subroutine was applied to test the sensitivities between TFH and Q_{max} with given SS, and SS and Q_{max} with given TFH at the dam site. The summary results were tabulated in the Appendix Table A. 3.

Result shows how the peak discharge at the dam site reacted to changes in the two parameters which are time to dam failure and side slope of breach. The figure shows four conditions of side slope of breaches. Each side slope of breach was combined with four time to dam failure and produced four maximum discharges at the dam site. This analysis was done four times for the four Z values.

It if found that the peak discharge decreased by 9% when the time to dam failure reduced by 33% and side slope of breach reduced by 50%. On the other hand, a 50% decrease in side slope of breach (1 to 0.5) without changing time to dam failure resulted in 0.0023% reduction in maximum discharge at the dam site.

Further graphical sensitivity analysis was done to determine a controlling parameter that influenced the maximum discharge at the dam site the most. Figure 4-6 illustrates the trend existed between the parameters at the dam site. The figure depicted that the change in side slope of breach resulted in closely spaced graphs for a wide range of change in time to dam failure. This implied that the maximum discharge at the dam site was highly sensitive to the time to dam failure than side slope of breach.

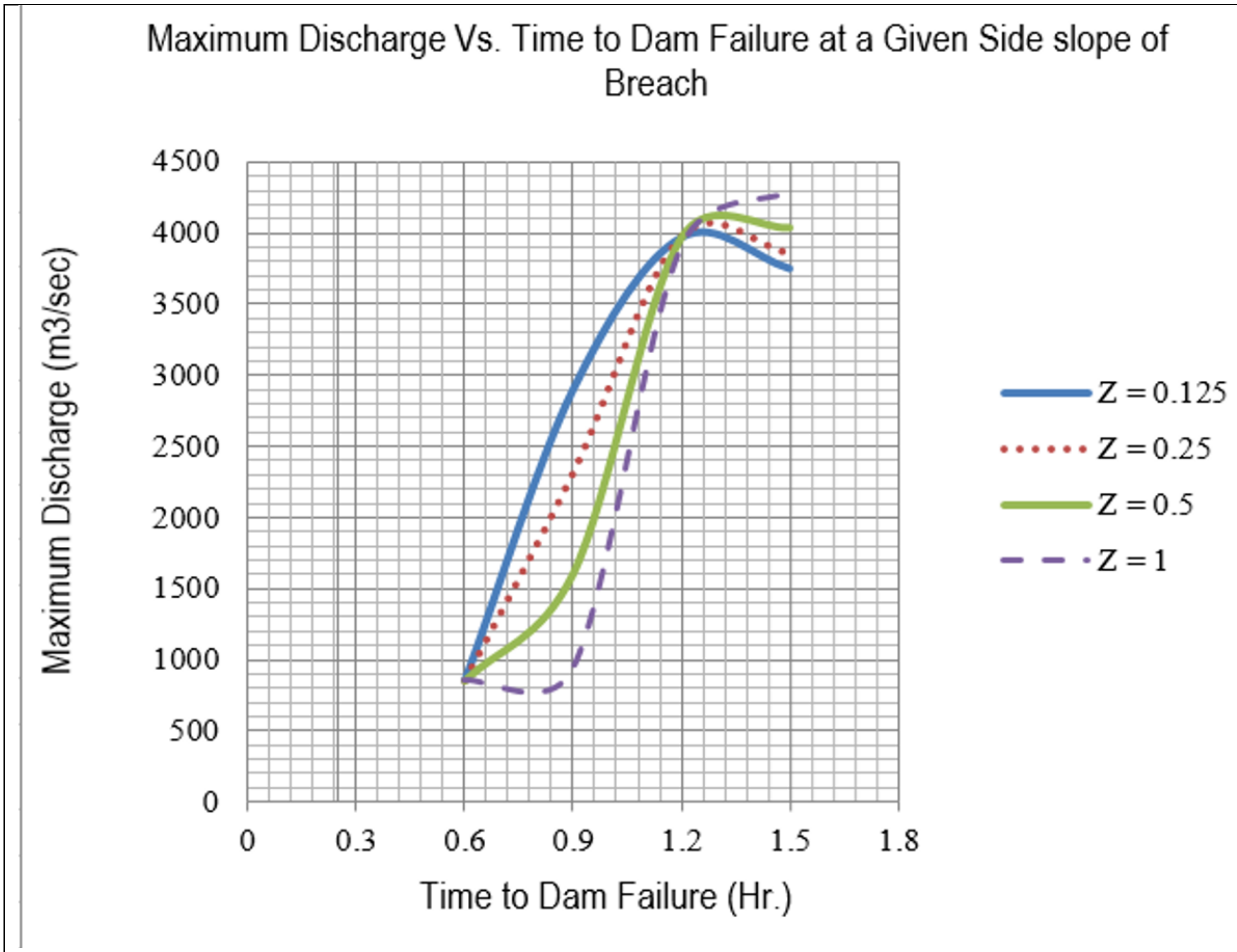

Figure 4. 5 Effects of Time to Dam Failure and Side Slope on Peak Flows at Dam Site

4.3. Downstream of the Dam

4.3.1. Effects of Time to Dam Failure and Manning Coefficient (n_o)

The effects of Time to Dam Failure and Manning Coefficient on Peak Flows at Specified River Stations. Two reach stations were selected to conduct a thorough investigation in identifying the biggest peak flow influencing parameter, Manning value or time to dam breach. Reach station 01 and 02 were selected and results of different computer runs were tabulated in the Appendix Table B. 1 and B. 2.

As shown in the figure below, an increase in time to dam failure by 66.67% (0.9hr to 1.5hr) and 10% (n_o to $1.1n_o$) increase in Manning coefficient resulted in 26.7% decrease in peak flow at reach station-01. Similarly, a 0% increase in time to dam failure and 10% increase in Manning value resulted in 0.33% reduction in peak flow at the same reach station. The figure below 4-7 and 4-8 shows the same pattern for reach station. A 66.67% increase in time to dam failure and 10% increase in Manning value resulted in 23.37% decrease in peak flow at reach station-02. Similarly, a 0% increase in time to dam failure and 10% increase in Manning value resulted in 2.1% decrement in peak flow at the same reach station. Overall observation of the results indicated that the change in peak flow at the two stations was highly sensitive to a minor change in time to dam failure than Manning coefficient. This finding was further illustrated in the following figures.

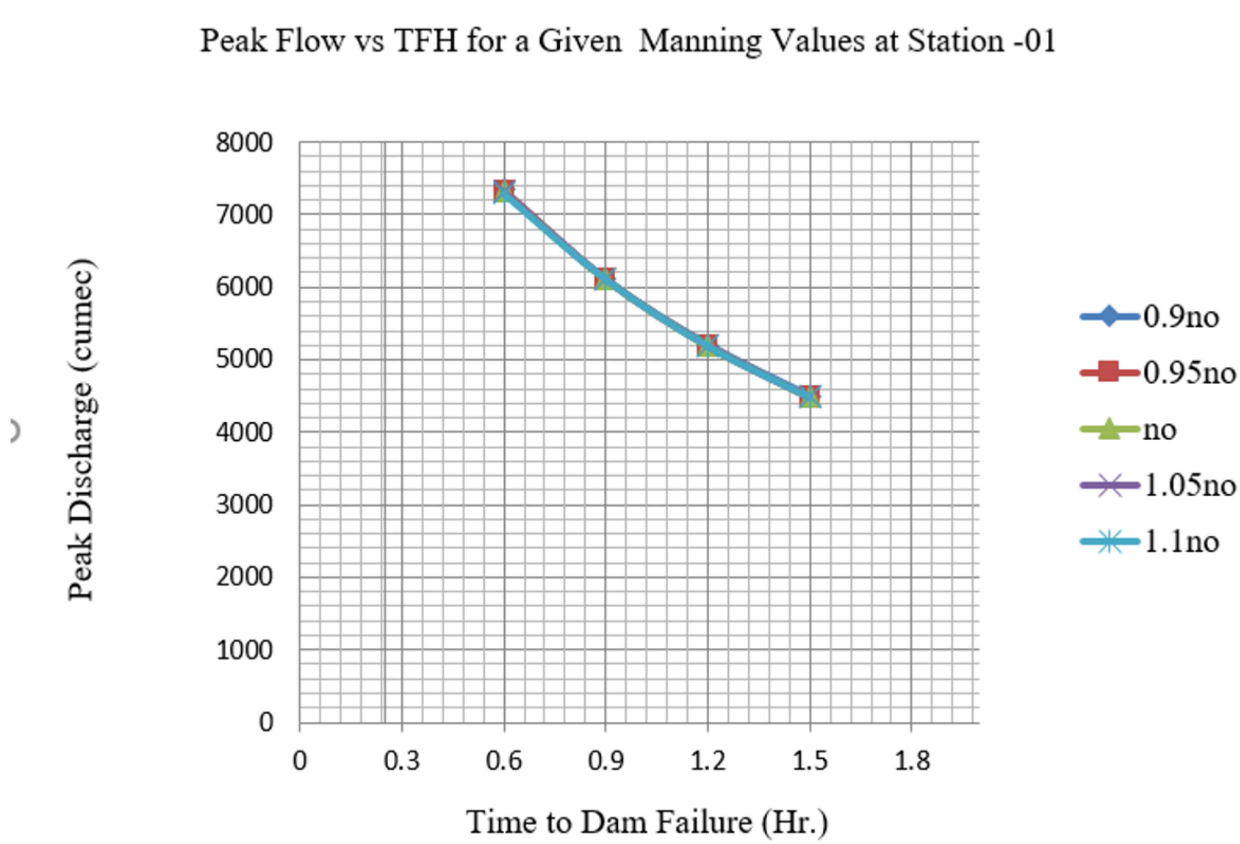

Figure 4. 6 Peak Flow Profiles for Given Manning Values and TFH at Station-01

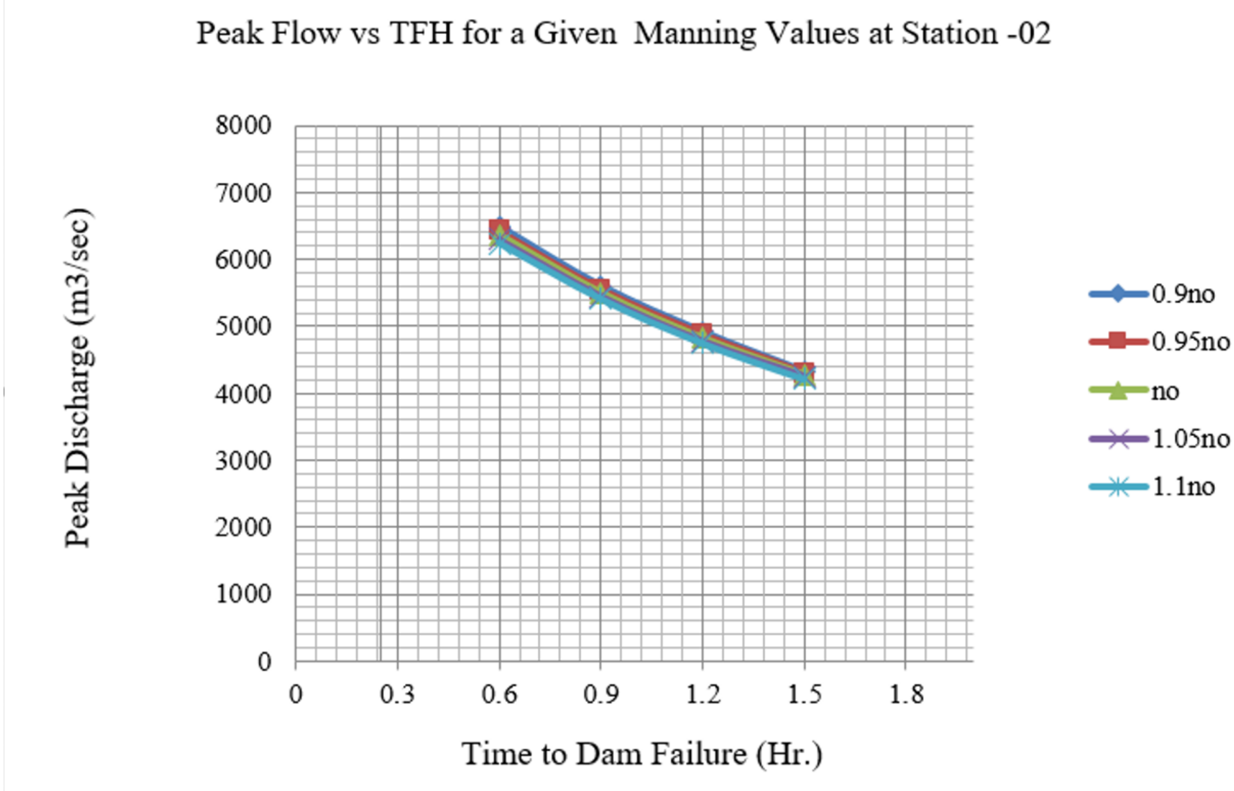

Figure 4. 7 Peak Flow Profiles for Given Manning Values and TFH at Station-02.

