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ADDIS ABABA UNIVERSITY SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY (AAIT)

Dam Break Analysis (The case study of Fincha'a Dam)

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

Prepared by: Jiregna Nugusa

Advised by: Dr. Ing Asie Kemal

December, 2016

Approved By Board of Examiners

Dr. Ing Asie Kemal

(Advisor)

Signature

Dr. Yilma Seleshi

(Internal Examiner)

Signature

Dr. Mebruk Mohammed

(External Examiner)

Signature

Chairman (Department of Graduate Committee)

Signature

DECLARATION

I declare that I have elaborated M.Sc. thesis aimed at Dam Break Analysis (case stud of Fincha'a Dam) independently under the guidance of my adviser, Dr.Ing Asie Kemal. I have used the literature review and other reference material those listed in the references at the end of this work and other information sources that are cited in work.

October 2016

Signature -----

ABSTRACT

Fincha'a Dam was constructed in 1973 for the purpose of electricity generation, irrigation, fishery and tourism with 22.2m height and 340m crest length. Fincha'a dam is rock fill dam with uncontrolled overflow spillway. There are different economic developments downstream of the Fincha'a dam including hydropower plant, irrigation project, residential house and sugar factory. These economic developments are affected either dam break by overtopping or piping mode of failure.

For this thesis study the input data were collected from ministry of water, irrigation and energy and from ministry of electric utility of Ethiopia. The calculated breach parameters by Von Thun and Gillette Regression equation are selected as input data into the HEC-RAS model for dam break analysis. Some of geometric data like: reservoir coverage area, location of inline structure, stream line, banks and cross-section data were created from Digital elevation model by using integrated Arc-GIS and HEC-GeoRAS model.

The general objective of the study is to analyze Dam break by using hydraulic models (Hydraulic Engineering Center's River Analysis system). For this study the failure location is assumed to be at the center of the dam due to presence of high hydrostatic pressure and develop equally in both sides. From the result of Fincha'a dam break simulation the peak discharge formed by overtopping mode of failure is more devastating than the piping mode of failure. The effect of dam breach parameters on discharge is more pronounced than that of the water level. Dam break has greater impact on the downstream location where is closer to the dam in accordance with the hydrograph at downstream locations.

The created Fincha'a river network was exported to HEC-RAS model for further dam break analysis by addition of different geometric data including dam information, calculated breach parameters, initial condition, upstream and downstream unsteady boundary condition. After full computation the model result were exported to integrated Arc-GIS and HEC-GeoRAS model for mapping flood inundation.

Developed inundation map guides the dam owners and emergency management authority to give emergency action plan for the highly affected area by flooding and used for planning future economic development activities.

Keywords: Dam break, Analysis, HEC-RAS, Hydrograph, HEC-GeoRAS

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LIST OF ABBREVIATIONS

B_{ave}	Average breach width
C	Weir coefficient
DAMBK	Dam Break-Forecasting Model
DEM	Digital Elevation Model
EAP	Emergency Action Plan
FEMA	Federal Emergency Management Agency
FLDWAV	Flood wave dynamic modeling
IDF	Inflow design flood
GIS	Geographic Information System
H_b	Breach height
H_D	Height of the dam
HEC-GeoRAS	Hydraulic Engineering Center Georeferenced River Analysis System
HEC-RAS	Hydraulic Engineering Center's River Analysis System
h_w	Height of water
LOB	Left over bank
m	Meter
m^3	Cubic meter
m^3/s	Cubic meter per second
MWL	Maximum water level
N	Manning roughness coefficient
N_1	Manning roughness coefficient of LOB
N_2	Manning roughness coefficient of channel
N_3	Manning roughness coefficient of ROB
NWS	National Weather Service
OTM	Overtopping mode
PM	Piping mode

PMF	Probable maximum flood
Q_p	Peak out flow
ROB	Right over bank
SMPDBK	Simplified Dam Break Model
t_f	Time of failure
USACE	United States of Army Corps of Engineering
W_b	Breach bottom width
WSE	Water surface elevation
Z	Side slope factor

1 INTRODUCTION

1.1 Background

Dams have been constructed for thousands of years around the world for different purposes: flood control, electricity generation, irrigation, water supply, recreation and etc (Tefera, 2006). But also, hundreds of dams have failed due to high flows in the river, sea, etc. In India the worst dam disaster occurred in Machhu II dam was constructed to serve an irrigation scheme (Gujarat, 1972-1979). This dam failed because of excess flood, inadequate capacity of spillway and due to overtopping of water over embankment dam in August 1, 1979. Kaddam Dam was failed due to overtopping of water above the crest by 46cm and due to this mode of failure 137.2m of breach width has been developed on the left bank in August 1958 (Andhra Pradesh). The world's worst dam failure "Banqiao Dam and the Shimantan Dam" occurred due to the overtopping in August 1975 and around 85,000 peoples were died by flooding in China (Sachin, 2014). However, we have to give deep attention for breaking of embankment dams comparing with other types of dams because the floods resulting from the failure of constructed dams produced some of the most devastating disaster of the last two centuries (Xiong, 2011). Simulation of dam break result and resulting floods are essential for characterizing and reducing negative effects occurred on the downstream area. Development of emergency action plan requires exact estimation of inundation level and the arrival time of flood wave at the downstream point. The breakage frequencies of earth dams are almost four times greater than concrete dams or masonries (Shahraki et al., 2012).

Dam failure results due to external force and internal erosion. The USACE Hydrologic Engineering center (HEC) research document lists 13 dam failure causes as: 1) earth quake 2) land slide 3) extreme storm 4) piping 5) equipment malfunctioning 6) structure Damage 7) foundation failure and Sabotage and etc (Xiong, 2011). Breach is defined as the opening formed in the dam body that causes the water to spread to the downstream location (Xiong, 2011). Different Case studies show that dam failure may arise due to different reasons ranging from seepage, piping (internal erosion), overtopping due to insufficient free board and settlement due to side slopes on the upstream shells and liquifaction due to earthquakes.

Piping is a flow of water in porous parts of the dam especially through high permeability regions, cavities, fissures or strata of sand and gravel. Such concentrated flow at hydraulic gradient may erode the soil part of the dam which causes the breakage of the earth dam. Piping through the dam body is caused by: faulty construction, insufficient compaction, cracks in the embankment due to foundation settlement, animal borrows pipes and conduits inside the dam body.

Overtopping is defined as uncontrolled flow of water over the crest of the dam. Overtopping may lead to failure of the dam due to excessive erosion of downstream slope. The main causes of overtopping are: under estimation of the design flood and inadequate spillway capacity, large and rapid landslides in the reservoir, insufficient free board and malfunctioning of the spillway gates. (Jabir, 2013). The three primary tasks in Dam Break Analysis are: estimation of dam break out flow hydrograph, routing of dam break hydrograph through downstream valley, estimation of inundation levels and damages to downstream structures. Many models have been developed before few years for dam break analysis such as DAMBRK, FLDWAV, SMPDBK, and HEC-RAS Model etc (Altinakar, 2008).

1.2 Statement of problem

Dam break can result in huge damages to downstream areas of the dam. Embankment dam breach may be caused by overtopping or piping mode of failure. Fincha'a dam will cross the Fincha'a River between two basalt abutments. Due to construction of Fincha'a dam, more than 3,115 households were resettled downstream of the dam near to the river. The average household size of one household is eight (8) people, which made an estimate of 22,250 people were resettled downstream of the dam during Fincha'a dam construction (Tefera, 2006). In 1987 Amarti dam is constructed and diverted to Fincha'a reservoir which dramatically increased the number of resettled people.

Due to lack of understanding of hydrological parameters, capacity of the reservoir and spillway, the embankment dam breach is the most challenging in most countries in the world. Fincha'a dam is rock fill dam, which was constructed by Impresit Rechi contractor, Italy. There are different economic developments downstream of the Fincha'a dam such as hydropower plant, irrigation project, residential house and sugar factory; these activities can be affected if the dam breaks (Ministry of water, irrigation and electricity, 1973).

There is probable loss of human lives, houses, buildings and other interest economic activities if dam failure occurs. Therefore, we have to give deep attention for breaking of embankment dams; because floods resulting from the dam breach produce huge damages to the downstream areas.

The outer coverage of Fincha'a dam is rock shell from the upstream and downstream sides of the dam and its core part is constructed from impervious earth and there is transition zone between internal and outer part of the dam.

Since Fincha'a dam is embankment dam there is probability of occurrence of dam break due to overtopping and internal erosion. Therefore, it is sensitive to analysis the downstream damages caused by the dam break and recommend methods which can be used to reduce its impact considering volume of water, water level, velocity and arrival time for flood wave from dam to downstream point.

1.3 Importance of the study

Studying of the Fincha'a dam break analysis helps to give effective and urgent action plan that requires accurate prediction of inundation levels. Thus the results of the study are important for the following reasons:

- ❖ The research result can be used to inform the flood prone area coverage to the downstream population if the dam breaks.
- ❖ The research result can be used to notify the effect of dam break on the human life, irrigated area, constructed infrastructures and other developed economic activity.
- ❖ The research outcome can be used to develop emergency action plan for Fincha'a dam based on the flood inundation map developed and
- ❖ The research finding will support other researchers to do analysis of similar dam break.

1.4 Objective of the study

1.4.1 General objective

The general objective of the study is to analyze the Dam break case study of Fincha'a dam using hydraulic models.

1.4.2 Specific objective

The specific objectives of this study are:

- To determine parameters of the dam break (breach size and time of failure);
- To determine peak outflow hydrograph when dam break occurs by either overtopping or piping mode of failure;
- To estimate the hydraulic conditions (water level, top width and arrival time of flood wave) at critical downstream locations and
- To inundate flood-prone areas of downstream area due to dam failure.

1.5 Scope of the study

The scope of the study is limited to 42 km downstream of the Fincha'a Dam, which is far distance from the downstream economic development activities and population settlements. In this study the analysis is proposed on the prediction of outflow hydrograph of dam break due to overtopping and piping mode of failure. HEC-RAS model is used to estimate peak discharge, water level, top width of water, arrival time of flood waves, flood velocity, etc for downstream area after dam break.

1.6 Limitation of the study

The problem faced through the study of this thesis is lack of primary data. When I collect secondary data the problem I come across are: Some of the government employees are not interested to give the documented data and full documented data is not available in their office.

1.7 Organization of the paper

The research is organized into five chapters. The first chapter is the introductory part which discusses the overall objective of the research, problem statement of research, scope of the study and limitation of the study. The second chapter discusses the literature review of the research. The third chapter discusses the methodology used to conduct the research. The dam break result and discussions are discussed in chapter four. The fifth chapter concludes the study with the points of recommendation. The list of references and appendix are annexed.

2 LITERATURE REVIEW

2.1 Introduction

Dam break analyses are the most required research to protect the downstream area of the dam from the risk due to dam breach. In Ethiopia the function of dam break analysis is not known clearly. In Ethiopia more than 85% of the dam haven't flood inundation map due to dam break. This shows that the effect of flood resulted from the dam break on the downstream area is not clearly understood in Ethiopia.

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).

Dam break is a complicated and comprehensive process and its actual failure mechanics are not well understood. Neither current physical based models nor empirical models could fully explain dam break mechanisms and impacts (Xiong, 2011).

The SMPDBK model was developed by National Weather Service (NWS) in 1984, and last updated for public use in 1991, to predict downstream flooding produced by a dam breach. It is a model which uses simple method to model dam breach. It can easily be used on a personal computer with a minimal amount of data. SMPDBK can produce approximate flood forecasts after adding only the dam height, reservoir storage volume, and depth versus width area for one cross-section of the downstream river valley (FEMA, 2013).

HEC-RAS can be used to compute and import a breach hydrograph and route the hydrograph using a steady or unsteady state solution. HEC-RAS can be used to route an inflowing hydrograph through reservoir with any of the three methods: one dimensional unsteady flow routing (full Saint Venant equations), two dimensional unsteady flow routing (full Saint Venant equations) and with level pool routing. But full unsteady flow routing (one or two dimensional) will be more accurate for both with and without breach (Gray, 2014).

In order to check the accuracy of existing breach prediction models in predicting the breach geometry, breach formation time and peak breach discharge, a geometric survey after the dam failure has been carried out to obtain the geometry (Abdulrahman, 2014).

2.2 Dam breach parameters

Breach parameter determination is fundamental to dam breach analysis. Typically, dam breach parameters are divided into two groups: geometric and hydrographic parameters. The form of embankment dam break is almost trapezoidal shape. Dam breach parameters define the development of geometric parameters of breach depth (h_b), height of water (h_w), top breach width (B_t), average breach width (B_{ave}), bottom breach width (W_b) and breach side slope factor (Z). Hydrographic parameters include peak out flow (Q_p) and failure time (t_f).

2.3 Importance of breach parameters

Variation of breach parameters can affect peak discharge and inundation levels. Change in breach formation time and breach width can produce large change in peak outflow, whereas breach height has relatively small effect on peak outflow. For locations downstream of the dam, timing of the flood wave peak can change significantly with changes in breach formation time, but peak discharge and inundation levels are insensitive to change in breach parameters (Wahl, 1998). In general, the importance of the breach parameters will become less significant as the location of interest point move farther downstream of the dam.

2.4 Dam breach estimation

A key element for calculating a dam breach hydrograph for specific dam involves estimating the dam breach parameters for dam breach modeling related to the geometry and timing (width, depth, shape, and time of failure) of the breach formation. It has been noted by several sources

that the selection of breach parameters are based on dam type, dam dimension, and dam materials of construction.

A number of methods are available for estimating breach parameters for use in dam breach studies. Since the selection of the breach parameters is specific to each dam, guidance is provided describing methods currently applied by dam safety professionals without recommending a standardized method (FEMA, 2013).

The three most widely used and accepted empirically derived enveloping curves and/or equations for predicting breach parameters for rock fill dam are: MacDonald and Langridge-Monopolis (1984), Froehlich (1995a, 1995b, 2008), and Von Thun and Gillette (1990) (Colorado Dam Safety Branch, 2010). MacDonald and Langridge-Monopolis (1984), Von Thun and Gillette (1990) and Froehlich (1987, 1995a, 1995b, 2008) methods are applied on the earth-fill and rock-fill embankment dam. Von Thun and Gillette (1990) and Dewey and Gillette (1993) used the data from Froehlich (1987) and MacDonald and Langridge-Monopolis (1984) to develop guidance for estimating breach side slopes, breach width at mid-height, and time to failure (Wahl, 1998). Published guidelines of breach width, side slopes, and development time for earth fill/ rock fill dam is shown in table 2-1.