The above figures demonstrate that as the Manning coefficient (n_o) and time to dam failure decrease, the peak flow increases at the two reach stations. The controlling parameter between the two variables was shown in the figures. However, the graphical results at the two stations indicated that the peak flow had an inverse relation with Manning coefficient and time to dam failure, i.e., the smaller time to dam failure and Manning value are, the higher the peak flow is. As shown in the result of Kalid-Dijo Dam analysis at TFH 0.6hr and Manning coefficient ($0.9n_o$ = 0.027) Q_{max} was 6494.75m3/sec. However; at TFH 0.9hr and Manning coefficient (no = 0.03) Q_{max} was 5506.73m3/sec. This shows the model result aligns with Frohlich 2008 equation.

4.4. Inundation Area Mapping

The downstream inundation area was evaluated with velocity/depth approach. The downstream area has 1,036ha of gravity irrigation system and 64ha of motorized irrigation system on the left banks of the Kalid Dijo rivers, one pump station (PS) which are constructed for development of

64ha irrigation. Therefore, the inundation area mapping focuses mainly on these infrastructures and inhabited area to evaluate the severity of the inundation.

The following model result shows the degree of severity of the inundation on the downstream area of the dam.

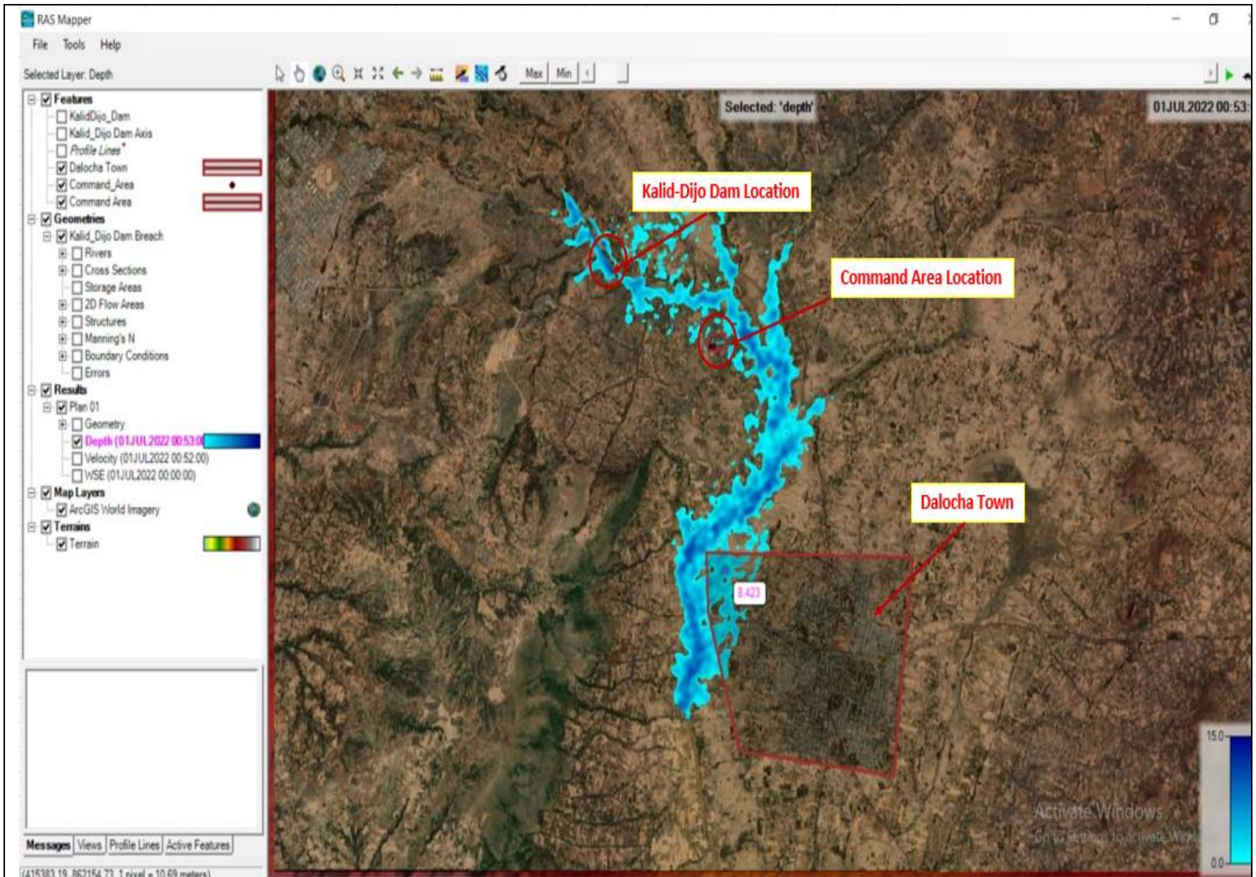

Figure 4. 8 The downstream inundation area up to 7 km from the dam

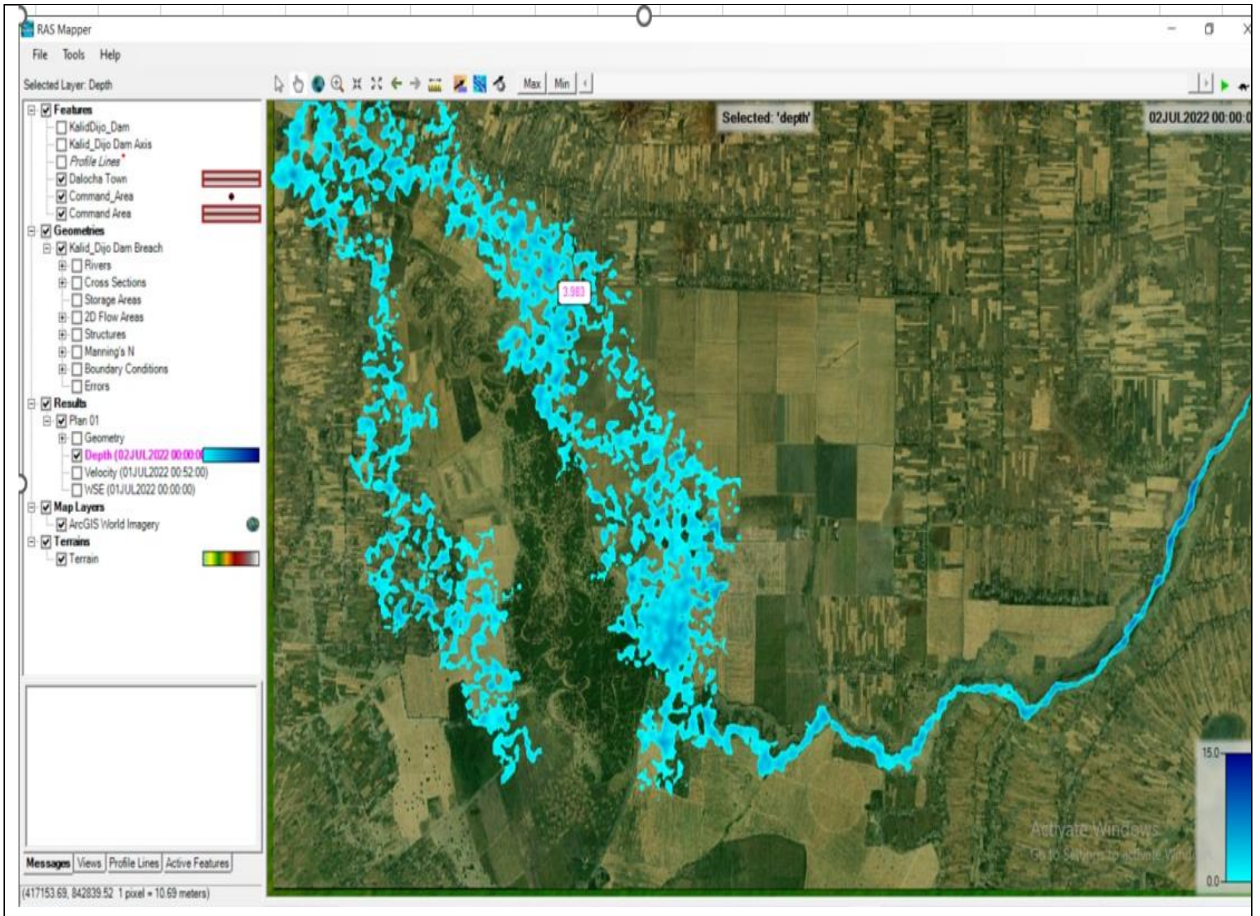

Figure 4. 9 Shows the downstream inundation area at the distance of 36km from the Dam

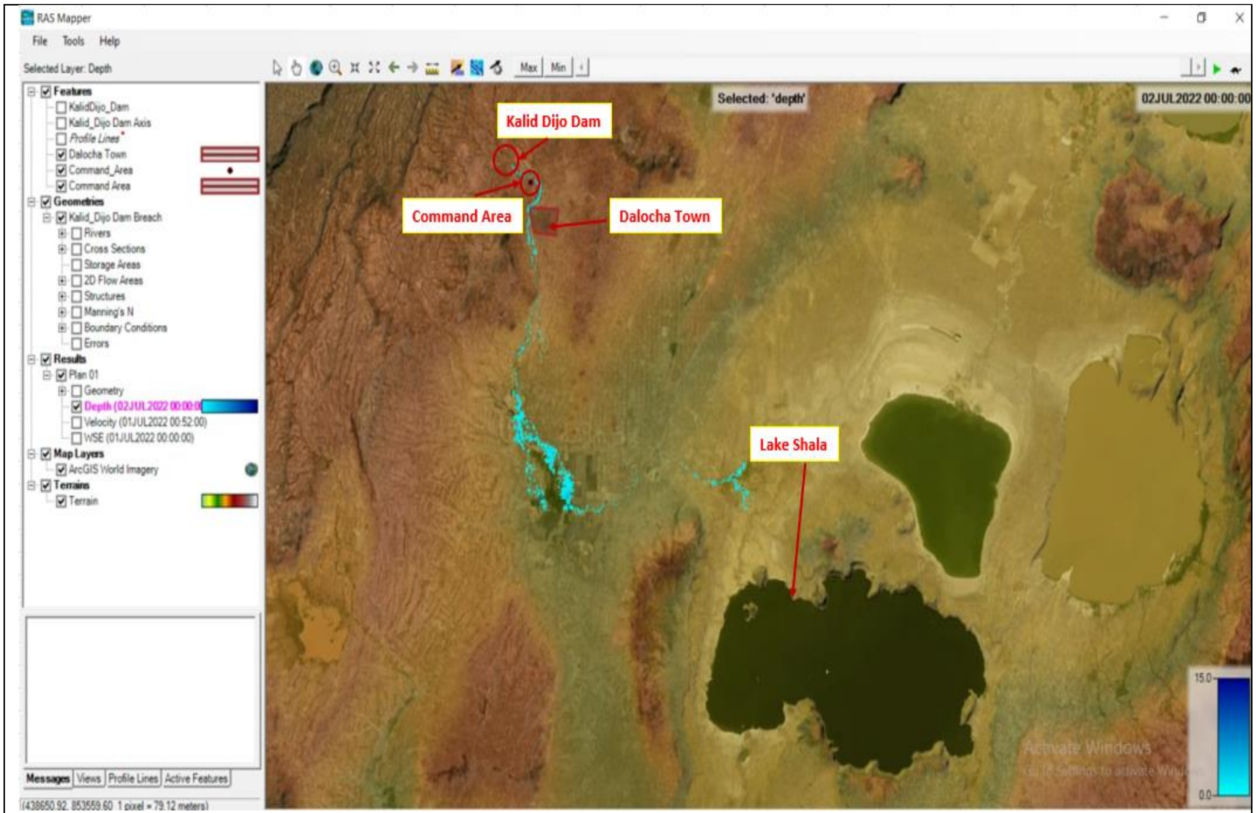

Figure 4. 10 Total downstream inundation area of Khalid-Dijo Dam

The above figures showed the upstream reservoir area and the downstream inundation of the major infrastructures at the downstream of the dam. As shown in the figures the circular delineated area (with red color) is Dalocha Town.

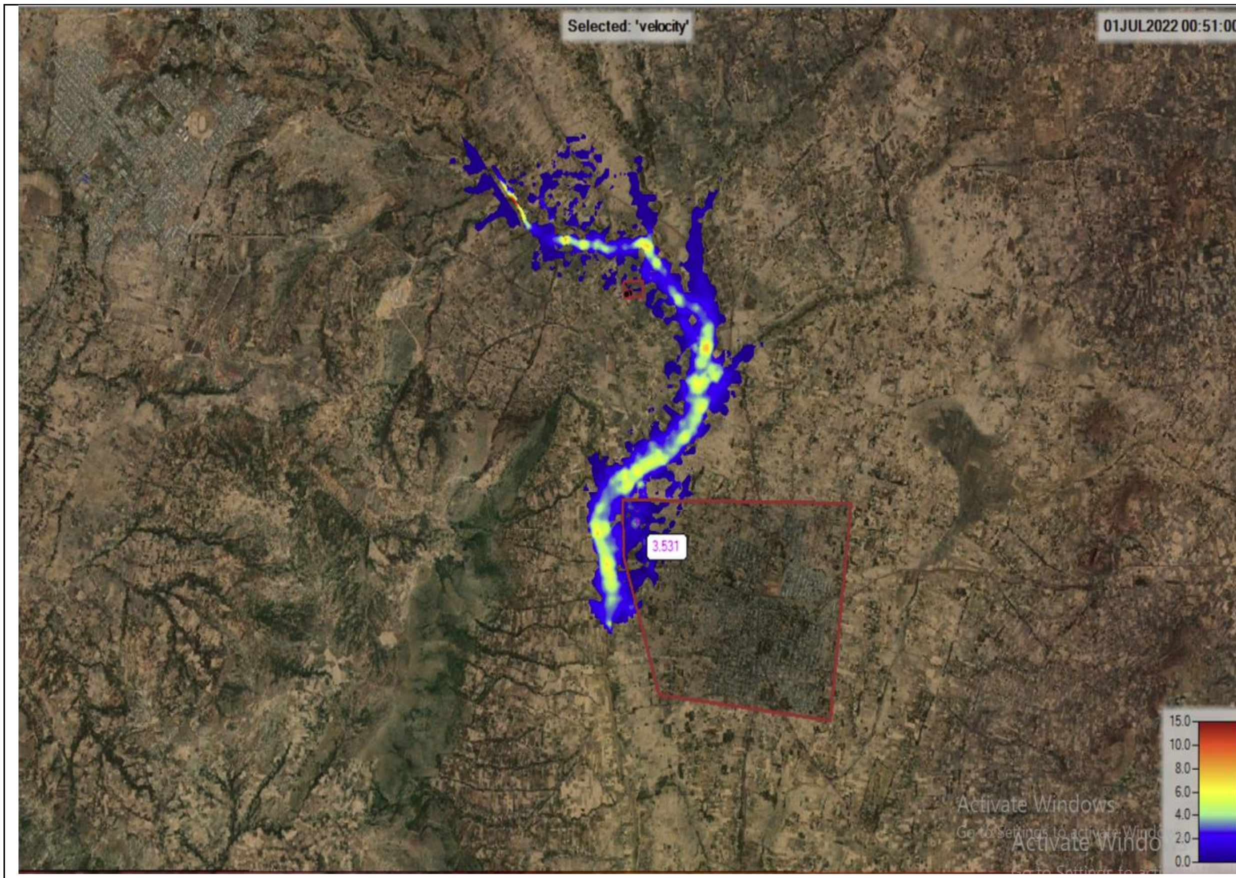

Figure 4. 11 Downstream inundation velocity profile up to 7km from the Dam

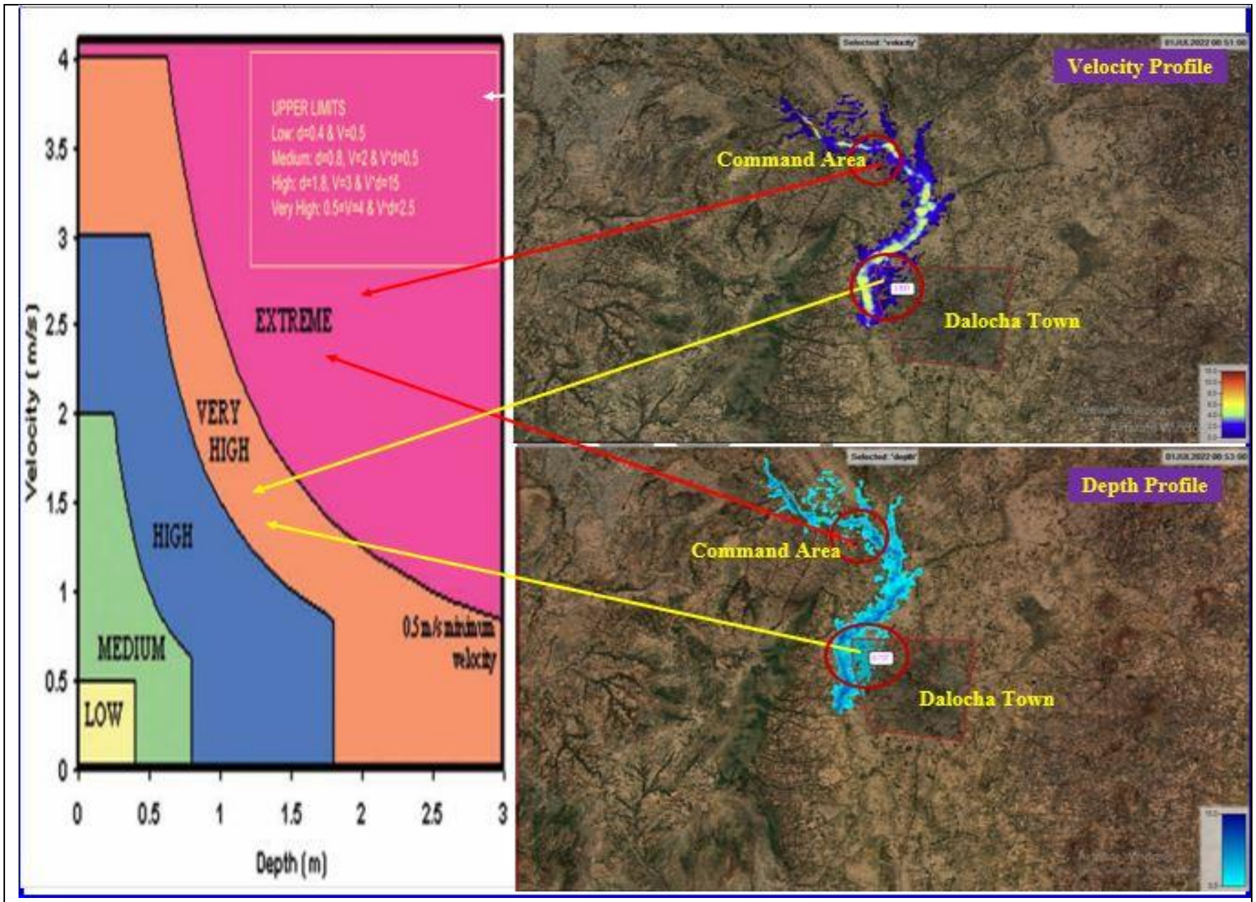

Figure 4. 3 Comparison Depth/Velocity flood hazard Approach (Committee, 2006).

Different studies and guidelines have different flood hazard classification limits and justifications. The FEMA (2014) guideline is used for this study. Flood severity grid prepared by FEMA is shown in the table 4-1 in order to obtain upper limits of the depth * velocity product for each category of hazard level.

Representing the information in the figure above in a simplified table, the FEMA document states:

"To produce a flood severity grid that exactly matches the categorization shown in the figure above, additional rules would need to be applied when calculating the depth * velocity product, to take into account the depth and velocity upper limits of each category. Additionally, the flood severity thresholds are different depending on whether they are being considered related to the impact on humans, vehicles, or buildings. As a simplified approach, the following depth * velocity categories were applied when symbolizing the results of the dataset. However, other categorizations of this data may be used where desired."

Table 4. 1 Simplified Flood Depth and Velocity Severity Grid Symbolization Categories (FEMA, May 2014)

Flood Severity Category	**Depth * Velocity Range (m^2/sec)**
Low hazard	< 0.2
Medium hazard	0.2 – 0.5
High hazard	0.5 – 1.5
Very High hazard	1.5 – 2.5
Extreme hazard	> 2.5

As shown in figure 4-13, some part of Dalocha town is in the range 0.170-2.081m/s for velocity and 0.342-3.448m depth; most part of the town falls under high hazard region. The average result of marked areas in terms of depth and velocity is as follows: Marked Area-1: 1.895m * 1.126 m/sec = 2.132m2/sec is in the range of 1.5-2.5. According to FEMA (2014) the result falls under very high hazard region. Moreover; most of the downstream of the dam including the developed irrigation area falls under the extreme hazard region. The result of the command area shows that 5.269m * 4.279 m/sec = 22.546 m2/sec >2.5; therefore, the hazard is extreme as shown in the figure 4.3 above.

Therefore, the inundation mapping result as shown in the figure 4-12 failure of the Kalid-Dijo dam breach directly and indirectly affect about 9,059 households/52,054 peoples living in and around the commend area. Besides, it damages the most important infrastructures such as roads, 1100 ha developed irrigation lands and structures. In general, 478.35Km^2 area is inundated by Kalid-Dijo Dam breach.

5. Conclusion and Recommendation

5.1. Conclusions

In general, the dam breach parameters, sensitivity analysis and inundation mapping result showed that, the failure of the dam inundated Dalocha town, developed irrigation, important hydraulic and road infrastructures located at the downstream of the dam. Based on unsteady computer run resulted from decreasing and increasing the values of time to dam failure and side slope, it is observed that the change of TFH in the range of 100% (0.6hrs to 1.2hrs) to 150% (0.6hrs to 1.5hrs) resulted 358% to 403% change of the maximum discharge at the dam site which is very high compared to 88% increase in SS value which produced only 211% decrease in maximum discharge.

The result of the downstream river analysis is found from forty full unsteady computer run for two randomly selected observation stations (station-01 and station-02) showed that the peak flows were sensitive to changes in time to dam failure, side slope of breach and Manning coefficient at various degrees and sensitive parameter was TFH as shown in the below points.

Based on Dijo dam breach parameters and inundation mapping analysis results, which were obtained through application of HEC-RAS-5.0.6, the following conclusions were made on the relationships among the key dam breaching parameters and open channel parameter.

- The analysis of dam breach parameters and inundation mapping showed that the PMF (The maximum breach outflow discharge) is 852.18 m3/sec which results in overtopping of the Kalid Dijo dam by 0.6m can inundated an area of 454.19km2.
- The resulting flood inundated the irrigation scheme on the banks of Dijo River with velocity range of 0 m/s - 6.4m/s and depth of water greater than 4 meters which falls under extreme hazard based on depth/velocity approach.
- For the analysis four TFH were used (0.6Hrs, 0.9Hrs, 1.2Hrs and 1.5Hrs), the smaller change 25 % increase in TFH (from 1.2Hrs to 1.5Hrs) resulted in 10% increase in peak discharge at the dam site while the maximum change 150 % in TFH (from 0.6Hrs to 1.5Hrs) resulted in 403 % increase in peak discharge at the same site which is five times higher. This implied

that when the dam collapses rapidly, the peak discharge became high at the dam site, which depicts the maximum discharge at the dam site was very sensitive to change in TFH.

- The peak discharge at the dam site increased by 88% when the TFH increased by 50% (from 0.6hr to 0.9hr) and SS reduced by 50% (from 1 to 0.5). And a 50% reduction in SS (from 1 to 0.5) without changing TFH resulted in 0.0023% reduction in peak discharge. Based on the test results, evaluation of the relative influences of the two parameters on the maximum discharge demonstrated that the peak discharge was highly sensitive to a smaller change in TFH than SS at the dam site.
- A 50% increase in TFH (from 0.6hr to 0.9hr) and a 10% increase in Manning coefficient (from n to 1.1n) resulted in 16.54% decrease in peak flow at reach station-01. Similarly, without any change in TFH (=0.6hr) and a 10% increase in Manning (from n to 1.1n) value resulted in 0.33 % reduction in peak flow at the same reach station. A similar observation made at a reach station-02, showed that a 50% increase in time to dam failure and 10 % increase in Manning value resulted in 15% reduction in peak flow. Similarly, a 0% increase in time to dam failure and 10% increase in Manning value resulted in 2.1 % reduction in peak flow at the same reach station. These results indicated the relative influences of the two parameters on the peak flow at the specified reach stations. Based on the results, the peak flows in the channel were more sensitive to time to dam failure than changes in Manning coefficient.

5.2. Recommendation

- The most sensitive parameter at the dam and downstream reach was time to dam failure. Therefore, careful consideration should be given for TFH in preparation of preparedness' and emergency action plan.
- The breach out flow or flood reaches Dalocha town instantly after 40 minutes of dam breach and inundated some parts of the town for 20 hrs. and 45 minutes. Therefore; high attention should be given to the emergency preparedness plan.
- This paper presented only the effect of Kalid Dijo dam, and its impounded water on the downstream area. However, all the important inhabited areas, road and irrigation infrastructures are found along the banks of the Dijo River. Therefore, the effect of breach outflow at the downstream of Kalid-Dijo dam should be studied to know the worst scenario and for preparation of preparedness plan & realistic EAP.