Table 2-1: Ranges of possible values for breach characteristics (Gee, 2008)

Dam type	Average Breach width (Bave)	Horizontal component of breach side slope (H), (H:1V)	Failure time, Tf, (hrs)	Agency
Earth fill/	(0.5 to 3.0)xHD	0 to 1.0	0.5 to 4.0	USACE (1980)
Rock fill	(0.5 to 5.0)xHD	0 to 1.0	0.1 to 4.0	USACE(2007)
	(1.0 to 5.0)xHD	0 to 1.0	0.1 to 1.0	FERC(1988)
	(2.0 to 5.0)xHD	0 to1.0 (slightly larger)	0.1 to1.0	NWS(Fread,2006)

2.4.1 Breach parameter definition

Breach depth (Breach height) - Breach depth is the vertical distance between dam crest and bed width. But some reference indicates it as the vertical distance between the upper level of water storage and bed width.

Breach width - Breach width is breach average width or breach width at top and bottom of the broken area. The final breach width and its expansion rate have a great effect on flow velocity and out flow to the downstream area.

Breach side slope factor (zH: 1V) - Side slope determines the whole dam breach with breach width and depth. Exact determination of side slope is less important than determination of breach width and depth.

Breach formation time –The duration of time between the first breaching of the upstream face of the dam (breach initiation) and when the breach has reached its full geometry. It includes breach initiation and breach formation.

Breach initiation time begins from the first flowing above or from inside the dam. The initiation time is an alarm to empty the hazardous areas. It ends at the start of the breach formation phase.

According to DAMBRK Model, the break formation time is the interval between formation of the first crack and completion of break. When the break reaches its final size, the break formation time is completed.

2.5 Breach mechanisms for embankment dams

Embankment dam breach may occur due to different reasons; most commonly breaches in embankment dams are modeled as overtopping or piping failures (FEMA, 2013).

2.5.1 Overtopping failure

When uncontrolled flow of water passes over the embankment dam's crest, the dam gradually washed away (eroded). Overtopping failure of embankment dams typically begin with head cutting at the downstream toe and developed upstream until the erosion reaches the dam crest and reservoir surface. According to a study by Ralston (1987), a small head cut typically forms on the downstream face of a cohesive soil embankment and progresses upstream as shown in figure 2.1. Dam overtopping is very difficult process to understand, predict, analyze, or model due to the complexity and contextual nature of the overtopping process (Steininger, 2014).

The breach is considered to begin when erosion occurs across the width of dam crest. Overtopping usually results from a design inadequacy of the dam/spillway system and reservoir

storage capacity to handle the resulting flooding event. It is the most common mode of failure for embankment dams.

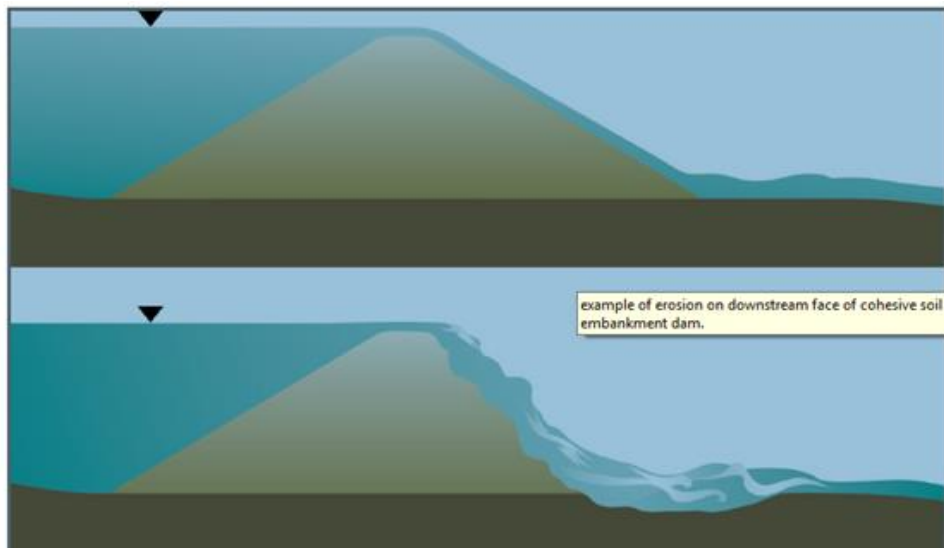


Figure 2-1: Erosion on the downstream face of Embankment dam (FEMA, 2013)

The breach may stop growing when the reservoir is emptied and there is no more water to erode the dam. The breach progression may be modeled as a linear progression or a sine wave progression. Linear progression is a breach in which rate of erosion remains the same for the duration of erosion development. Sine wave progression is breach grows very slowly at the beginning and end of development and grows rapidly in between. Generally the trapezoidal breach progression is shown as follows in figure 2.2.

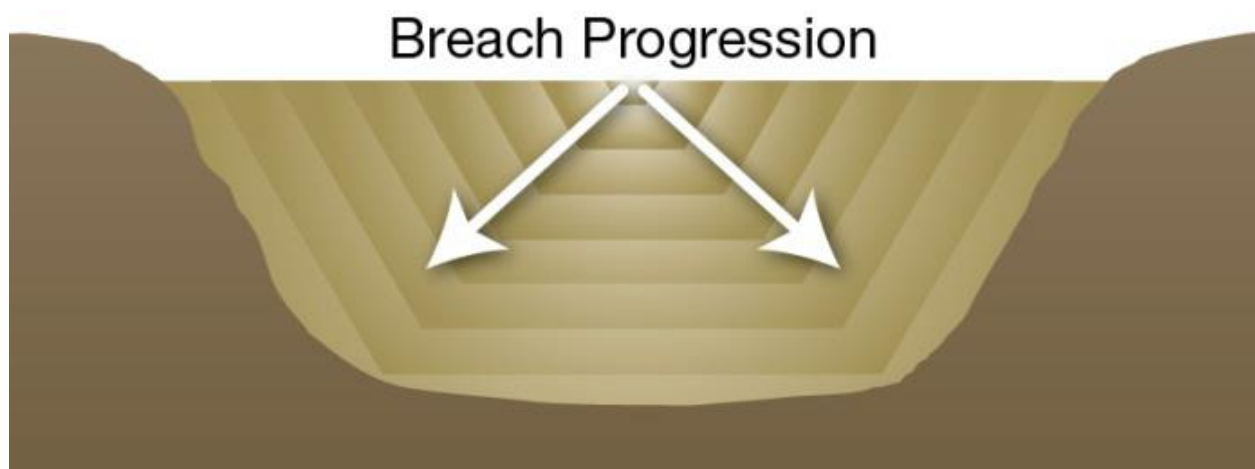
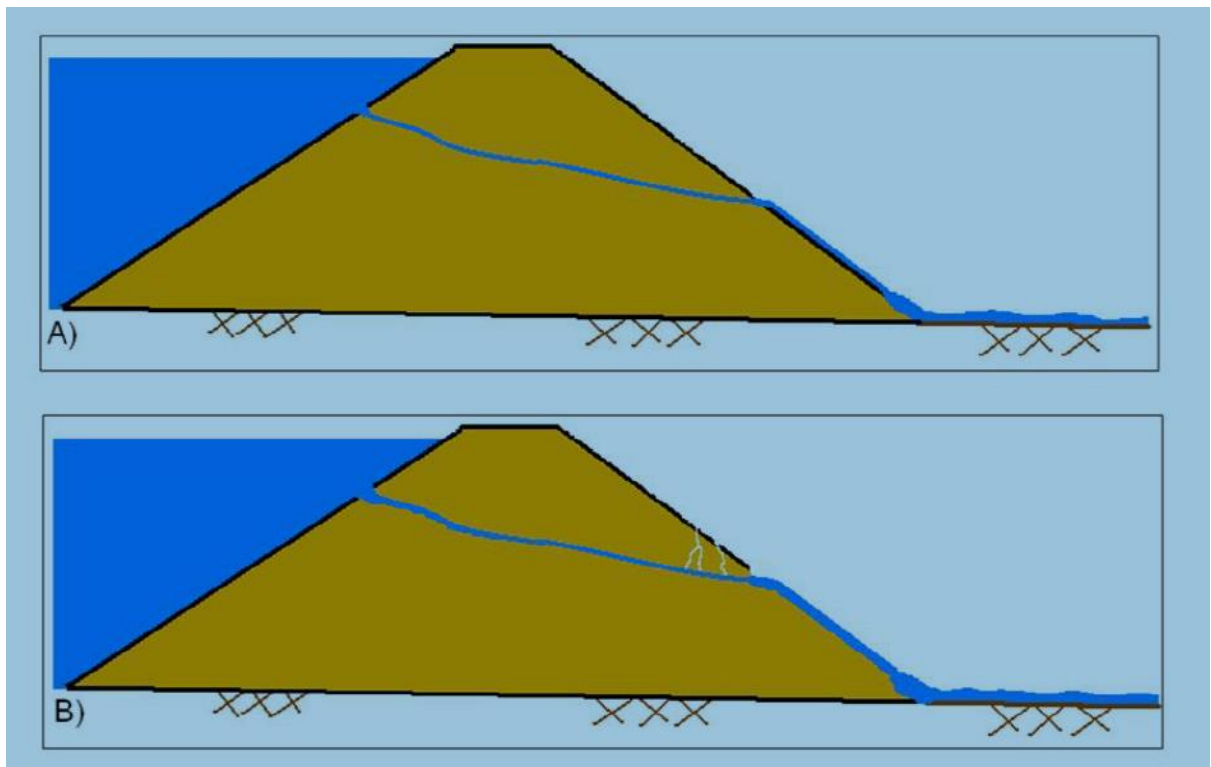


Figure 2-2: Overtopping trapezoidal breach progression (FEMA, 2013)

2.5.2 Piping and internal erosion failures

Piping and internal erosion occurs when concentrated seepage develops within an embankment dam. Piping failure mode can start at any elevation/location and grows to maximum extents because it is based on the reservoir water level which is available always behind the dam. The terms “piping” and “internal erosion” are often used synonymously. The seepage slowly erodes the dam, leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and continues its action towards the upstream. As the hole of piping increases the capacity of eroded material transport is increased. Depending on the volume of water behind the dam, the breach may continue to cut down and widen until the natural channel bed is reached (Gray, 2014). Internal erosion occurs when the water flowing through the crack erodes the soil from the walls of the crack. If the eroding water has enough velocity to continue to erode the soil in contact with the crack, the crack will enlarge. Both internal erosion and piping will progress until the flow path is large enough to empty the reservoir (Colorado Dam Safety Branch, 2010). Therefore, the continuous breach process for a piping failure can be shown schematically on figure 2.4 as follows.



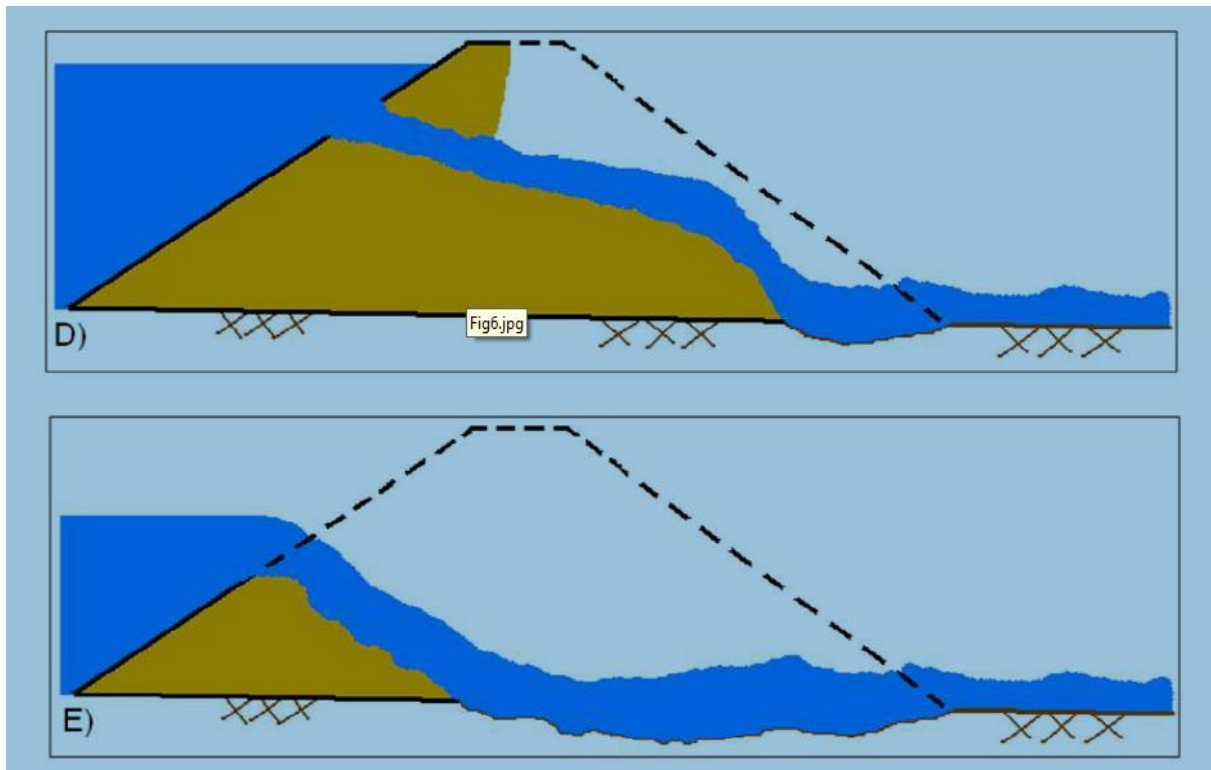


Figure 2-3: Breach process for a piping failure (FEMA, 2013)

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 (Colorado Dam Safety Branch, 2010).

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. Figure 2-4 shows a schematic of a fully formed piping hole.