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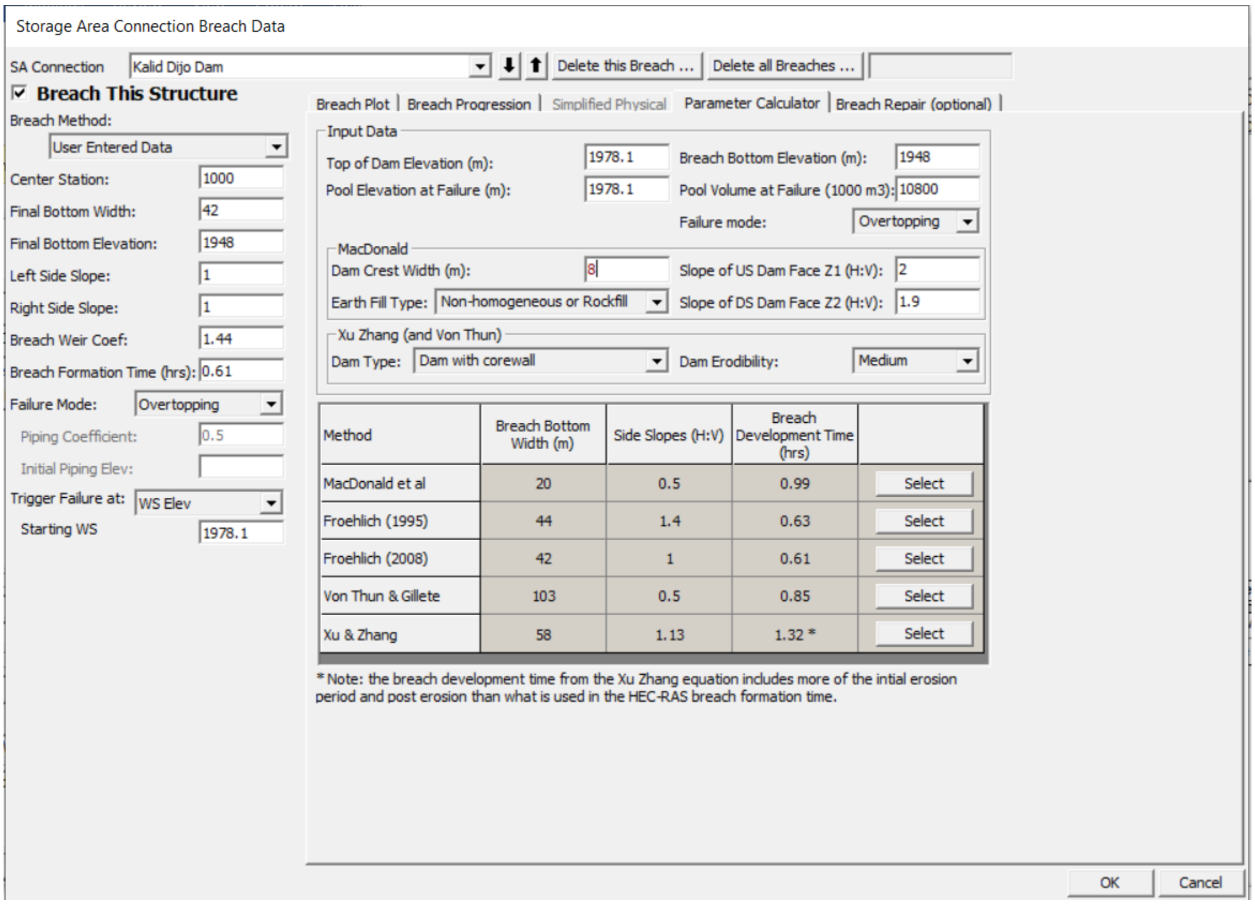
Appendices

HEC-RAS Dam Break Analysis Outflow Output

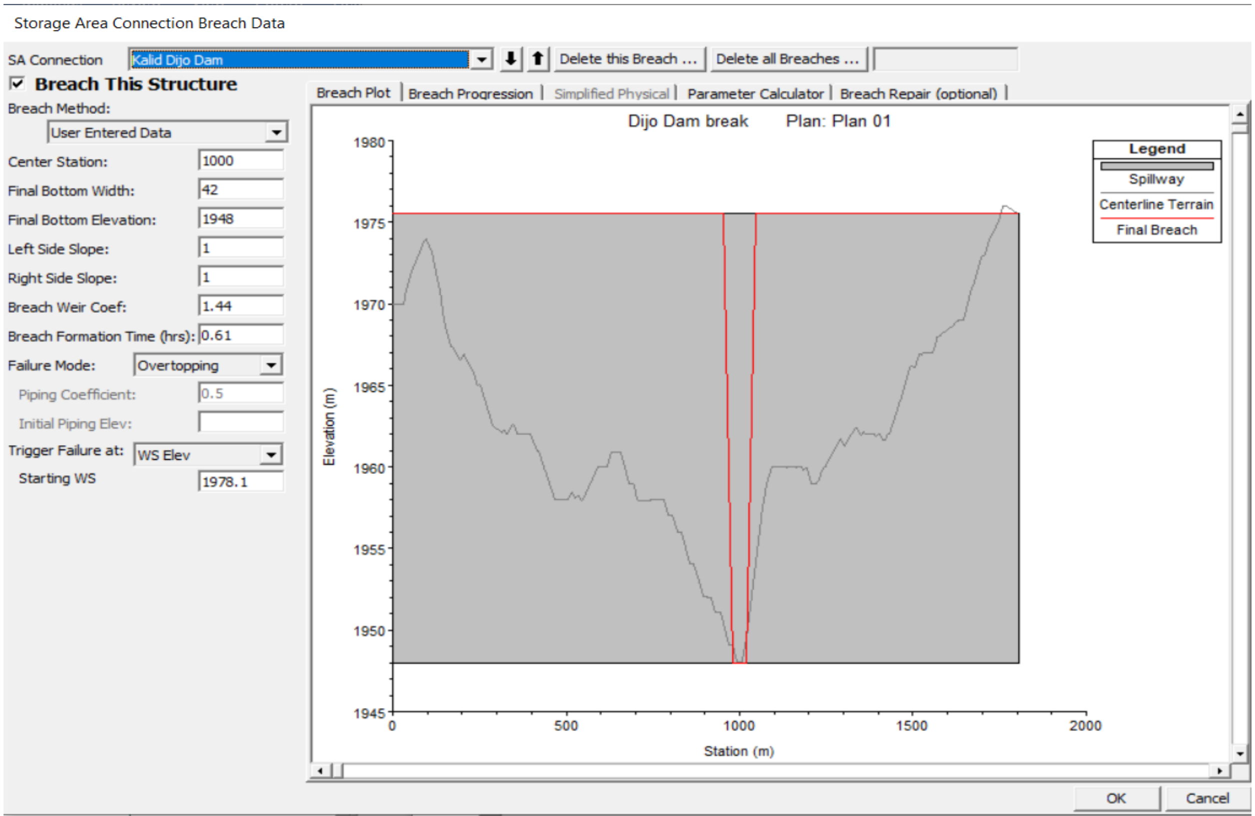
Appendix – A. Dam Site Analysis Model Output and Tabulated Results

Appendix Figure A. 1 Storage area connection breach data and parameter calculator result of five methods.

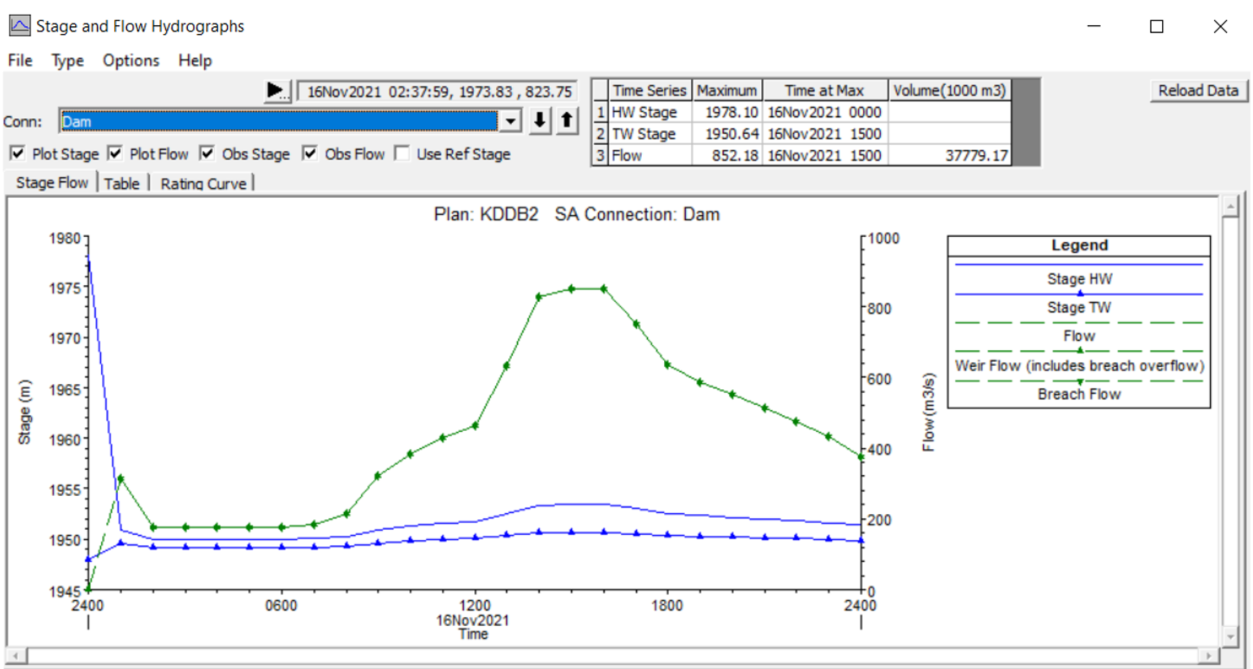


Method	Breach Bottom Width (m)	Side Slopes (H:V)	Breach Development Time (hrs)	
MacDonald et al	20	0.5	0.99	Select
Froehlich (1995)	44	1.4	0.63	Select
Froehlich (2008)	42	1	0.61	Select
Von Thun & Gillete	103	0.5	0.85	Select
Xu & Zhang	58	1.13	1.32 *	Select

Appendix Figure A. 2 Final breach and spillway orientations.

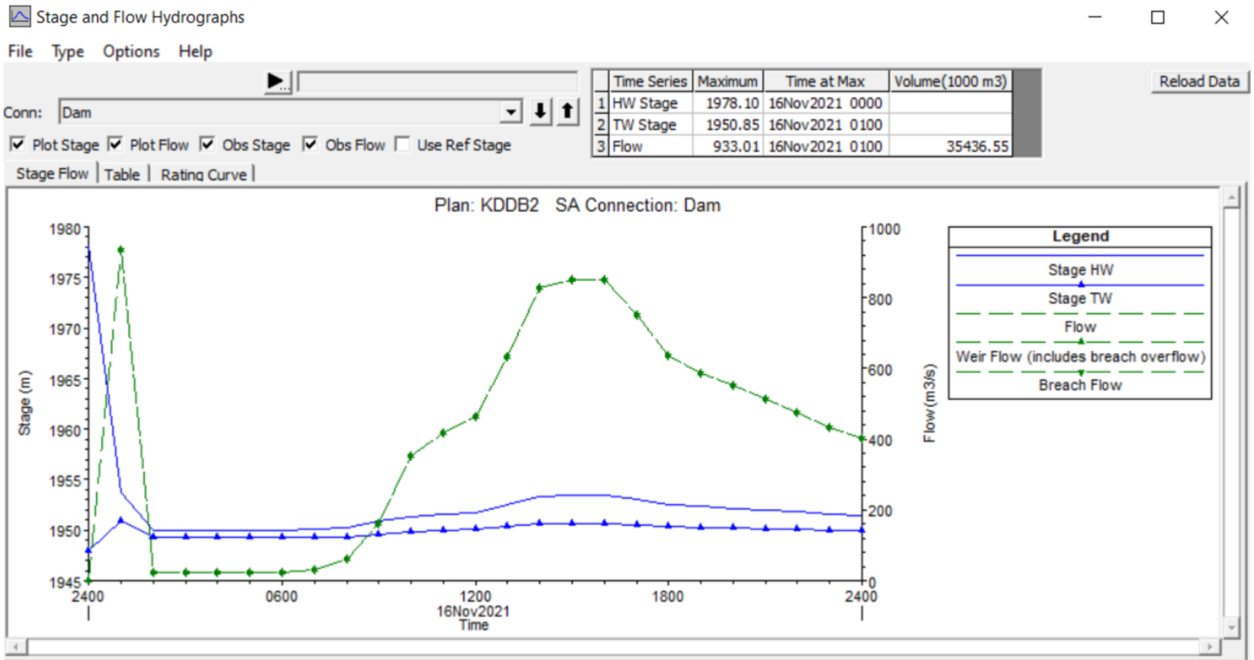


Appendix Figure A. 3 Breach outflow hydrographs TFH=0.6hr and Z=1

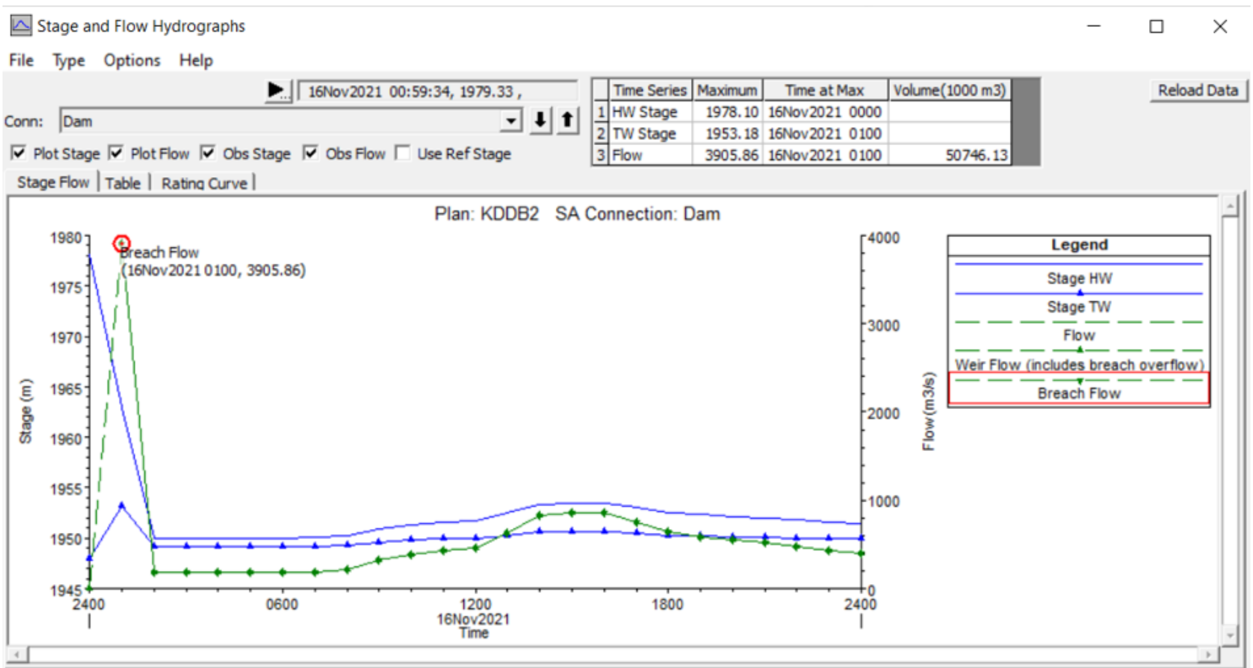


	Time Series	Maximum	Time at Max	Volume(1000 m3)
1	HW Stage	1978.10	16Nov2021 0000	
2	TW Stage	1950.64	16Nov2021 1500	
3	Flow	852.18	16Nov2021 1500	37779.17