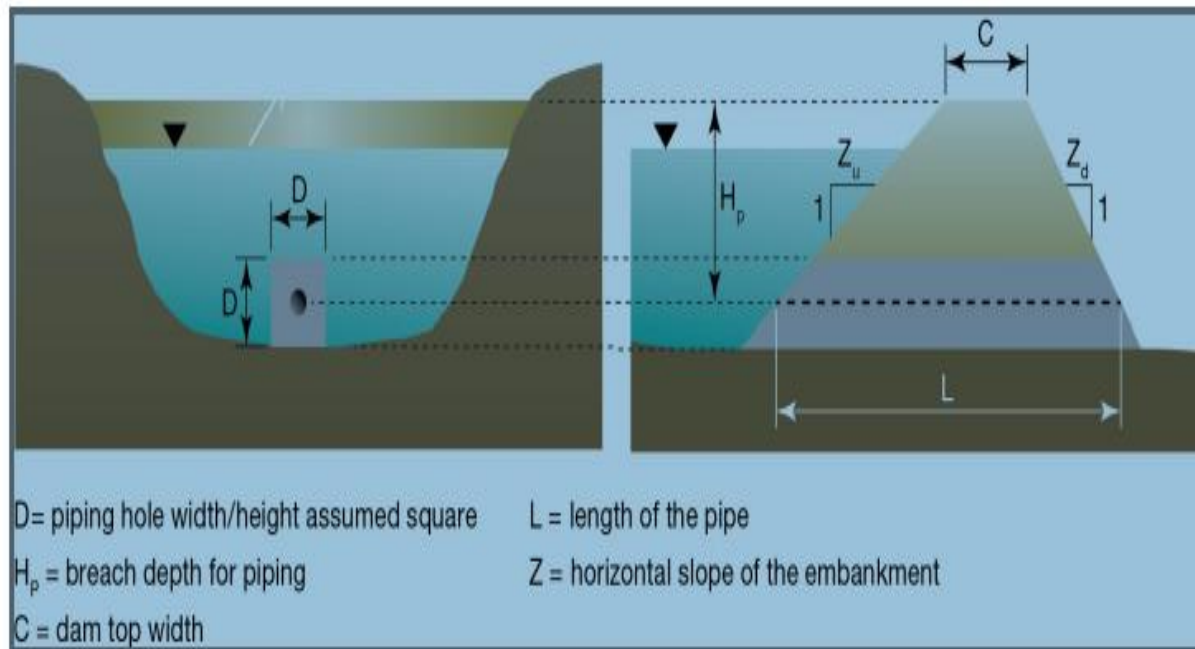


Figure 2-4: Piping hole (Colorado Dam Safety Branch, 2010)

2.6 Dam breach analysis approach

The two most commonly used approaches for dam breach analyses are: event-based approach and a risk-based approach.

2.6.1 Event based approach

An event based approach is deterministic method. It uses specific precipitation and non-precipitation events for dam break analysis and downstream inundation mapping. Event based approach include extreme rainfall and runoff events that result different magnitude of natural floods.

Several hydrologic and non-hydrologic (fair weather) events are evaluated as part of an event-based dam safety analysis. For hydrologic failure events, an extreme flood event ranging from the 50-year event for low-hazard dams up to the PMF for high-hazard dams is selected based on the potential for loss of life due to a dam failure or for significant economic and environmental losses (FEMA, 2013). The advantages of event-based approach over risk-based approach is that it is a direct approach, less complicated to perform and regulate, and produce more conservative breach inundation zone mapping.

2.6.1.1 Fair weather (non-hydrologic) failure

A fair weather (sunny day) breach is a dam failure that occurs during fair weather (i.e. non-hydrologic or non-precipitation) conditions. The 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 (FEMA, 2013).

According to FEMA-2013, the three most common initial water level elevations for fair weather breach analysis are:

- ✓ Normal Pool Elevation (invert of the highest elevation of the primary outlet)
- ✓ Invert of Auxiliary spillway (lowest uncontrolled spillway)
- ✓ Top of Dam/Maximum High pool

2.6.1.1.1 Normal pool elevation (invert of the highest elevation of the primary outlet)

A breach at this elevation of the reservoir is used to estimate the volume and breach discharge that resulted from a failure event during fair weather condition. This type of event is modeled as piping/internal erosion failure and a monolith collapse resulting from sliding, foundation instabilities (a seismic event) for an embankment dam and concrete dam respectively.

2.6.1.1.2 Invert of auxiliary spillway (lowest uncontrolled spillway)

A breach of the dam at this reservoir water level is common practice to simulate a breach during misoperation of the primary outlet works. Typically initiation of dam failure is the same as that of the reservoir level at normal pool.

2.6.1.1.3 Top of dam/maximum high pool

This reservoir level represents the maximum amount of volume of water that stored in the reservoir. This condition is may be selected to evaluate the most conservative non-hydrologic event (dams with non-adequate spillways or pump storage facilities). A breach event at this elevation of the reservoir may be modeled as a piping/internal erosion failure or as an overtopping failure with water level just top of the dam.

2.6.1.2 Hydrologic failure

A hydrologic (rainy day) breach is dam failure that occurs during extreme precipitation and runoff. Hydrologic failures that cause dam breach events are generally analyzed based on the IDF established by the dam's hazard potential and hazard size classification. Typically a PMF is for high-hazard potential dams and a range from one-percent annual-chance flood event (often called the 100-year flood) to the percentage of the PMF is for significant hazard potential dams.

2.6.2 Risk-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 based on the potential loss of life or increase in economic losses caused by a potential dam failure.

2.7 Hazard potential classification

Hazard potential is classified based on loss of different thing as follows (FEMA, 2013).

a) High hazard potential

Dams classified under high-hazard potential are those where break or misoperation will probably cause loss of human life.

b) Significant hazard potential

Dams classified under significant hazard potential are where break or misoperation are not result in loss of human life but may cause economic loss, environmental damages, or impact of other concerns. It is located in predominantly rural or agricultural areas.

c) Low-hazard potential

Dams classified under low - hazard potential are where break or misoperation are not result in loss of human life and only low economic and/or environmental losses. It is principally limited to the owner's property.

2.8 Dam failure inundation map

Dam failure inundation map is a map showing the area downstream of the dam that would reasonably be expected to be flooded in the event of a failure of the dam. Dam failure inundation

maps are required to classify dams as high hazard and significant hazard (Colorado Dam Safety Branch, 2010).

2.8.1 Uses of inundation mapping

Inundation maps can have a different uses including emergency action plans, mitigation planning, emergency response, and consequence assessment (FEMA, 2013).

2.8.1.1 Emergency action plans (EAP)

An Emergency Action Plan is used to identify potential emergency conditions at a dam and specifies preplanned actions to minimize property damage and loss of life. The downstream inundation map is the basis for developing Emergency Action Plan and used to show the emergency management authorities and the critical areas for action in case of an emergency (Safety, 2009).

2.8.1.2 Emergency response

Emergency response represents the actions taken in the 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 to minimize the consequence of actual dam failure or incident.

2.8.1.3 Hazard mitigation planning

Mitigation is the proactive effort to reduce loss of life and property by understanding the effect of disasters. This is achieved through identifying hazard potentials and the risks they pose in a given area. In the case of dam failure, hazard mitigation planning involves identifying the population at risk and identifying the action to reduce their vulnerability.

Information required by hazard mitigation planners includes the breach inundation zone boundary, depth of flooding, velocity and timing.

2.8.1.4 Dam breach consequence assessment

Dam breach consequence assessment includes identifying and quantifying the potential consequence of a dam failure or incident. While hazard mitigation planning focuses on the

specific projects to reduce flood risk, consequence assessment focuses on the economic and social impacts of a potential disaster and the organizational and government action needed in the result of a dam breach to respond and recovery.

2.8.2 Mapping guidance

2.8.2.1 Recommended inundation map elements

I. Map collar information

The latitude and longitude coordinates are can be referenced at the corners of the neatline. The other important information displayed on the map collar can be horizontal reference grid ticks to help orient map users to real world coordinates. Adjacent map panel numbers should also be listed along the neatline borders that correspond with adjacent map panels.

II. Base map data

Base map data provide the background from which inundation hazard information is overlaid and interpreted. Clear, easy to interpret base maps are critical for the effective use of an inundation map.

III. Inundation polygons

Inundation polygons are used to define the horizontal limits of the inundated area for one or more breach events. The inundation polygon shows the intersection of the peak water surface elevations from the dam break model with the ground elevation with the terrain surface. If multiple breach events will be shown on the inundation map, the polygon representing the event that would result in smallest inundation area should be displayed on the top of those representing the events with larger inundation areas.

IV. Inundation elevation

Inundation elevation can be explained at key locations along the inundation polygon if desired and extracted directly from dam break model. Elevations are not always a critical element for an inundation map. Elevation may be important for flood warning, if early warnings are possible.

V. Flood wave arrival time

Flood wave arrival times can be explained at key locations along the inundation polygons if desired. The flood wave arrival time is the time from dam breach initiation until the leading edge of the inundation arrives at specific location. This is typically determined by inspecting the stage (time versus stage) or flow (time versus flow) hydrograph plots of interest. Points of interest typically include road crossings, significant population at risk, and critical infrastructure. The arrival time is often characterized by a sharp increase in the flow (FEMA, 2013).

For a fair weather failure, the arrival time can be considered the first time that a notable change in the base flow is observed.

For a hydrologic failure event, the arrival time is best determined by comparing two simulations for the same hydrologic event. The first simulation would be a non-hydrologic event, while the second simulation would be the exact same hydrologic event with the dam breaching. The downstream hydrograph of both events can be overlaid to identify what time the effects of the dam breach would be first observed. The separation of the two hydrographs at the point of interest indicates the effects of the dam breach at that location. The arrival time for hydrologic events is normally defined as the time lapse from breach initiation until the differential stage for with or without failure simulation for the river or creek to exceed a defined depth. The recommended intervals for flood arrival times that should be include in the inundation map can be shown in table 2-2.

Table 2-2: Time intervals to include on inundation maps (FEMA, 2013)

Time after breach	Mapped arrival time intervals
0-30 minute	5 minute
30-90 minute	10 minute

3 METHODOLOGY

3.1 Study Area

3.1.1 Fincha'a dam and its location

Fincha'a River is one of the tributary of Blue Nile River originating in the high plateau of Ethiopia. The Fincha'a dam is Rock fill dam and which is constructed on a plateau creating a Reservoir with volume of 406Mm^3 at a full reservoir level of elevation 2220m (m.a.s.l). Fincha'a dam is constructed in 1973 E.C with uncontrolled overflow spillway which provided in the left abutment of the dam. In 1987, Amarti River is diverted to Fincha'a reservoir through a tunnel to fulfill the requirement of electricity generation (Sterk B. T., 2008). The spillway is designed to handle reservoir overflow resulting from the occurrence of the excess flood. During construction of Amarti Diversion Project, the crest elevation of the uncontrolled spillway has been raised by 2m. Providing an additional storage capacity of about 219Mm^3 through tunnel, the total storage capacity of the Reservoir becomes 625Mm^3 at elevation of 2222m (m.a.s.l). In the Harza Feasibility report on the Fincha'a power development, the peak spillway discharge is 213 cumecs. The climate of Fincha'a watershed is tropical highland monsoon with an average annual rainfall of 1604mm. Most of the rain falls during the month of June to September. The crest length of the Fincha'a dam is 340m with a maximum height, top width and crest elevation 22.2m, 7m and 2225m (m.a.s.l) respectively. The bed level of reservoir is 2207m from mean sea level. Its Reservoir area at 2222m (m.a.s.l) is 170 km^2 . This dam is constructed for hydropower, irrigation, fishery and tourism purposes (Sterk B. T., 2008).

The Fincha'a River Basin is bordered on the north by Abbay River, on the east by the Guder River Basin, on the south- west by the Awash River Basin, and on the west by the Hanger River basin. The Fincha'a dam is located 300km North-West of Addis Ababa in Oromia region, Horro Guduru Wollega zone, Abbay Choman Woreda, Ethiopia, between latitudes $9^{\circ}10'05''$ N to $10^{\circ}00'59''$ N and longitudes $37^{\circ}00'16''$ E to $37^{\circ}33'20''$ E. Fincha'a hydro- electric power plant is located on Fincha'a River about 3.523km downstream of the dam.

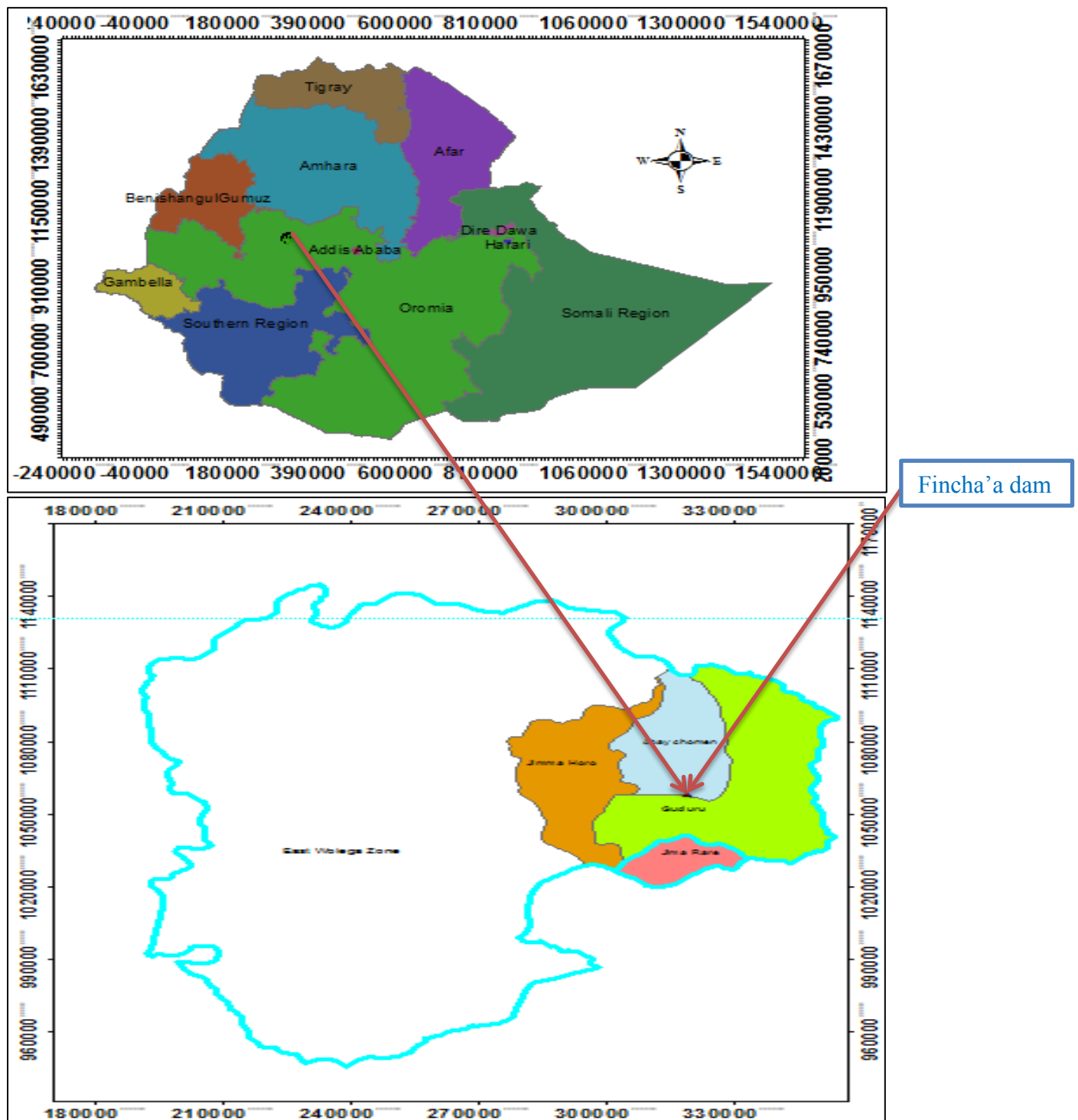


Figure 3-1: Location of Fincha'a Dam

3.2 Methods of dam break analysis

3.2.1 General

The most commonly used methods to determine the required breach elements for dam break analysis are: Comparative analysis, Empirical methods, Physically- based models, and parametric models. For this thesis study empirical methods and hydraulic models are used for the dam break analysis. Hydraulic model (HEC-RAS) is characterized under parametric models and the most commonly used for steady and unsteady flow analysis.