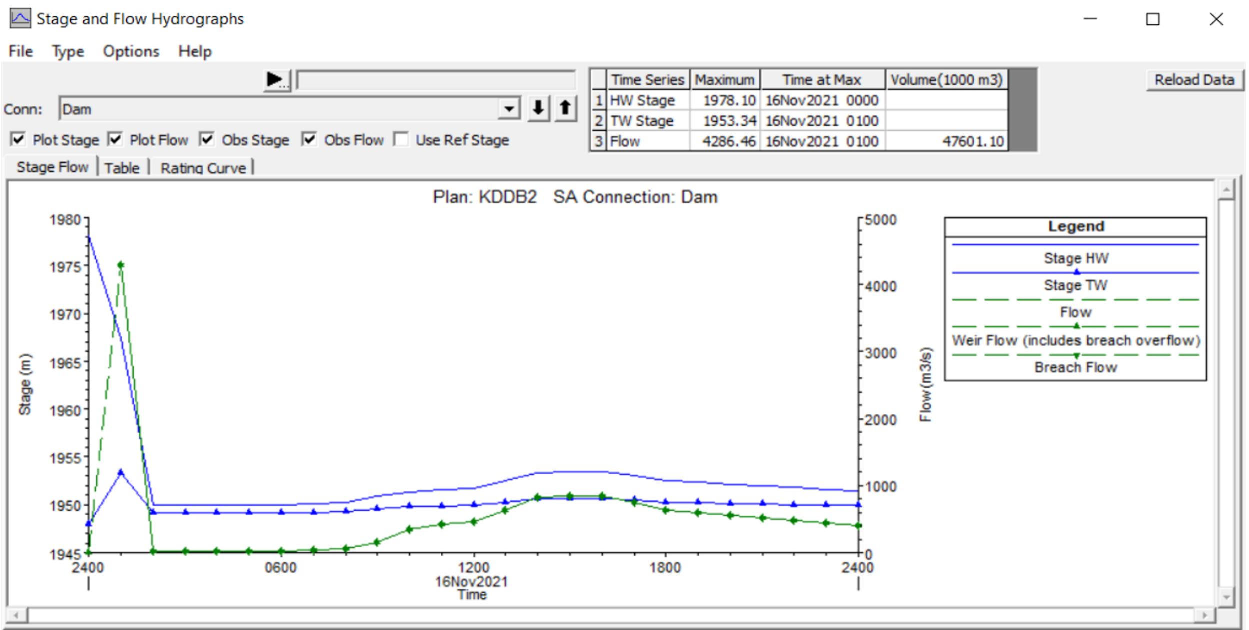
Appendix Figure A. 4 Breach outflow hydrographs TFH=0.9hr and Z=1



Appendix Figure A. 5 TFH=1.2hr and Z=1



Appendix Figure A. 6 TFH=1.5hr and Z=1



Appendix Table A. 1 Maximum Discharge and Time to Dam Failure at Dam Site

Time of Breach Development (Hr.)	**Side Slope of Breach (1 : Z)**	**Maximum Discharge (m3/sec)**
0.6	1: 1	852.18
0.9	1: 1	933.01
1.2	1: 1	3905.86
1.5	1: 1	4286.46

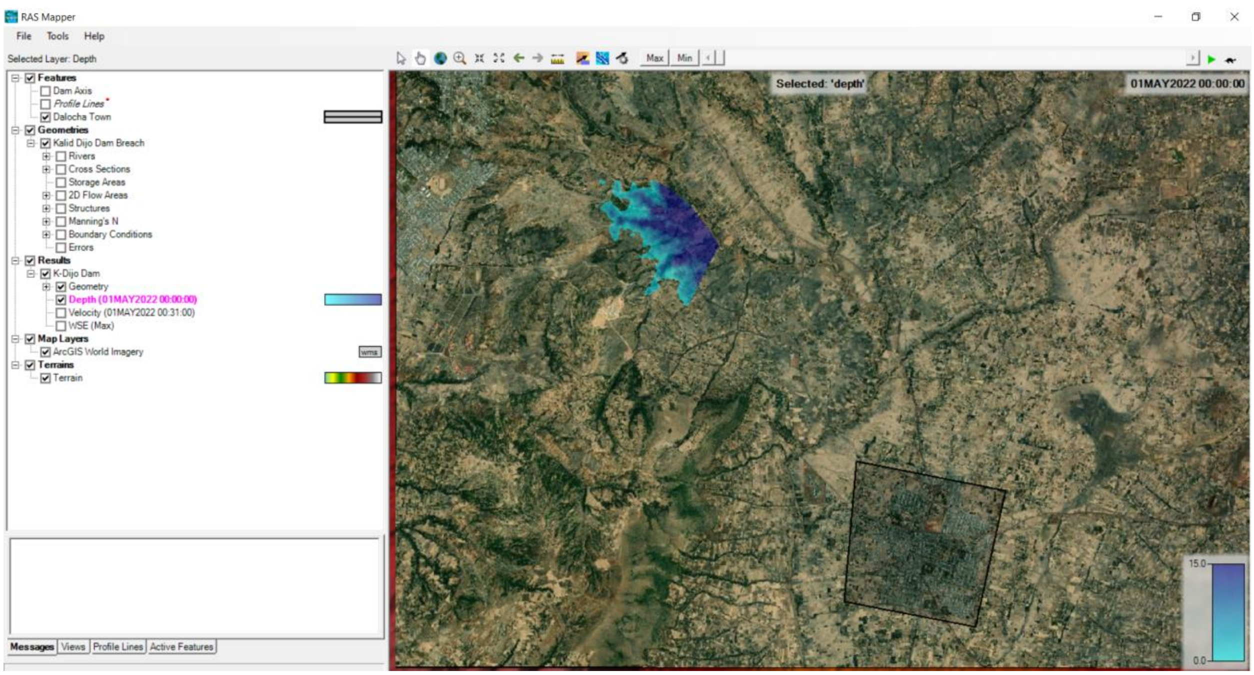
Appendix Table A. 2 Maximum Discharge and Side Slope of Failure at Dam Site

Time of Breach Development (Hr.)	**Side Slope of Breach (1 : Z)**	**Maximum Discharge (m3/sec)**
0.6	1: 8	852.17
0.6	1: 4	852.15
0.6	1: 2	852.16
0.6	1: 1	852.17

Appendix Table A. 3 Maximum Discharge Vs. Time to Dam Failure at a Given Side Slope of Breach

Breach Hydrograph Based on TFH (Hr.)	**Side Slope of Breach, SS (1: Z)**			
	0.125	**0.25**	**0.5**	**1**
	Peak Flow (m3/sec)			
0.6	852.17	852.15	852.16	852.18
0.9	2,899.52	2,312.64	1,603.25	933.01
1.2	3,969.17	3,975.19	3,968.03	3905.86
1.5	3,750.62	3,859.65	4,038.31	4286.46

Appendix Figure A. 7 Up Stream Reservoir Area



Appendix – B. Downstream River Analysis Results

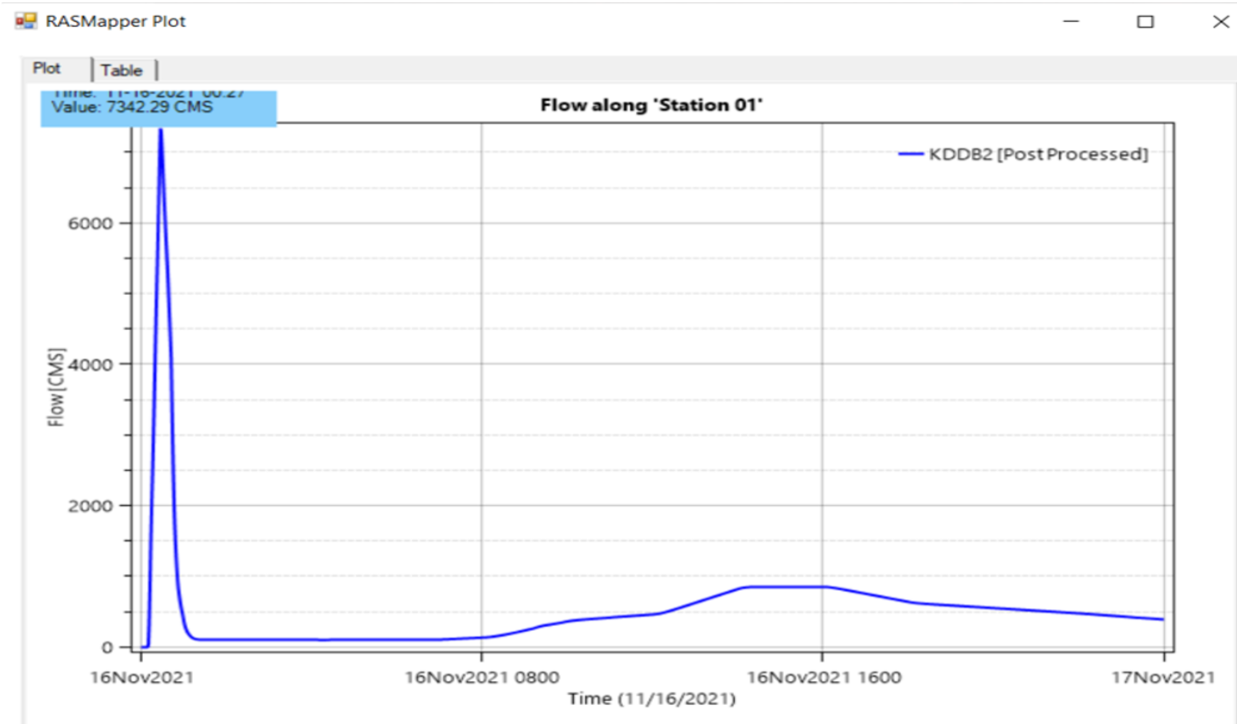
Appendix Table B. 1 Peak flow Table (Peak Flows for Different Manning and TFH Values at Station-01)

Time to Dam Failure (Hr.)	$0.9n_o$	$0.95n_o$	n_o	$1.05n_o$	$1.1n_o$
	Peak Flow (m^3/sec) at Station-01				
0.6	7,342.29	7,336.69	7,325.19	7,313.99	7,300.91
0.9	6,108.83	6,111.04	6,114.55	6,114.28	6,113.46
1.2	5,206.55	5,199.45	5,195.67	5,195.04	5,192.58
1.5	4,493.18	4,481.87	4,492.28	4,488.72	4,481.77

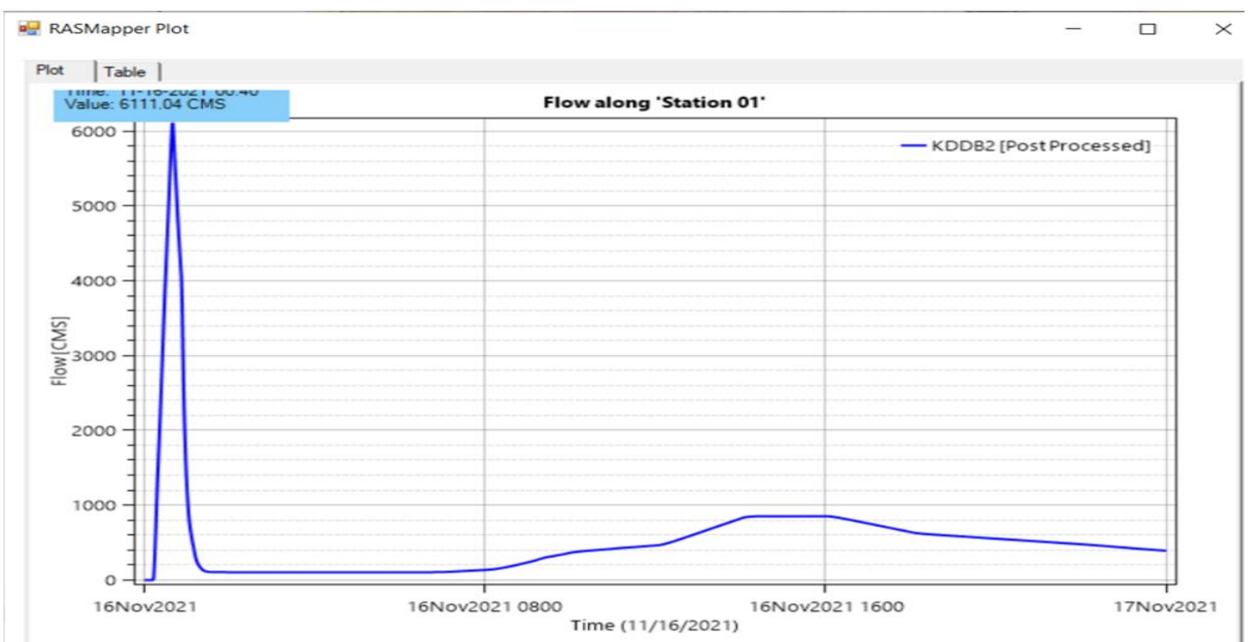
Appendix Table B. 2 Peak Flows for Different Manning and TFH Values at Station-02

Time to Dam Failure (Hr.)	$0.9n_o$	$0.95n_o$	n_o	$1.05n_o$	$1.1n_o$
	Peak Flow (m^3/sec) at Station-02				
0.6	6,494.75	6,432.08	6,371.92	6,307.33	6,237.96
0.9	5,605.95	5,555.70	5,506.73	5,454.95	5,410.93
1.2	4,919.92	4,878.65	4,829.48	4,782.33	4,750.07
1.5	4,332.45	4,303.66	4,283.72	4,257.26	4,219.91

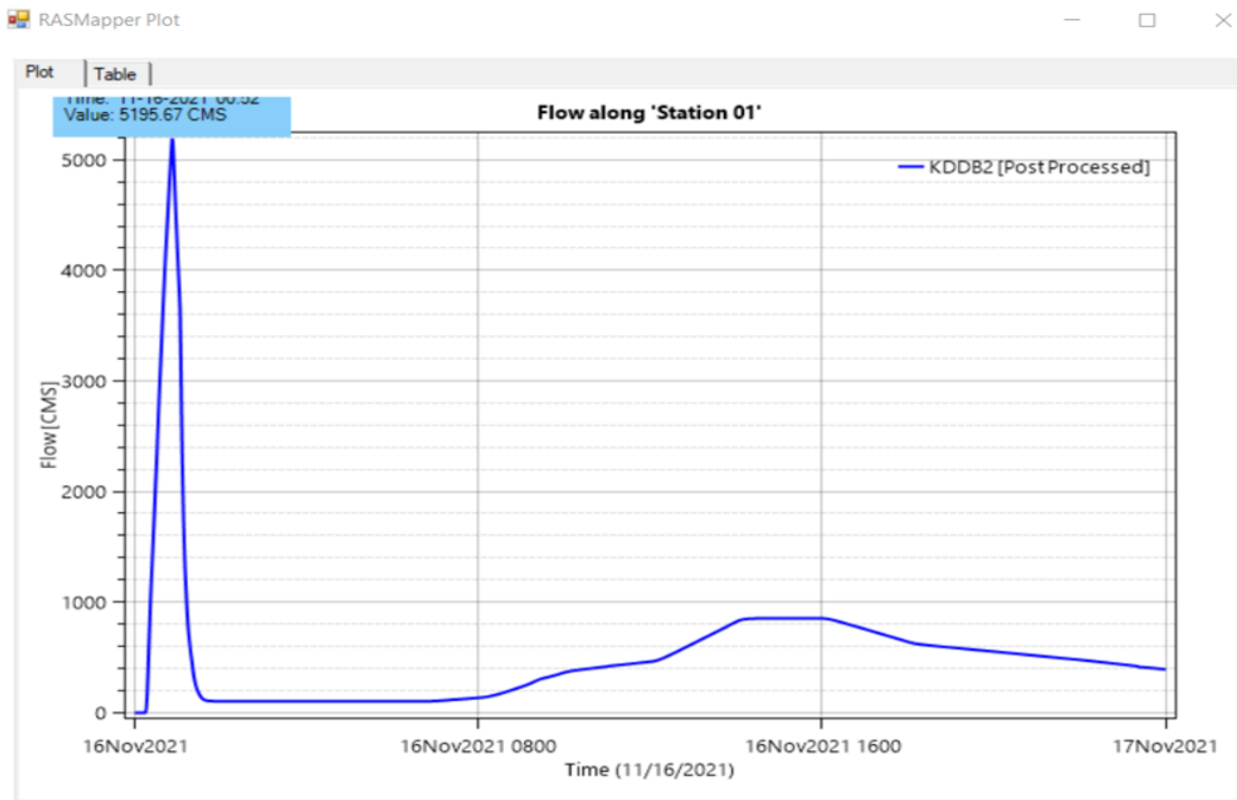
Appendix Figure B. 1 Peak flows at station-01 TFH=0.6, n=0.027



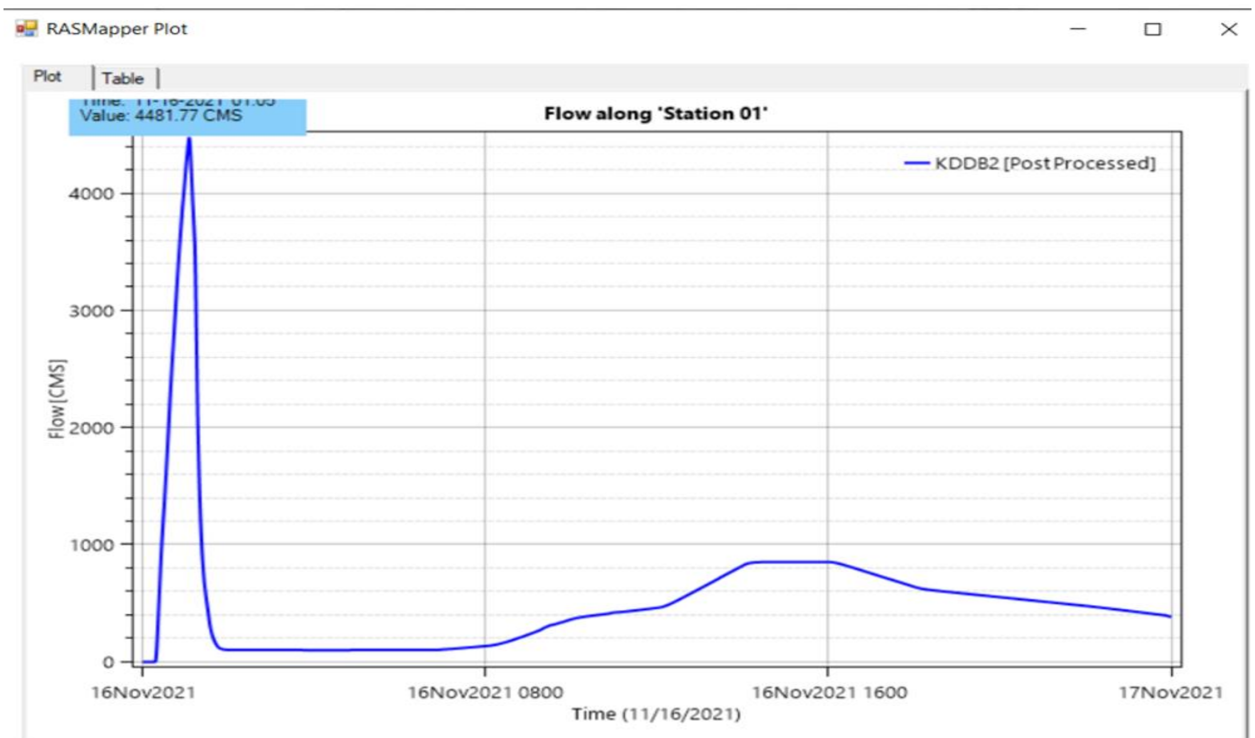
Appendix Figure B. 2 Peak flows at station-01 TFH=0.9 n=0.0285



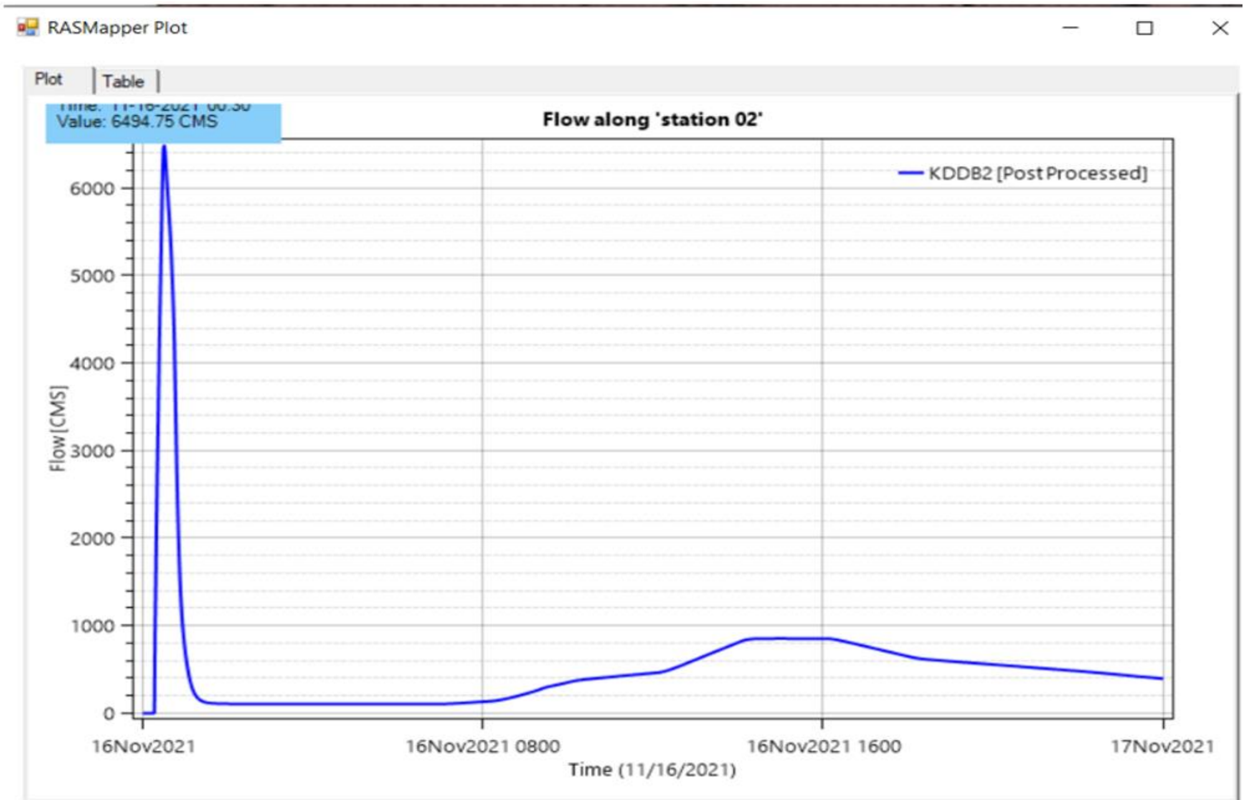
Appendix Figure B. 3 Peak flows at station 01 TFH=1.2 n=0.03