Generally the potential of risk formed due to dam break are classified into four including very high risk, high risk, average risk and low risk based on arrival time, speed of flood, depth of flood and width of flood.

3.2.2 Data collection

The required data were collected from different institutions. The hydrological data is collected from hydrology department of Ministry of Water, Mineral and Electricity of Ethiopia. The Metrological data is collected from Ministry of Metrological central Agency of Ethiopia. Physical characteristic data of dam (top width, height, crest length, crest elevation and etc), hydropower information and other information will be received from Ministry of Ethiopian Electric Utility. The GIS (DEM) data is received from GIS department of Ministry of Water, Mineral and Energy of Ethiopia. Image map is collected from Ethiopian mapping agency.

3.2.2.1 General information

Different data's are required to analyses the effect of dam break on the downstream areas during dam failure. Floods related to dam failure are generally significantly larger than natural floods. The information's that should be included in the dam break analysis are: size of available flood storage, spillway type and its capacity, hydrologic data, reservoir impoundment, dam information such as height, crest length, width and types of dam and downstream cross sections along the stream line.

3.2.2.2 Peak flood flow

In the Harza Feasibility report on the Finch'a power development, the probable maximum flood routed through the reservoir was 1560 cumecs. Annual flood exceeded once for different return period of Finch'a dam is given in table 3-1.

Table 3-1: Peak discharge at different return period (Source Ministry of Water, Irrigation and Energy, 1973)

Return period (year)	Peak discharge (m ³ /s)
5	224
10	254
25	285
50	325
100	355

3.2.3 Comparative analysis

Comparative analysis method is the simplest approach for dam breach flood estimation. In this method a given dam geometry, height, slope angles, and reservoir areas and volumes are compared with listed document of similar sized dams that have been failed. The dam breach parameters and peak discharges reported from similar dam failures are directly used for the dam being analysed.

3.2.4 Physically-based model

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. The most widely used physically- based model is the National Weather Service's breach program (NWS breach), which is used to predict the development of breach and the resulting outflow using an erosion model based on principles of hydraulic, sediment transport and soil mechanics (Colorado Dam Safety Branch, 2010).

3.2.5 Hydraulic model (HEC-RAS)

HEC-RAS (Hydraulic Engineering Center's River Analysis System) is free downloadable software which was developed by Hydrologic Engineering Center of the U.S. Army Corps of Engineering. HEC-RAS Model is public domain and an integrated system of software, designed

for interactive use multitasking use in the environment. The system is comprised of a graphical user interface (GUI), separate analysis components, data storage and management capabilities, graphics and reporting facilities. The HEC-RAS System contains four one dimensional river analysis components: steady flow water surface computations, unsteady flow simulation, movable boundary sediment transport computation, and water quality analysis. A key element is that all four components use a common geometric data representation and hydraulic computation routine.

The main objective of HEC-RAS Model is to compute water surface elevation at all location of interest for either a given set of flow data (steady flow simulation) or by routing hydrographs through the system (unsteady flow simulation). It is used to model both overtopping and piping failure breaches for embankment dam. The data needed for these computations are divided into the following categories: geometric data, steady flow data and unsteady flow data.

The basic geometric data used in the HEC-RAS software consists of establishing the connectivity of the river system (River system schematic); cross section data; reach lengths; energy loss coefficients (friction losses, contraction and expansion losses); and stream junction information (Brunner, 2010). Cross sections are located at interval along streams to characterize the flow carrying capability of the stream and its adjacent flood plain.

3.2.6 HEC-GeoRAS model

HEC-GeoRAS model is a set of GIS tools that used to create Fincha'a River Network including stream line, cross-section, banks, inline structure and storage area for import into HEC-RAS and generate the flood inundation data from the HEC-RAS output. The resulting flood from dam failure can be inundated with HEC-GeoRAS at available GIS data. RAS Mapper creates an inundation width and floodplain boundary from computed water surface profiles and HEC-RAS geometry.

3.2.7 Geographic information system (GIS)

A geographic information system is a powerful tool used for computerized mapping and spatial analysis. Arc-GIS tools are used to create triangulated irregular network from digital elevation model of study area for creation of river network and flood inundation mapping. It is used to develop flood plain maps by using computed water surface through HEC-RAS model.

3.2.8 HEC-RAS model setup

For developing dam break analysis using hydraulic model different component of the project have been represented in the model as follows.

3.2.8.1 Fincha'a River

Fincha'a River is created by HEC-GeoRAS software from DEM data of that location with 42 kilometer length and 106 total number of cross-section perpendicular to the stream line. The 42km length indicates that distance from the dam to further distance downstream of Fincha'a sugar factory. From 106 total number of cross-section 104 of it is cross-section at downstream of the Fincha'a dam and 2 of it is in the reservoir. The Fincha'a River created through HEC-GeoRAS software is exported to HEC-RAS software for model setup.

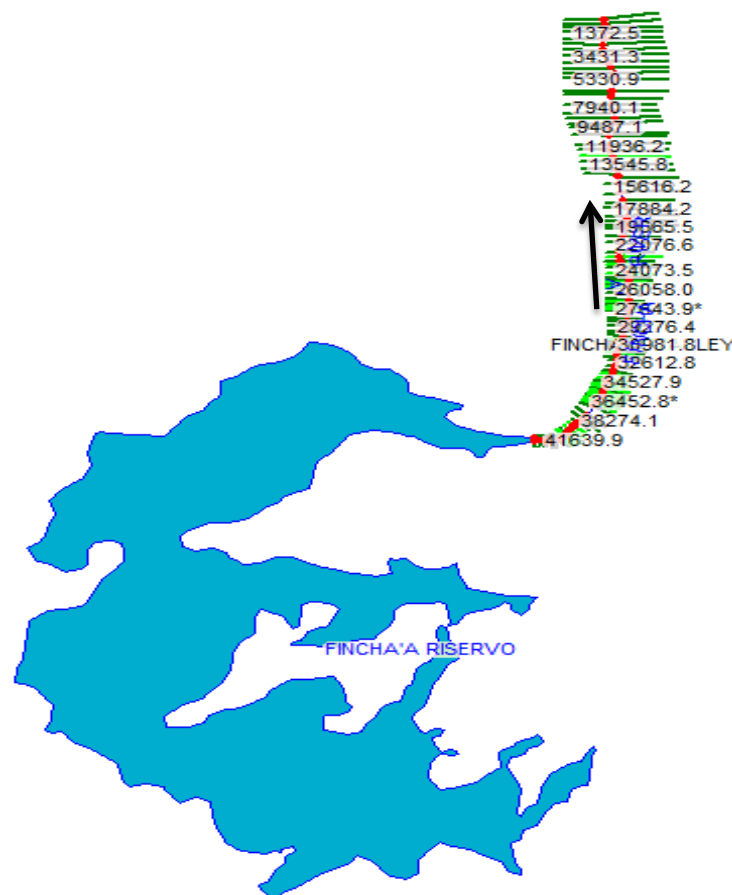


Figure 3-2: River Network for Fincha'a River and its reservoir coverage in HEC-RAS model

3.2.8.2 Fincha'a Reservoir

Fincha'a reservoir is routed by HEC-RAS model. To model a reservoir using level pool routing in HEC-RAS, the pool area is modeled with a storage area. The Fincha'a reservoir area is created by HEC-GeoRAS with Fincha'a River from DEM data as shown in figure 3-2 and later exported to HEC-RAS for simulation purpose. Table 3-2 shows elevation- volume data of Fincha'a reservoir.

Table 3-2: Elevation-Volume curve of Fincha'a Reservoir (Source Ministry of Water, Irrigation and Energy, 1973)

Elevation (m) from a.s.l	Volume (10^6 m^3)
2207	0
2209	12
2211	75
2216	304
2218	401
2220	486
2222	625

3.2.8.3 Boundary condition

Boundary conditions are necessary to define the upstream and downstream ends of the river system. The unsteady component of the HEC-RAS can perform subcritical, supercritical or mixed flow regime computation. In this study mixed flow regime simulation is selected in the HEC-RAS software.

3.2.8.3.1 Upstream boundary condition

The reservoir area upstream of the dam can either be modeled with cross-sections or by using storage area. If cross-sections are used, then HEC-RAS will perform full unsteady flow routing through the reservoir pool and downstream of the dam. If storage area is used, HEC-RAS uses level pool routing through the reservoir and unsteady flow routing through downstream of the dam. For this thesis study storage area is used for upstream boundary condition for dam break simulation and it is connected to upstream end of the reach. The probable maximum flood is used as lateral inflow hydrograph for the reservoir and which considered as inflow to the reservoir.

Table 3-3 shows the value of PMF of Fincha'a River which taken from feasibility study of Fincha'a dam.

Table 3-3: PMF of Fincha'a River (Ministry of Water, Irrigation and Energy)

Time(hrs)	Inflow (m ³ /s)	Time (hrs)	Inflow (m ³ /s)
0	150	13	900
1	230	14	830
2	280	15	710
3	285	16	540
4	290	17	400
5	300	18	330
6	310	19	300
7	340	20	250
8	380	21	210
9	400	22	180
10	490	23	160
11	900	24	150
12	1560		

3.2.8.3.2 Downstream boundary condition

For open ended river normal depth is most commonly used as downstream boundary condition. Therefore, Normal depth is used as downstream boundary condition for this thesis study. The frictional slope 0.012 is used as normal depth for downstream boundary condition. Frictional slope is equal to the average slope of the two cross –section from the end of the stream line.

3.2.8.4 Initial condition

In addition to boundary condition the user required to enter initial condition in the HEC-RAS system as an input data at the beginning of the unsteady flow simulation. Initial condition consists of flow at upper end of each reach and starting elevation of storage area. The flow at upper end of each reach (initial flow) is assumed to be 1% to 10% of peak probable maximum flood of each river according to rule of thumb (Brunner, 2010). Therefore, in my study to gate maximum peak

discharge I assumed initial flow and initial storage elevation of Fincha'a reservoir $156\text{m}^3/\text{s}$ and 2222m from mean sea level respectively.

3.2.9 Estimating breach parameters

The estimation of the breach location, size, and development time are critical in order to make accurate estimate of the outflow hydrographs and downstream inundation (Gray, 2014). Once the breaching parameters are estimated, the HEC-RAS can be used to compute the out flow hydrograph from the dam breach and perform downstream routing. The user is required to enter information like: failure location, failure mode, breach development time, breach shape, weir and piping coefficient and trigger mechanism into HEC-RAS model to define a dam breach.

3.2.9.1 Failure location

The breach failure location is based on type and shape of dam, mode of failure, and driving force of the failure. In general, the researcher should consider all the factors of the dam, including seepage and foundation problems, and place the breach location in the most possible location for each failure type. Since high hydrostatic pressure is percent at the center of bottom breach width of the dam for my study the failure location is assumed to be at the center of bottom breach width of the dam and develop equally in both sides.

3.2.9.2 Failure mode

HEC-RAS hydraulic computation model is used to analysis overtopping and piping failure mode. Failure mode is the mechanism for starting and growing a breach. Overtopping failure starts at full reservoir condition (crest of the dam) and develops to maximum degrees. Piping failure starts at normal reservoir condition and develops to maximum degrees.

3.2.9.3 Breach development time

Breach development time is estimated outside of the HEC-RAS software and entered as input data.

During overtopping failure the HEC-RAS breach start time is considered to be when the erosion process has migrated to upstream face of the dam. This is the point at which the outflow from the dam will start to increase due to the breach. The HEC-RAS breach end development time is

measured when the breach is fully formed and significant erosion has stopped. This mean breach ending time is not the time until the reservoir pool drain completely.

During piping failure the HEC-RAS breach start time is considered to be when a significant amount of flow and materials are coming out of the piping failure. The HEC-RAS breach ending time is considered when the breach is fully formed for most party (significant erosion has stopped, not the time until the reservoir pool is emptied).

3.2.9.4 Breach weir coefficient

Weir coefficient must be entered by the user in the HEC-RAS. Weir coefficient is used to compute overtopping mode of failure, and an orifice coefficient is used to compute piping/pressure flow mode failure. These coefficients directly affect the magnitude of the peak outflow hydrograph of the dam breach. In order to estimate weir coefficients for overtopping, it is necessary to understand the basic failure process. A breach process is a function of height of the dam, volume of water behind the dam (including the inflowing hydrograph), materials used for dam construction, depth and duration of overtopping, outer protective cover on the downstream and upstream side of the embankment, and other parameters.

In HEC-RAS software the user is only allowed to enter a single value of breach weir coefficient and the piping coefficient, because the estimation of peak flow is important in dam breach process. A weir coefficient (C) for embankment dam with medium to large storage volume of water in comparison to height of the dam, broad-crested weir, long crest length, and gravel/sand dam will be assumed to be 2.6, while for embankment dam with low volume of water in comparison to height of the dam, sharp-crested weir, and clay dam will be assumed to be 3.2 (Gray, 2014).

During a piping failure breach, the rate of water flowing through the dam is modeled with an orifice pressure flow equation. This equation also requires a discharge coefficient which is a measure of how efficiently the flow can get into the pipe orifice. Recommended values for piping/pressure flow coefficients of different dams are in the range of 0.5 to 0.6 (Gray, 2014). Guidelines for breach weir and piping flow coefficients of different type of dams are given in table 3-4.

Table 3-4: Dam breach weir and piping coefficients (Gray, 2014)

Dam type	Overflow/weir coefficients	Piping/pressure flow coefficients
Earthen clay or clay core	2.6-3.3	0.5-0.6
Earthen sand and gravel	2.6-3.0	0.5-0.6
Concrete Arch	3.1-3.3	0.5-0.6
Concrete Gravity	2.6-3.0	0.5-0.6

Since the storage capacity of Fincha'a embankment dam is medium with comparison to height of the dam and its spillway type is uncontrolled over flow spillway (broad crested weir) the selected value of weir coefficient is 1.8 and 0.5 for overtopping and piping mode of failure respectively.

3.2.9.5 Trigger mechanism

Trigger mechanism is defined as reservoir pool elevation; reservoir pool elevation plus duration of dam failure. For this thesis study reservoir pool elevation is used as an input into HEC-RAS.

3.2.9.6 Breach shape definitions

The shape of the breach during dam break can be rectangular, triangular, or trapezoidal. In major case studies the shape of the breach is trapezoidal. The trapezoidal shape of the breach will consist of height, width and side slope (H: V) of the dam breach. Side slopes are expressed in units of distance horizontal to every one unit in the vertical (zH: 1V). The HEC-RAS models simulate a failure by enlarging a trapezoidal-shaped breach with time in accordance with the specific progression and geometry (Colorado Dam Safety Branch, 2010). Therefore, trapezoidal shape is selected in may study as shown in fig 3-3.

The breach width is described as the average breach width (B_{ave}), while HEC-RAS requires the breach bottom width (W_b) for input. Therefore, breach bottom width is calculated by using different side slopes those recommend by each regression equations. The breach height (h_b) is vertical extent from the top of the dam to the average invert elevation of the breach. For overtopping mode of failure the breach height is equal with height of water (h_w), which is the vertical distance from the maximum water surface to the invert elevation of the breach.

The breach physical dimensions as well as the breach formation time must be estimated outside of the HEC-RAS software and entered into the program.

$$t_f = 63.2 \sqrt{\frac{V_w}{gH_b^2}} \text{-----} -3 - 2$$

Where:

B_{ave} = average breach width (meters)

K_O = 1.3 for overtopping and 1.0 for piping failure

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

H_b = height of the final breach (meters)

g = gravitation acceleration (9.80665 meters per second squared)

t_f = breach formation time (seconds)

The height of the breach for overtopping mode of failure is calculated by assuming that the breach goes from the maximum reservoir elevation to the natural ground elevation at the centerline of the breach location, while for piping mode of failure calculated by assuming that the breach initiates at normal pool elevation. Froehlich (2008) regression equation states that the average side slopes 1.0H: 1V for overtopping and 0.7H: 1V for piping failures.

Froehlich's regression equation for determination of peak discharge is:

$$Q_p = 114.8 B_{avg} h_w^{1.5} \left(\frac{\gamma}{\gamma + t_f \sqrt{h_w}} \right)^3 \text{-----} -3 - 3$$

Where:

Q_p = peak discharge outflow (cubic meter per second)

B_{avg} = average breach width (meters)

h_w = maximum depth of water stored behind the breach (meters)

t_f = breach development time (hours)

Υ = flow reduction factor = $23.4A_s/B_{avg}$

A_s = surface area of the reservoir corresponding to h_w (meter square)

3.2.10.2 Estimation of breach width and time by MacDonald and Langridge- Monopolis, 1984

MacDonald and Langridge –Monopolis (1984) utilized 42 earth fill dams, earth fill dams with clay core, and rock fill dams data sets to develop breach formation factor. The Breach Formation Factor (BFF) is the product of the reservoir storage volume and maximum water depth of reservoir, which used to calculate the volume material eroded from embankment dam during breach development. They related the breach formation factor to the volume of material eroded from embankment dam. For the MacDonald and Langridge –Monopolis regression equation the breach shape is trapezoidal with side slope of 0.5H: 1V. The data that used by MacDonald and Langridge –Monopolis for their regression analysis had the following ranges:

- Height of the dam: 4.27 - 92.96 meters
- Breach outflow volume: $0.0037 - 660 \times 10^6$ cubic meters

MacDonald and Langridge –Monopolis regression equations for determination of volume of material eroded, breach formation time and bottom breach width are:

$$V_{eroded} = 0.00348(BFF)^{0.852} \text{ --- 3 - 4}$$

$$t_f = 0.016(V_{eroded})^{0.364} \text{ --- 3 - 5}$$

$$B_{avg} = \frac{V_{eroded}}{(H_b * W_{avg})} \text{ --- 3 - 6}$$

$$W_{avg} = C + H_b * \frac{(Z_u + Z_d)}{2} \text{ --- 3 - 7}$$

V_{eroded} = Volume of material eroded from the embankment dam (cubic meters)

BFF = Breach Formation Factor = reservoir storage volume *reservoir water depth

t_f = Breach formation time (hours)

W_{avg} = Average width of dam in the direction of flow (meters)

H_b = Height from the top of the dam to bottom of the breach (meters)

C = Crest width of the dam (meters)

Z_u = Slope of upstream face of the embankment dam (Z_uH : 1V)

Z_d = Slope of downstream of the embankment dam (Z_dH : 1V)

$Z_t = Z_u + Z_d$

Z_b = Side slope of the breach (0.5:1)

MacDonald and Langridge –Monopolis regression equation for determination of peak discharge is:

$$Q_p = 1.154(V_w h_w)^{0.412} \text{ --- --- --- --- --- } -3 - 8$$

Where:

Q_p = peak breach outflow (cubic meter per second)

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

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

3.2.10.3 Estimation of breach width and time by von Thun and Gillette, 1990

Von Thun and Gillette used 57 earthen, zoned earthen, earthen with clay core, and rock fill data sets from MacDonald and Langridge –Monopolis (1984) and Froehlich (2008) to develop regression equations to predict average breach width, side slopes and failure time. According to this method the breach parameters calculated for both overtopping and piping mode of failure is assumed to be the same. The method proposes to use breach side slope of 1H: 1V for homogeneous earth fill dam and 0.5H: 1V for the others. The data that used by Von Thun and Gillette (1990) for their regression analysis had the following ranges:

- Height of the dam: 3.66 - 92.96 meters
- Volume of water at breach time: 0.027 - 660*10⁶ cubic meter

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

$$B_{av} = 2.5h_w + C_b \text{-----} 3 - 9$$

Where:

B_{av}–Average breach width (meters)

h_w – Depth of water above the bottom of the breach (meters)

C_b – Coefficient

C_b value is based on the size of reservoir and which is given in the table 3-5.

Table 3-5: Coefficient as function of reservoir size (Gray, 2014)

Reservoir size (cubic meters)	C _b (meters)
< 1.23*10 ⁶	6.1
1.23*10 ⁶ -6.17*10 ⁶	18.3
6.17*10 ⁶ -1.23*10 ⁷	42.7
>1.23*10 ⁷	54.9

Since Fincha’a dam is rock fill dam and its reservoir capacity is greater than 1.23*10⁷ cubic meters the breach side slope and coefficient (C_b) value selected are respectively 0.5H: 1V and 54.9meters.

Von Thun and Gillette developed two different sets of equation for the breach development time. The first set of equation shows breach development time as function of water depth (h_w) above the breach bottom for erosion resistance and easily erodible respectively as follows.

$$t_f = 0.02h_w + 0.25 \text{-----} 3 - 10$$

$$t_f = 0.015h_w \text{-----} 3 - 11$$

$$t_f = \frac{D_{av}}{4h_w} \quad \text{--- --- --- --- ---} \quad 3-12$$

$$t_f = \frac{B_{av}}{4h_w + 61} \quad \text{--- --- --- --- ---} \quad 3-13$$

$$t_f = \frac{B_{av}}{4h_w + 61} \text{-----} 3-13$$

As stated above under empirical methods different peak flow equations are developed by different researchers from historic dam failure data. The peak flow equations were derived from data for earthen zoned, earthen with impervious core (i.e. clay, concrete, etc.) and rock fill dam only, and don't apply to concrete dams (Gray, 2014).

Figure 3-4 shows the envelope curves of different peak flow regression equation which used for comparison with the computed peak flow in the HEC-RAS model.

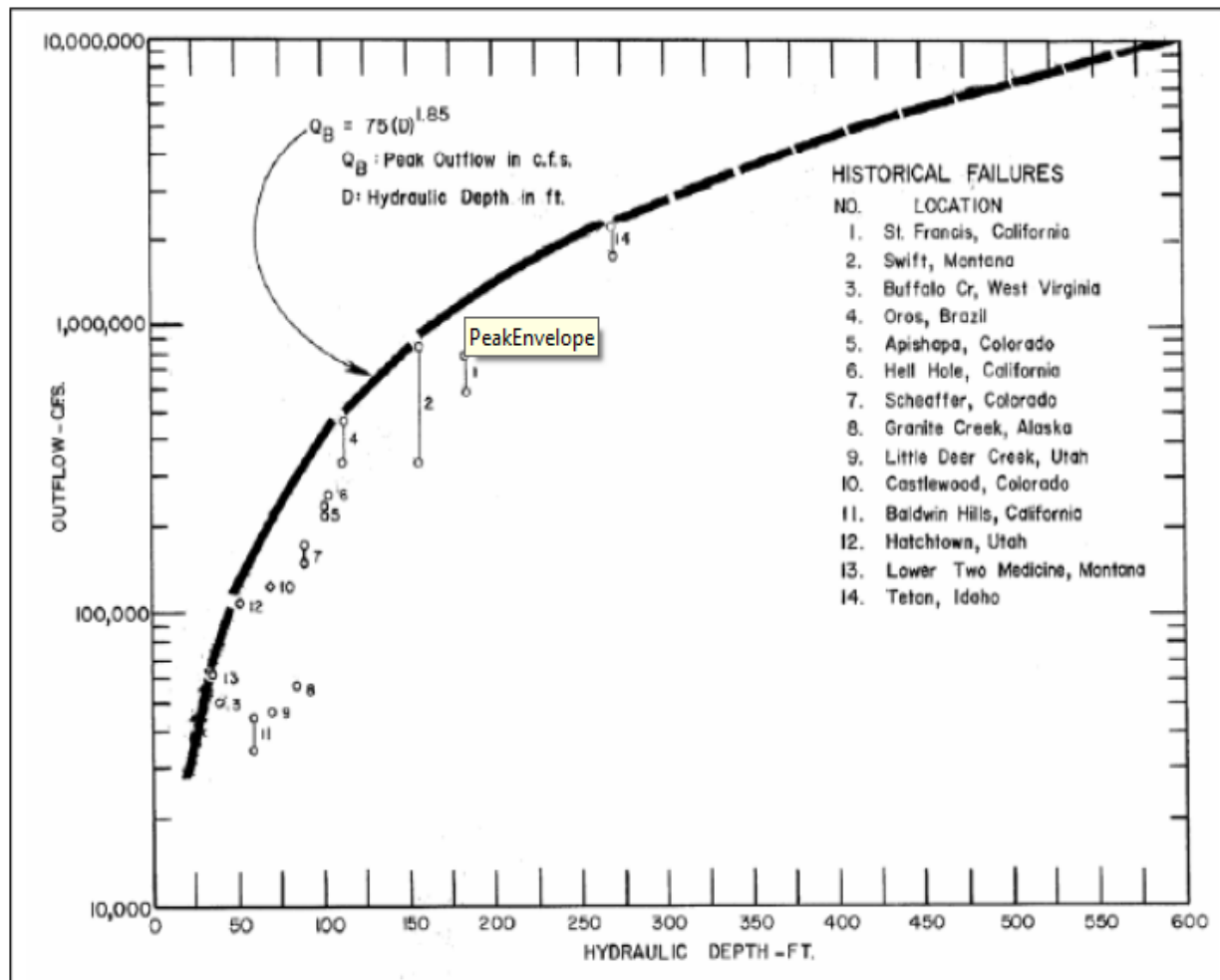


Figure 3-4: Envelope of experienced outflow rates from breached Dams (Gray, 2014)

3.2.11 Manning roughness coefficient

Roughness coefficients represent the resistance to flow and flood pains. Manning's n coefficient is used in the manning equation to calculate discharge in the open channel. The manning roughness coefficient is required to enter into HEC-RAS software by user for each channel cross-section. I assumed that the same manning roughness coefficient along the wetted perimeter of the channel cross section by considering similar river bed and also the same manning's roughness coefficient for both river banks. The natural streams which has rocky river beds with grass banks usually steep, trees and brush along banks has manning roughness coefficient in the range of 0.03 to 0.05 (Chow, 1959). Since the Fincha'a river bank surface is rocky the friction resistance

through the bank is low the manning roughness coefficient value is assumed to be 0.033 for bed channel and 0.046 for river banks.

3.3 Collection of cross-section

Fincha'a River cross-section is collected from digital elevation model (DEM) by using HEC-GeoRAS model. The upstream end cross-section is used as upper boundary condition of Fincha'a River. The second cross-section in the pool area is required as the bounding cross section for the inline structure. The extracted cross-section from digital elevation model is used as input data for HEC-RAS model.

3.4 Inundation map preparation

The inundation map should be developed at a scale sufficient to be used for identifying downstream inhabited areas subject to possible danger (Fread, 2011). GIS data required for mapping inundation is exported from HEC-RAS model to Geographic information system. The exported GIS data's are entire cross-section and maximum water surface elevation through the cross-section. Later inundation map is created by RAS mapper by using exported geographic information system data.