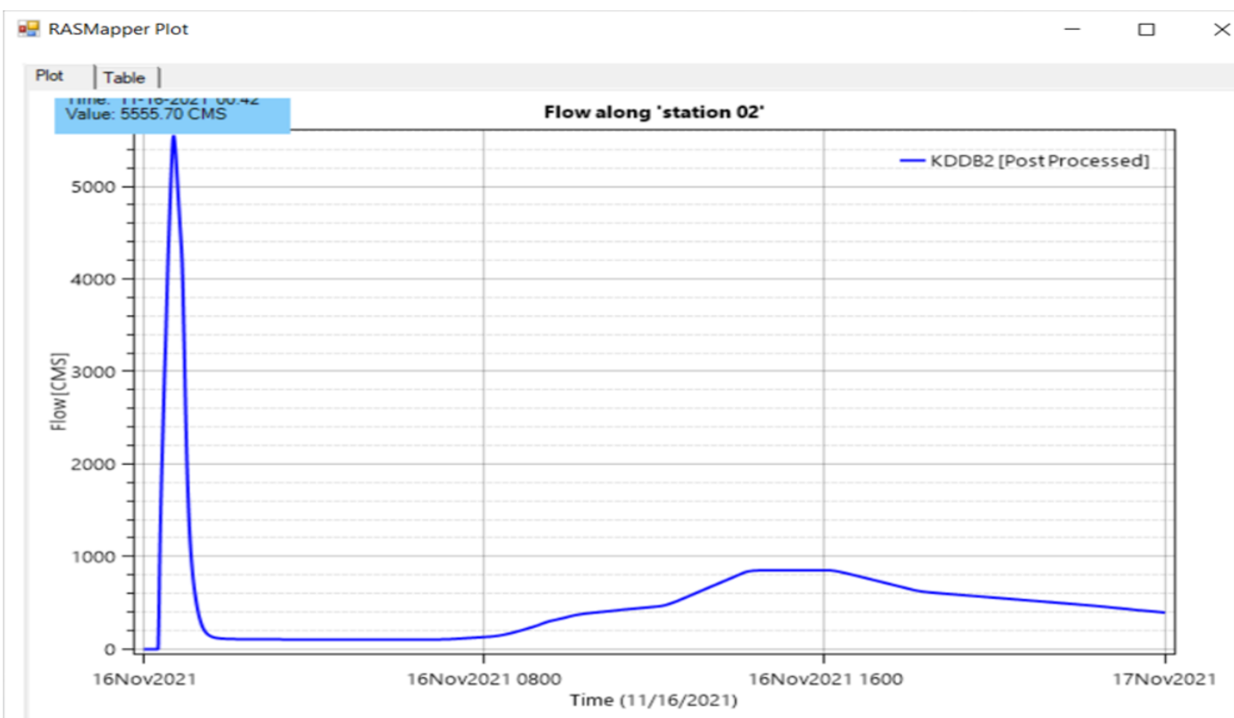
Appendix Figure B. 4 Peak flows at station-01 TFH=1.5 and n=0.033



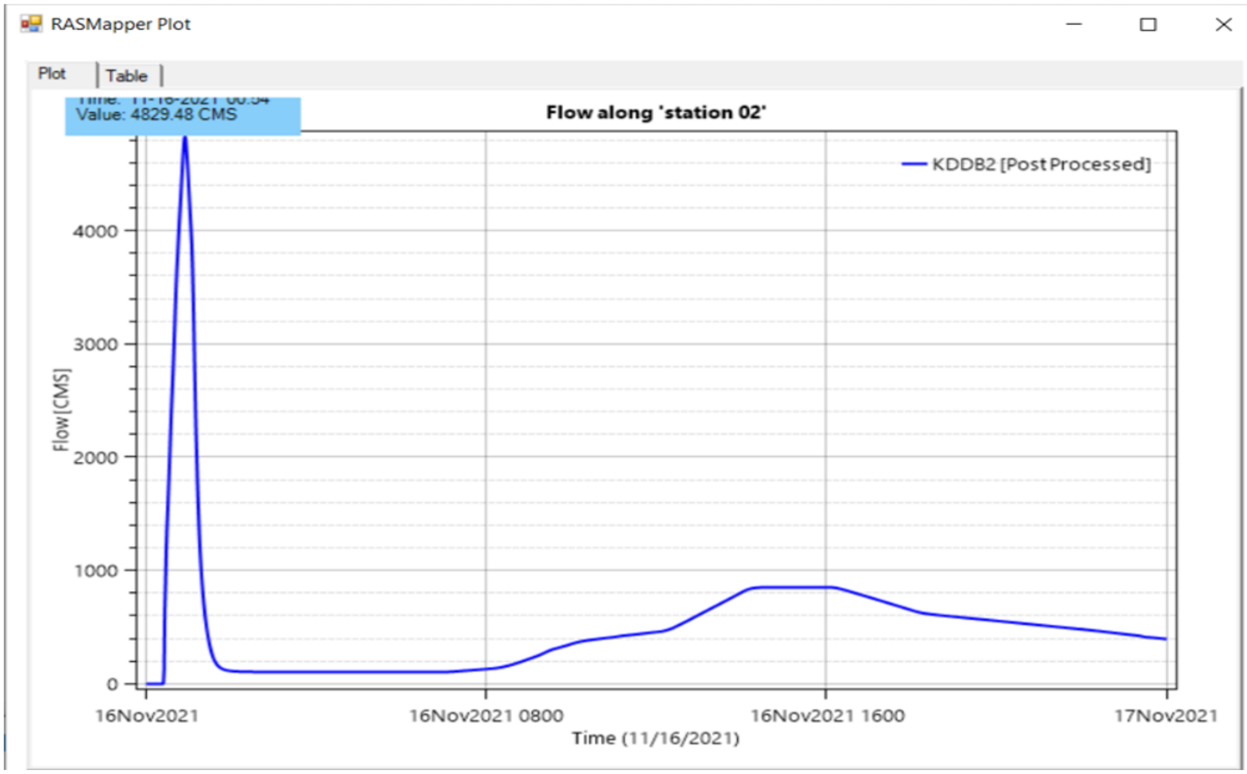
Appendix B. 5 Peak flows at station-02 TFH=0.6 and n=0.027



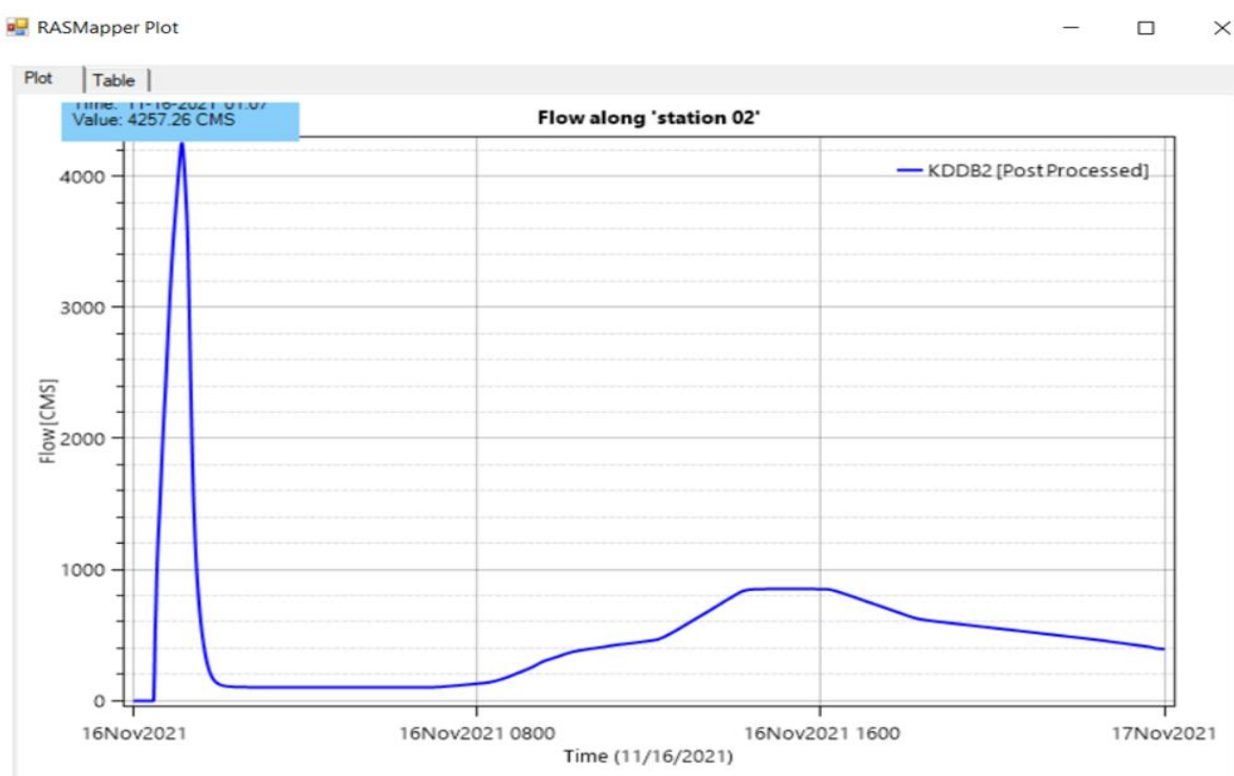
Appendix Figure B. 6 Peak flows at station-02 TFH=0.9 and n=0.0285



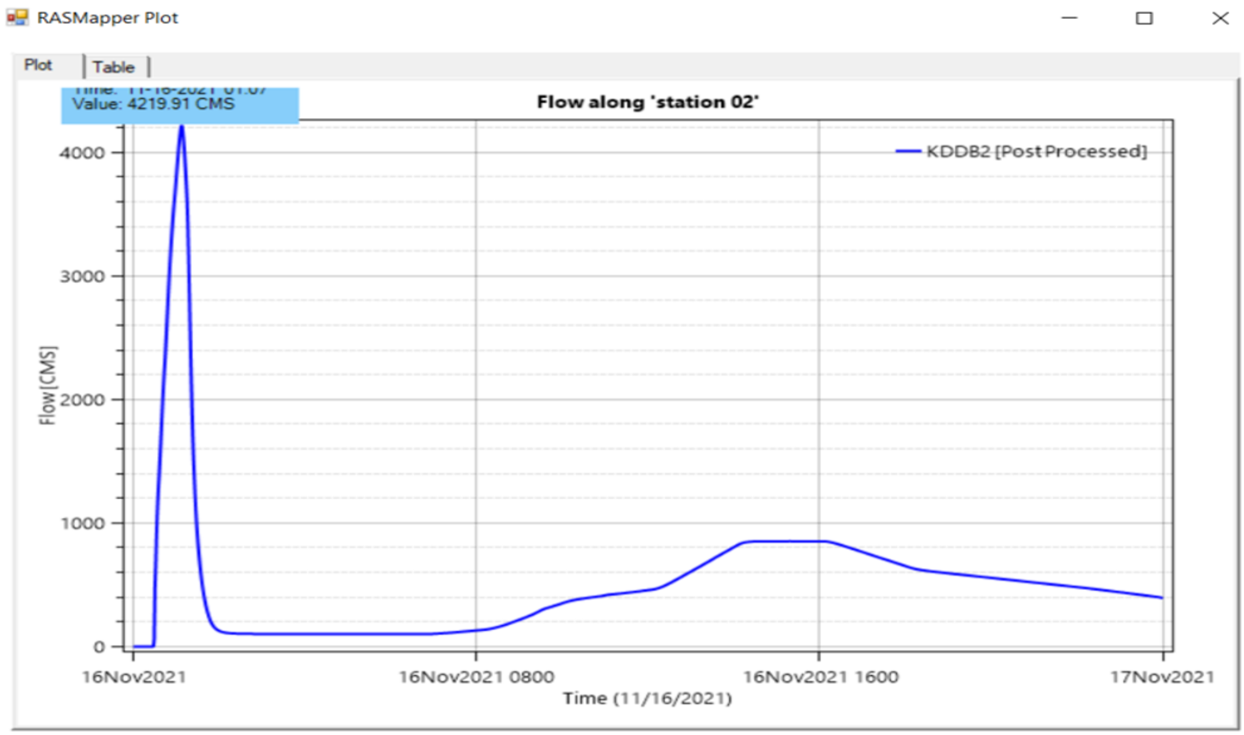
Appendix Figure B. 7 Peak flows at station-02 TFH=1.2 and n=0.03



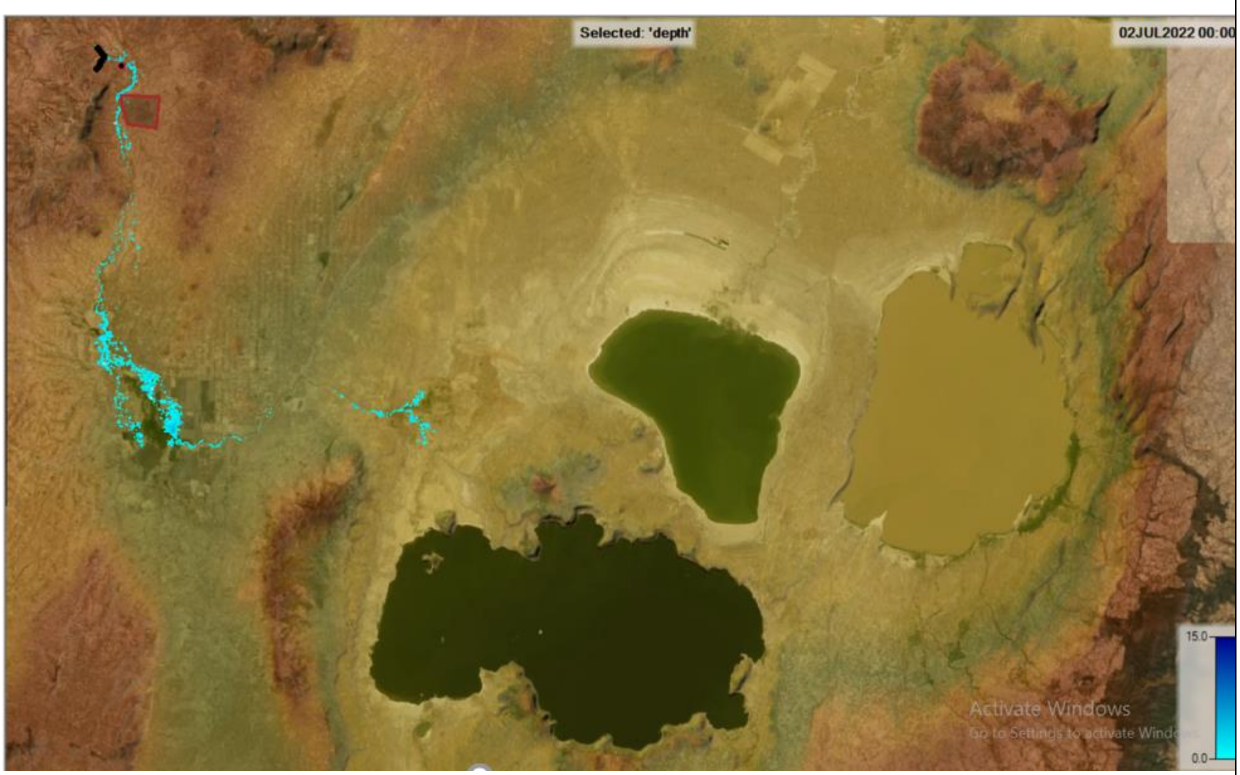
Appendix Figure B. 8 Peak flows at station 02 TFH=1.5 and n=0.0315