4 RESULTS AND DISCUSSIONS

4.1 Breach width and time development

The average breach width and breach development time of the Fincha'a dam is calculated by using the information of case study of the dam through regression equation of the MacDonald and Langridge –Monopolis (1984), Von Thun and Gillette (1990) and Froehlich (2008). The average breach width calculated by MacDonald and Langridge –Monopolis (1984) is out of limitation (greater than the crest length of the dam) for both mode of failure. The breach development time calculated by MacDonald and Langridge –Monopolis (1984) are 2.62hr and 2.67hr for overtopping and piping mode of failure respectively. The bottom breach width of the dam is calculated by using average breach height, average breach width and side slope. Therefore, the calculated bottom width by MacDonald and Langridge –Monopolis (1984) which used for HEC-RAS model as input is also out of limitation for both overtopping and piping mode of failure.

The average breach width and breach development time calculated by Froehlich (2008) are 257.16m, 196.88m and 7.79hr, 8.65hr for overtopping and piping mode of failure respectively. The bottom breach width of the dam which used as input for HEC-RAS model is 239.2m and 185.54m for overtopping and piping mode of failure respectively.

The average breach width and breach development time calculated by Von Thun and Gillette (1990) are 95.9m and 0.58hr for both of overtopping and piping mode of failure. The bottom breach width of the dam which used as input for HEC-RAS model is 87m for both mode of failure.

Since the breach parameters calculated by Von Thun and Gillette (1990) regression equation is more reasonable than the other regression equation it is selected for Fincha'a dam break analysis.

4.2 Overtopping mode of failure

For this failure mode the most critical situation is when the reservoir is at full reservoir condition and when excess flood passes over the crest of the dam. The spillway capacity is $213\text{m}^3/\text{s}$ which is 34.37 times less than the maximum flood passing over the dam. Since the spillway is uncontrolled type of spillway the flow greater than its capacity flows over the dam. So, it is

assumed that the dam is failed due to overtopping. Breach parameters such as bottom breach width of dam and breach formation time calculated by Von Thun and Gillette regression equation are used as input data for HEC-RAS model. The reservoir water level when dam break started will be 2222 m from mean sea level and the breach continues up to 2209 m (a.s.l).

4.2.1 Dam breach statistics and flood hydrograph

The maximum discharge flows out from the breached dam is $7436.8 \text{ m}^3/\text{s}$. The maximum discharge is attained at 3:00 hrs from the start of dam breach and the water is coming out with the velocity of 22.65 m/s . The hydraulic depth used to check the reality of peak discharge is calculated through the HEC-RAS model and its unit is converted from meter to ft. The flood routed value by overtopping mode of failure at dam site is written under appendix A-1. The reality of this result is checked with envelope curve of experienced outflows rates from breached dam as indicated in figure 4-1.

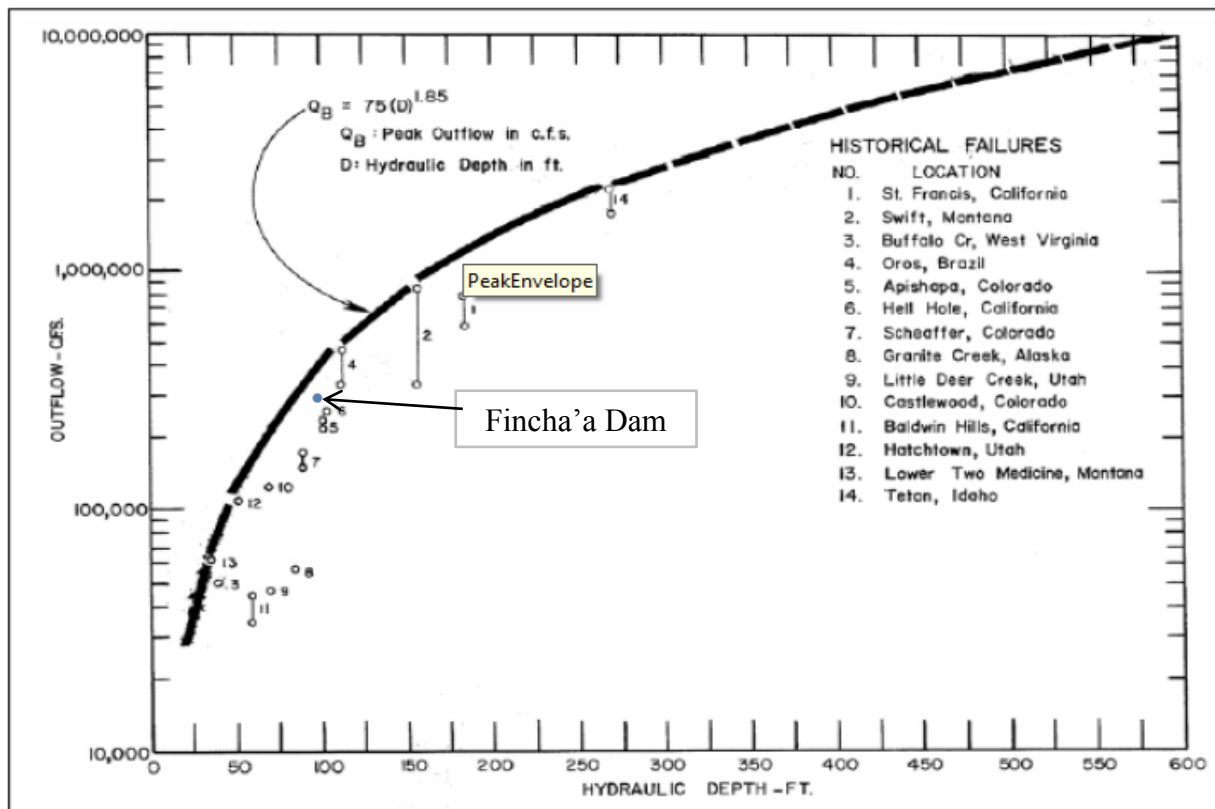


Figure 4-1: Location of Fincha'a dam on envelope of experienced outflow rates from breached dam by overtopping

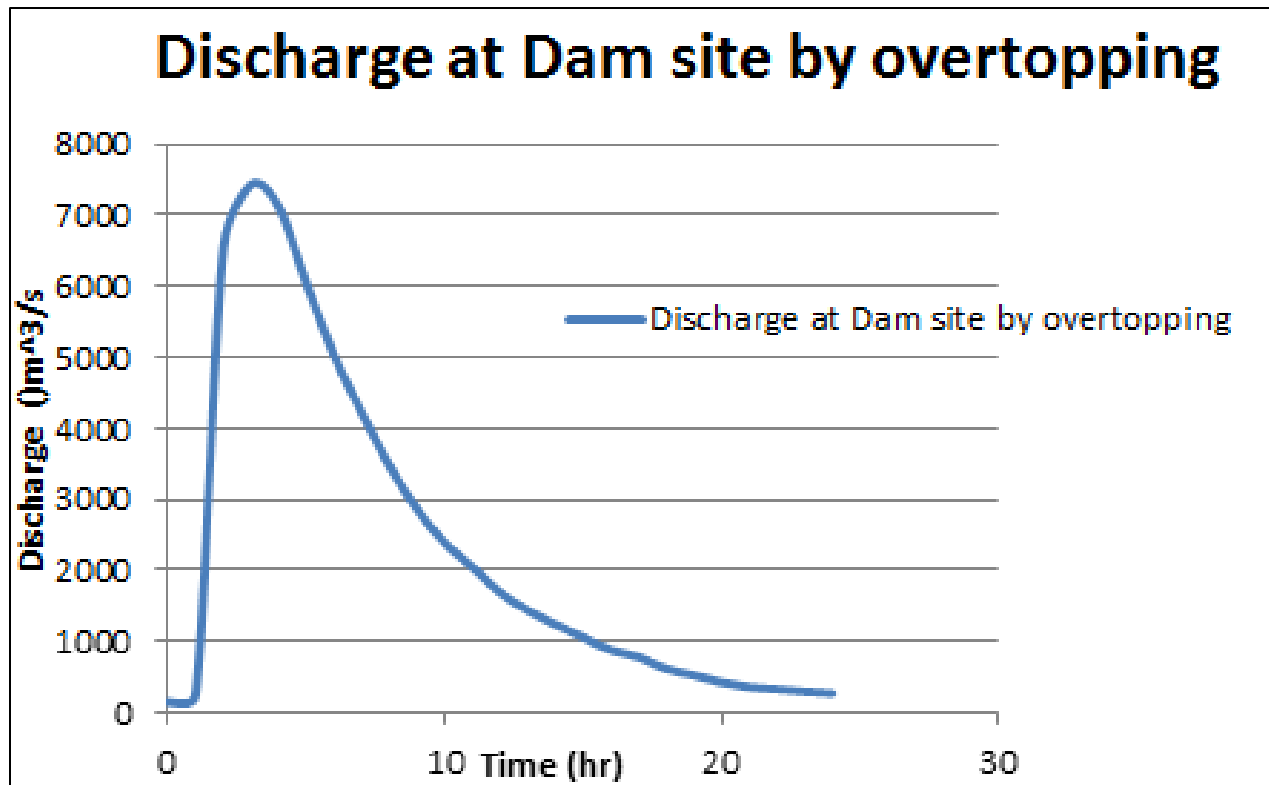


Figure 4-2: Flood hydrograph at Dam site by overtopping

4.2.2 Routing of flood hydrograph at different chainage

Routing of flood hydrograph is analyzed at three chainage points, 6.5km, 24.5km, and 42km downstream of the dam. Fig 4-3 shows the flood hydrographs for these different chainage points. At 6.5km downstream location, the peak flood discharge is about $7187.45\text{m}^3/\text{s}$ which is 2.49% less than the peak discharge coming out from the breached dam at dam site and it arrive at 4:00 hrs. Again for further downstream distance the peak flood discharge $6589.69\text{m}^3/\text{s}$ at 24.5km arrives at 7:00hrs and at 42km the peak flood discharge is $5448.86\text{m}^3/\text{s}$ at 10:00hrs.

Generally peak flood discharge decreases starting from dam site to further downstream distance. This indicates that upstream area of downstream location is more affected than the downstream area.

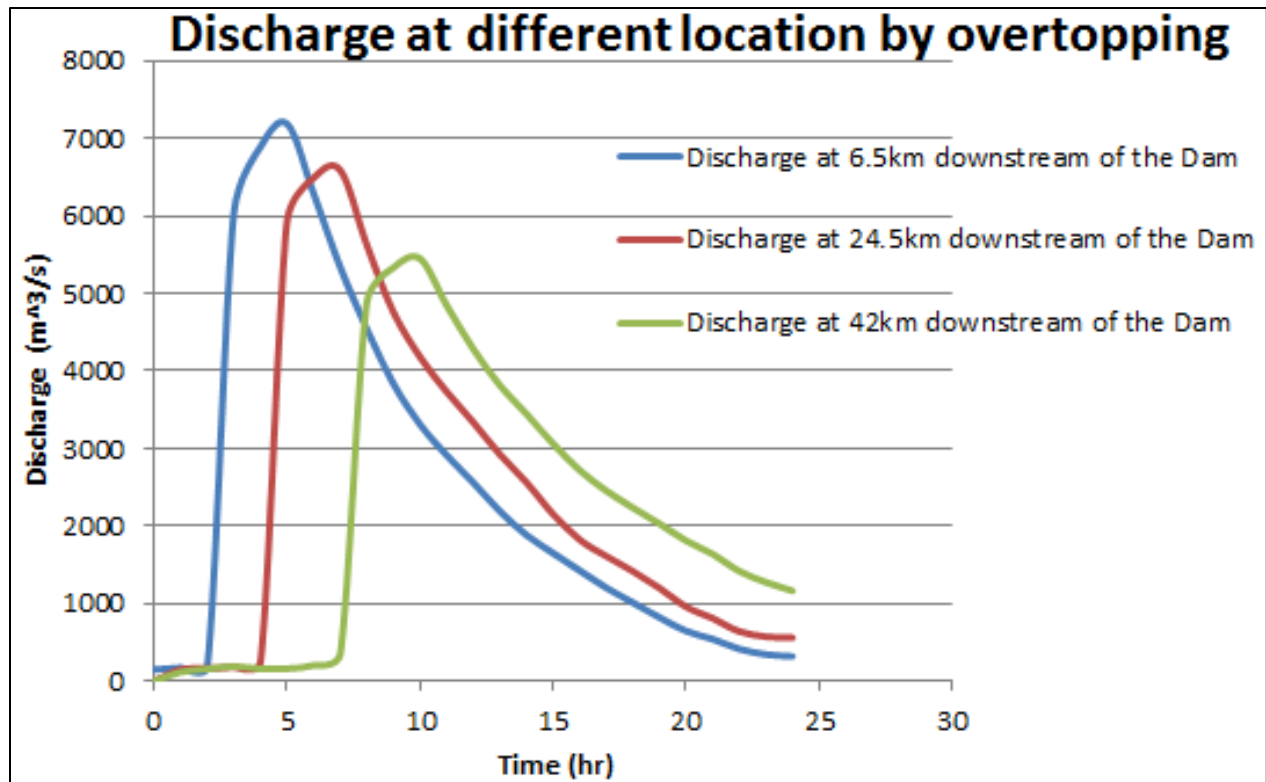


Figure 4-3: Flood hydrograph at 6.5km, 24.5km and 42km downstream of the dam by overtopping

4.2.3 Longitudinal profile

Fig 4-4 shows Longitudinal bed profile of Fincha'a River including bed level, minimum bank level and maximum water level occurred during dam break. As we observe from the longitudinal profile the water surface decreases from the dam to downstream points. We understand from this bed profile the bed slope is flat.

4.2.4 River cross-section at different chainage for overtopping mode of failure

Fig 4-5 to 4-7 shows the cross section of river at 6.5km, 24.5km and 42km respectively with maximum water level. As we observed from these cross sections their water stage is decreasing in the downstream direction. The water depth and top width over the banks at stated chainage points are: 3.63m, 2.67m, 1.74m and 752.69m, 1909.76m, 2047.01m respectively. From this number what we understand that the water depth is decreasing and top width of water is increasing along downstream direction.

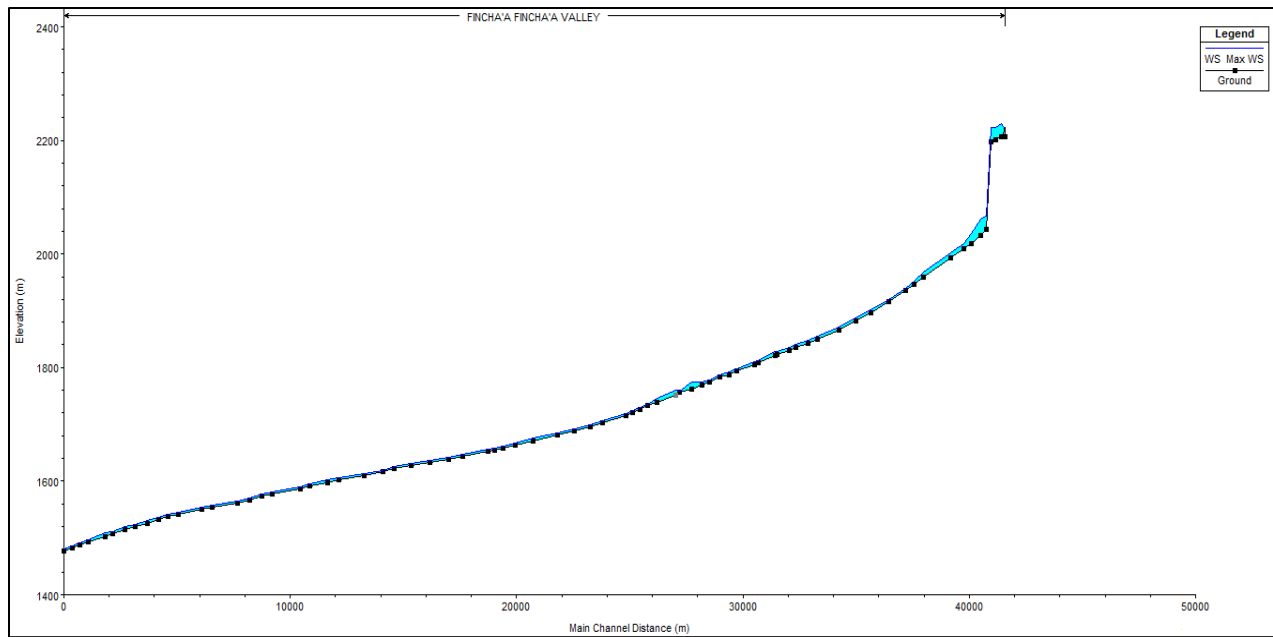


Figure 4-4: Longitudinal bed profile of Fincha'a River showing maximum water level

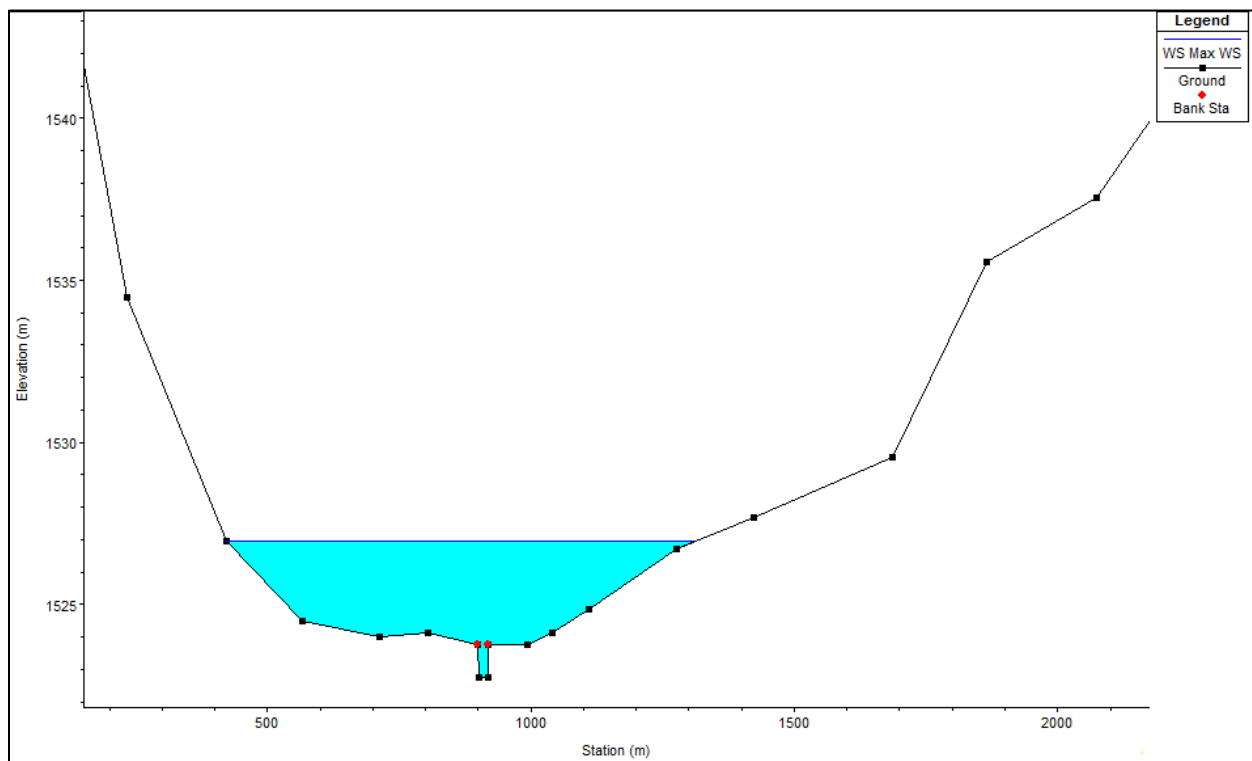


Figure 4-5: River cross section at 6.5km from Fincha'a Dam

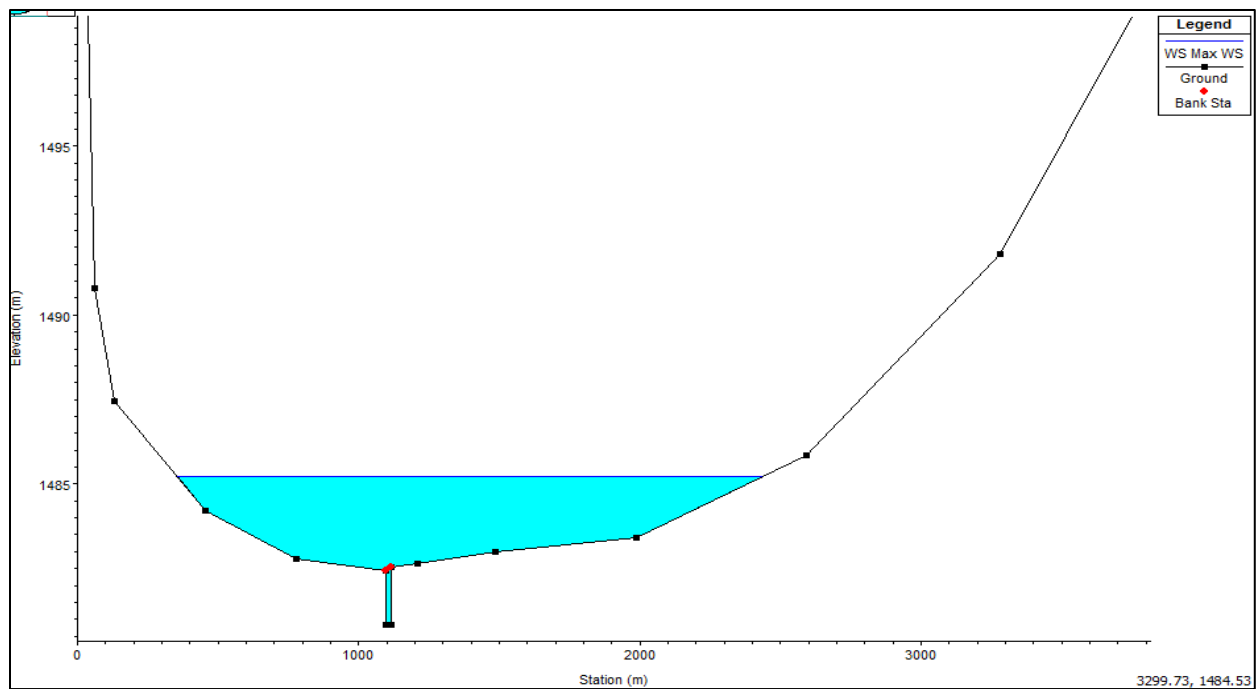


Figure 4-6: River cross- section at 24.5km downstream of the dam

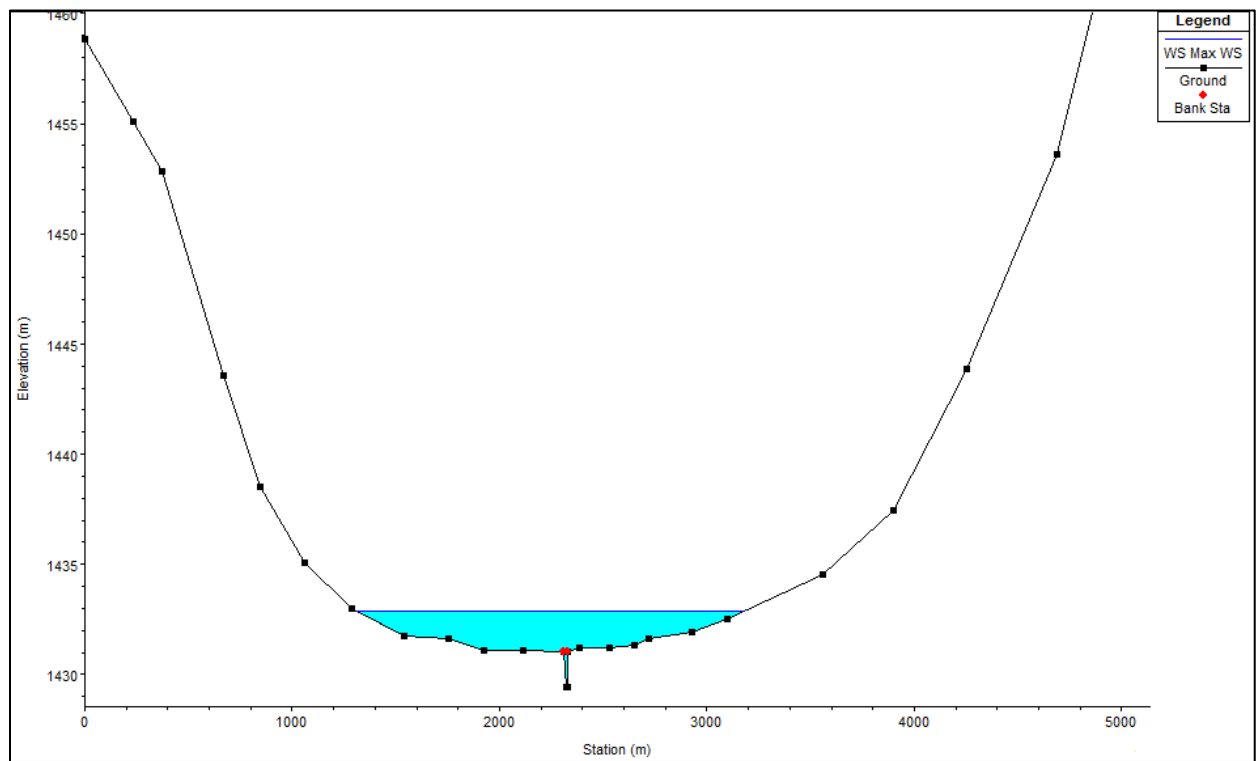


Figure 4-7: River cross-section at 42km downstream of the dam

4.2.5 3-Dimensional water surface profile

To show clearly the 3-Dimensional profile the downstream distance from the dam is divided into four with different interval distance. Figure 4-8, 4-10, 4-11 and 4-12 shows 3-Dimensional water surface profile at 6.5km, 16.5km 24.5km and 42km and maximum water surface in all cross sections respectively.