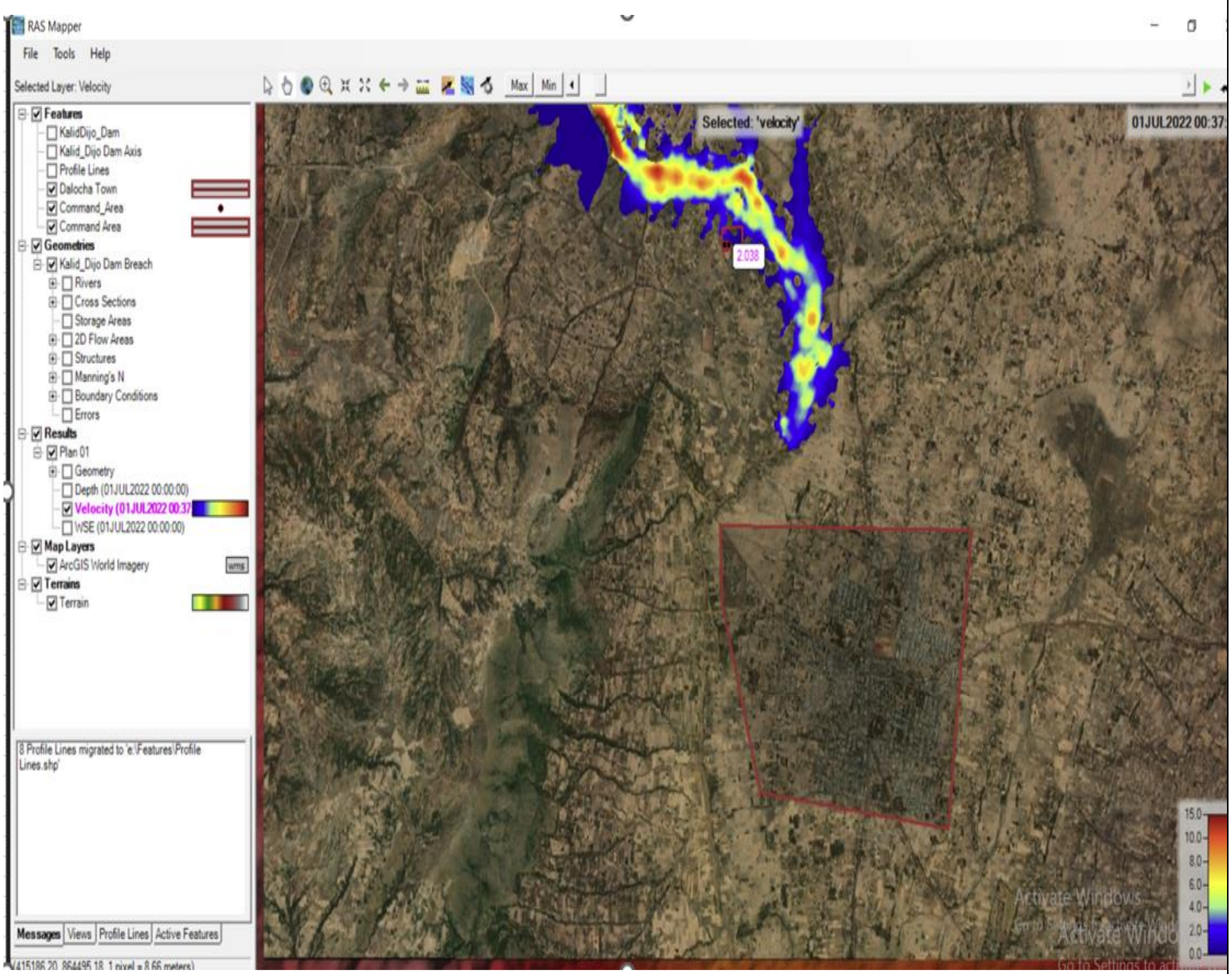
Appendix Figure B. 9 Peak flows at station 02 TFH=1.5 and n=0.033



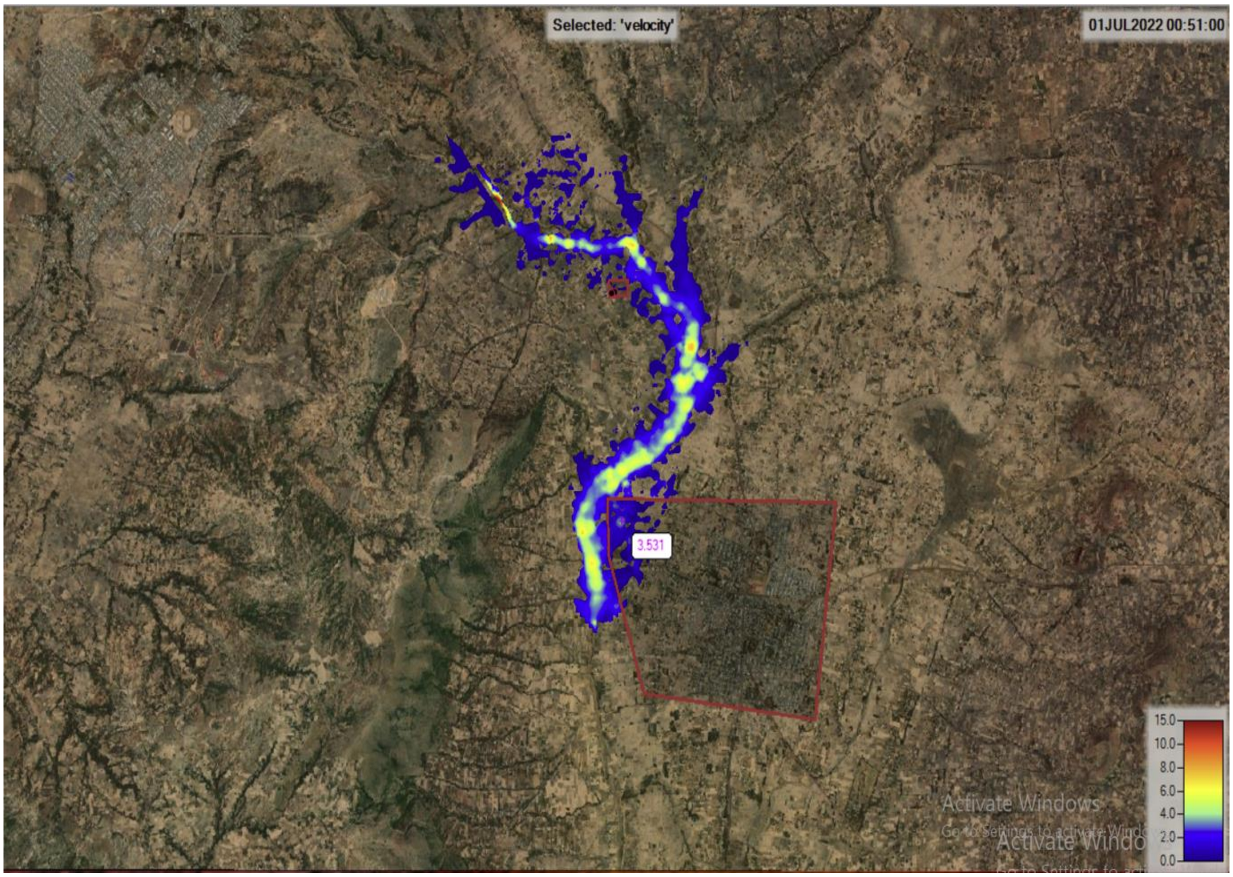
Appendix Figure B. 10 Depth profile



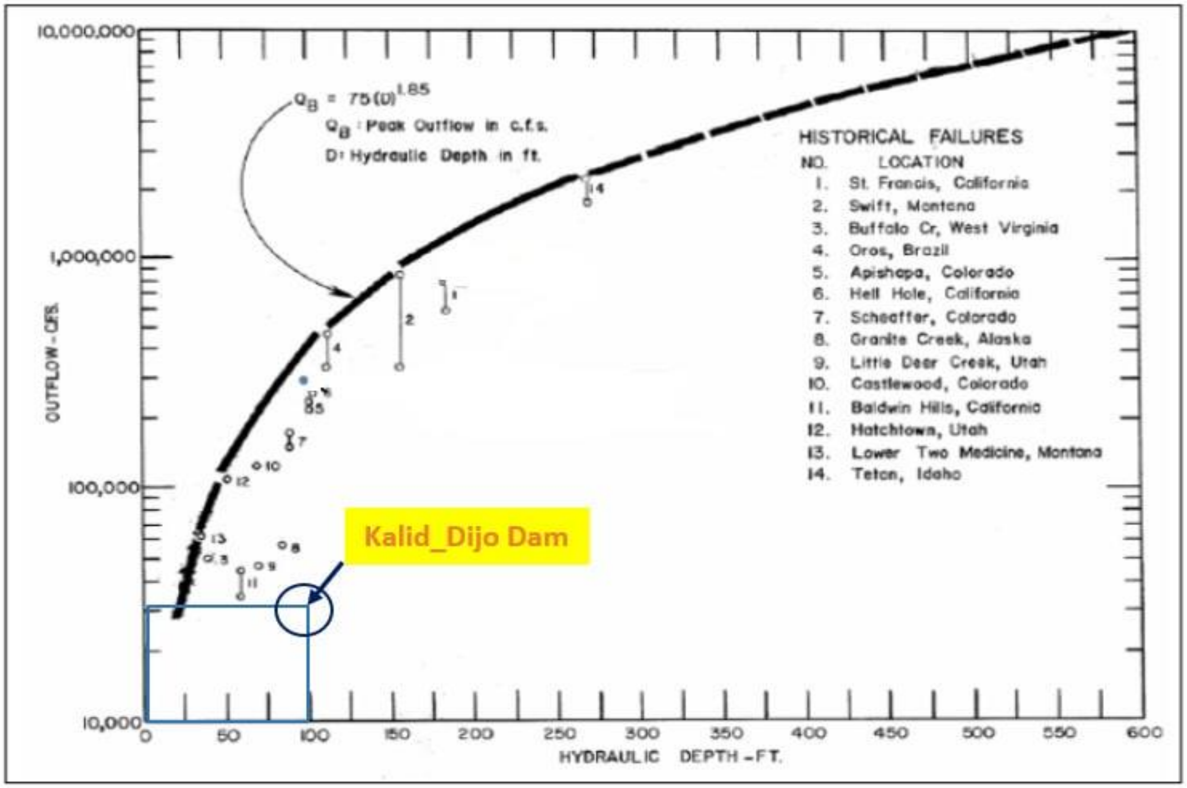
Appendix Figure B. 11 Downstream inundation velocity profile around command area



Appendix Figure B. 12 Downstream inundation velocity profile around Dalocha Town



Appendix Figure C. 1 Comparison with envelope curve (USACE, 2014)



Appendix Figure C. 2 Kalid-Dijo Dam Location

Appendix Table C. 1 Probable Maximum Flow (PMF)

Time(hr)	Inflow (m3/sec)
0	0.07
1	0.07
2	0.07
3	2.71
4	5.34
5	22.98
6	40.61
7	56.28
8	71.94
9	131.40
10	190.85
11	212.90
12	234.95
13	330.59
14	426.23
15	426.07
16	425.90
17	367.87
18	309.84
19	291.23
20	272.62
21	253.79
22	234.95
23	212.90
24	190.85

Appendix Table C. 2 Kalid-Dijo Dam Storage Capacity Vs. Elevation

S.No	Elevation (m a.s.l)	Reservoir Capacity (M^3)
1	1950	0.00
2	1951	113.33
3	1952	226.67
4	1953	340.00
5	1954	453.33
6	1955	566.67
7	1956	680.00
8	1957	793.33
9	1958	906.67
10	1959	1020.00
11	1960	1133.33
12	1961	1246.67
13	1962	1360.00
14	1963	1951.82
15	1964	2543.64
16	1965	3135.45
17	1966	3727.27
18	1967	4319.09
19	1968	4910.91
20	1969	5502.73
21	1970	6094.55
22	1971	6686.36
23	1972	7278.18
24	1973	7870.00
25	1974	8456.00
26	1975.1	9042.00
27	1976	9628.00
28	1977	10214.00
29	1978.1	10800.00