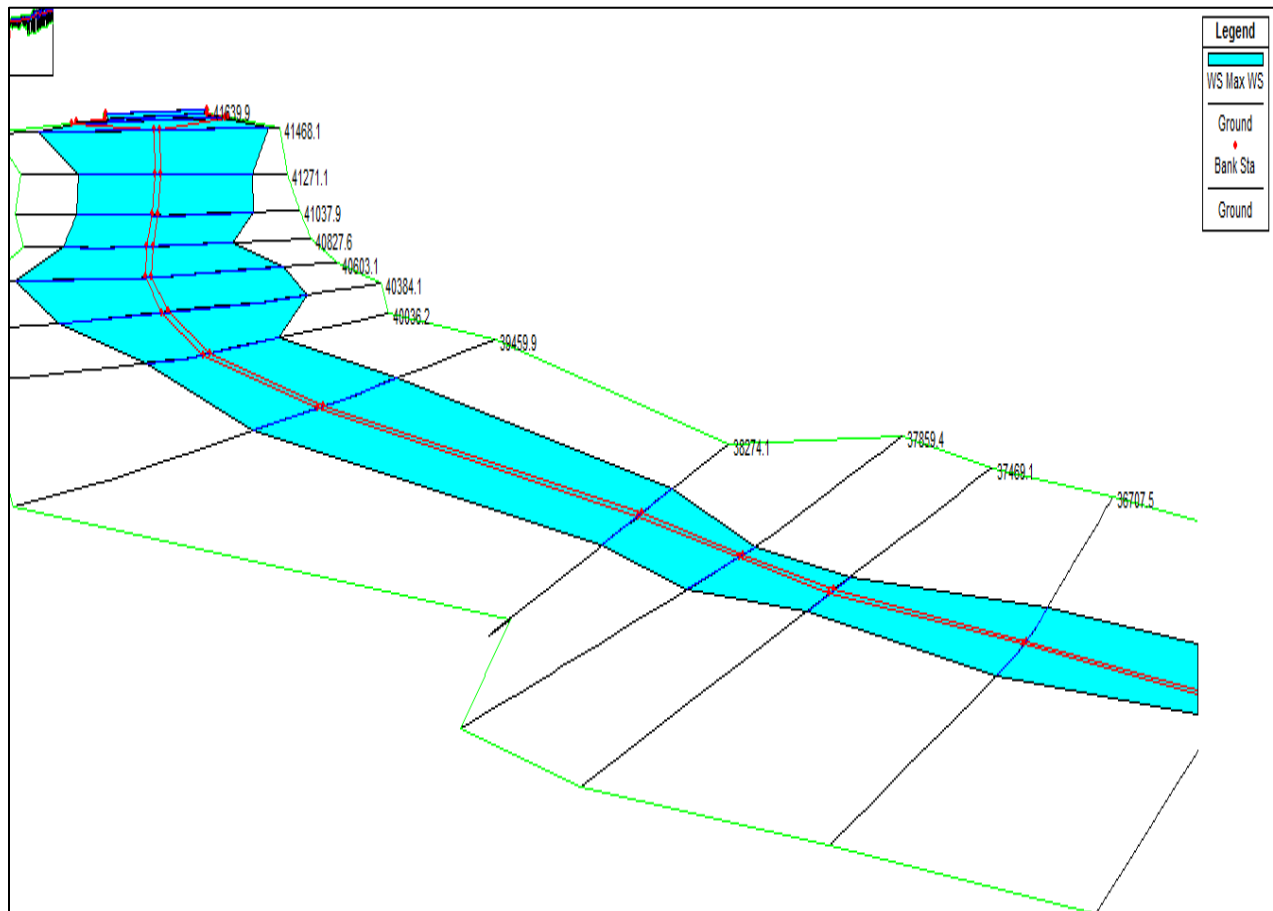


Figure 4-8: 3-D Water surface profile of peak discharge between dam and 6.5km

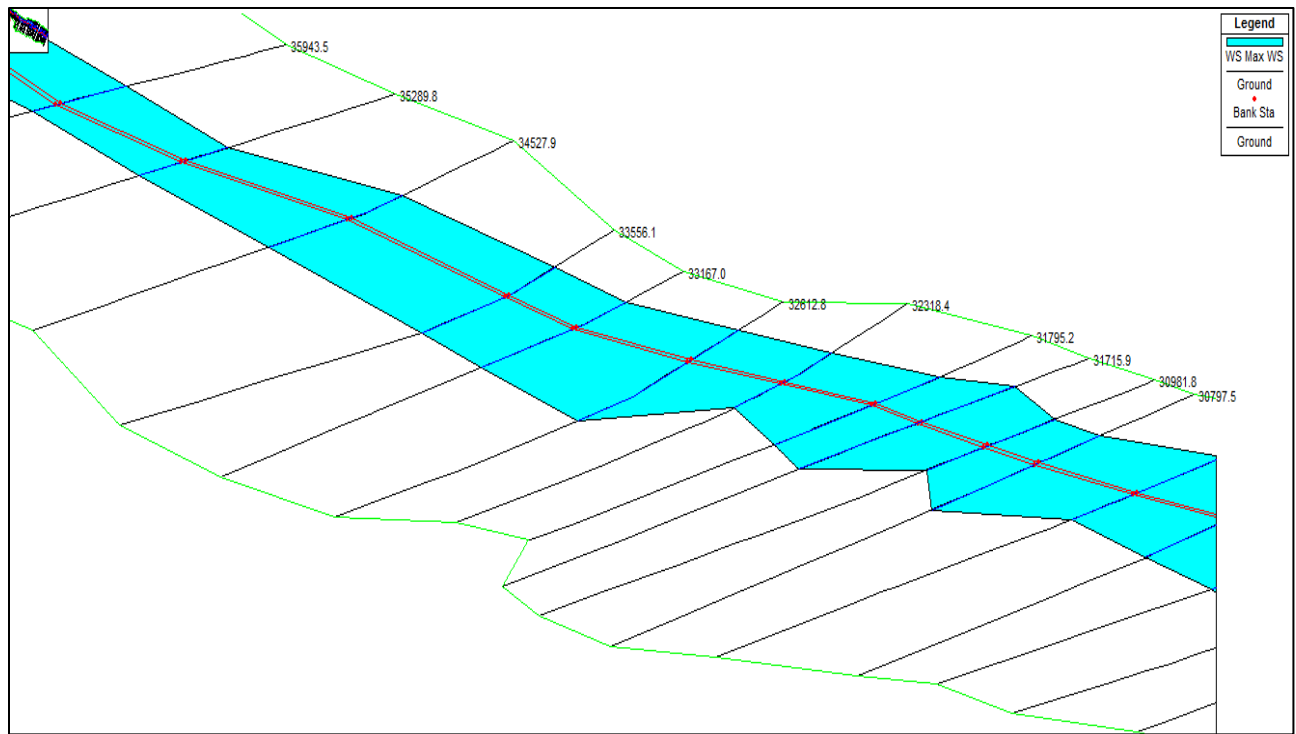


Figure 4-9: 3-D Water surface profile of peak discharge between 6.5km and 16.5km

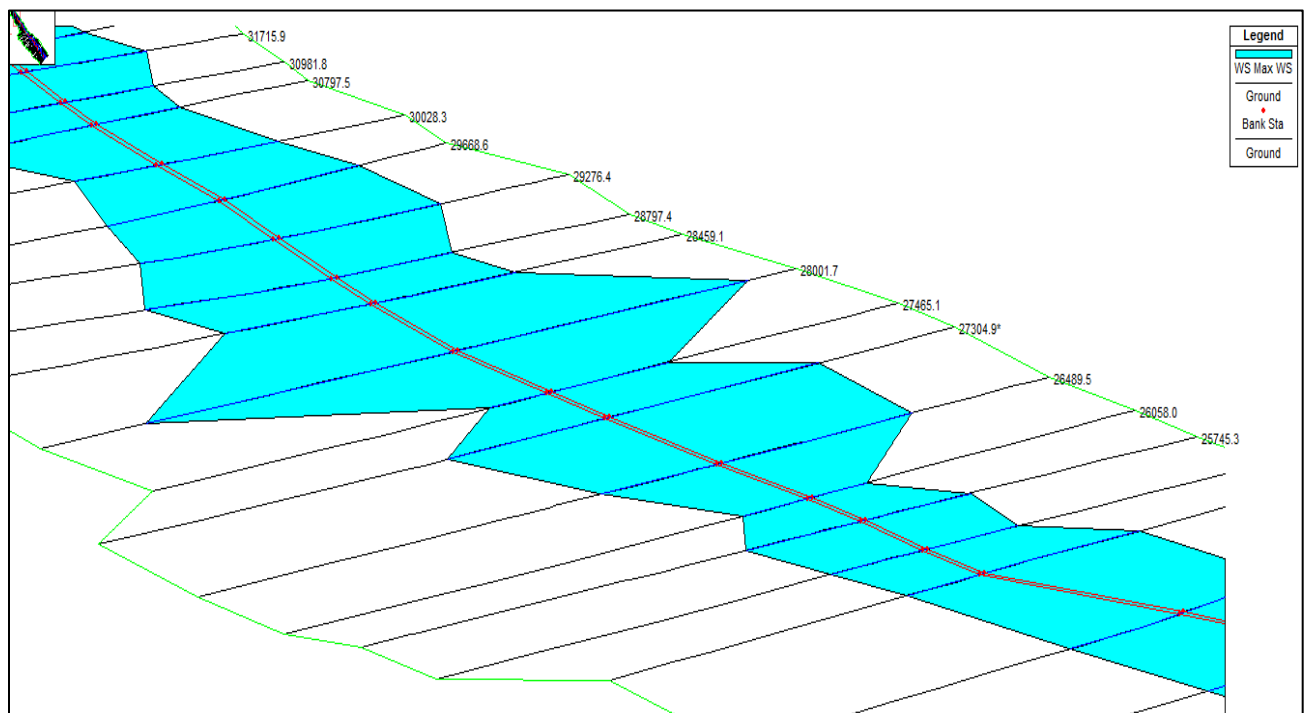


Figure 4-10: 3-D Water surface profile of peak discharge between 16.5km and 24.5km

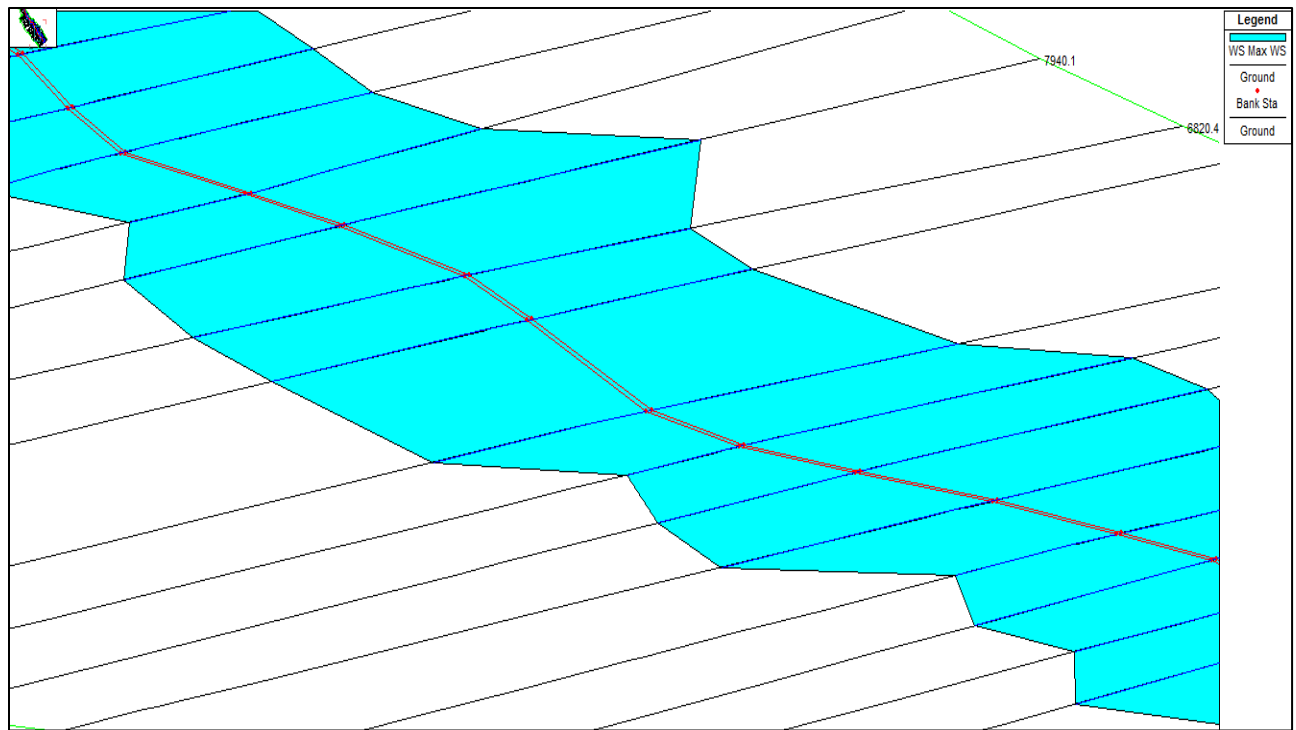


Figure 4-11: 3-D Water surface profile of peak discharge between 24.5km and 42km

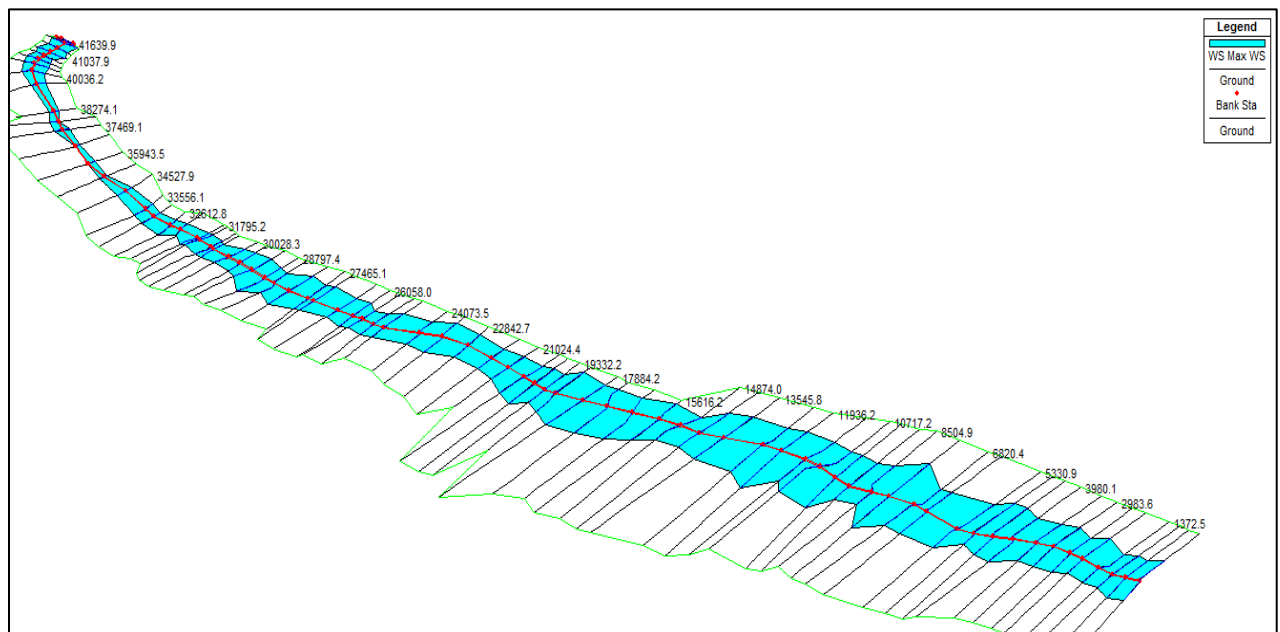


Figure 4-12: 3-D Water surface profile for all Fincha'a River cross-sections

4.3 Piping failure (No PMF) full Reservoir

In this method the most critical condition is at normal reservoir condition. The dam breach due to piping depends on the sunny day condition. As stated under overtopping mode of failure the breach parameters used for input for HEC-RAS model are calculated by Von Thun and Gillette. When dam breach started the water level of reservoir is 2222m (a.s.l) and the breach will continue up to 2209m (a.s.l). The initial piping elevation for piping mode of failure is assumed to be at center of bottom breach width due to occurrence of high hydrostatic pressure.

4.3.1 Dam breach flood hydrograph by piping method

At dam site the peak discharge flows out from dam breach by piping is $7241.22\text{m}^3/\text{s}$ at 3:00hrs and it attains maximum water level 2222.01m. This peak flood is coming out with maximum velocity 20.94m/s. Since the amount water available during dam breach for piping failure is less than that of overtopping the amount of peak discharge formed by piping mode of failure is less than that of the overtopping mode of failure. This shows that the dam breached by overtopping is more devastating than that of piping mode. The flood routed value by this mode of failure is written under appendix A-2.

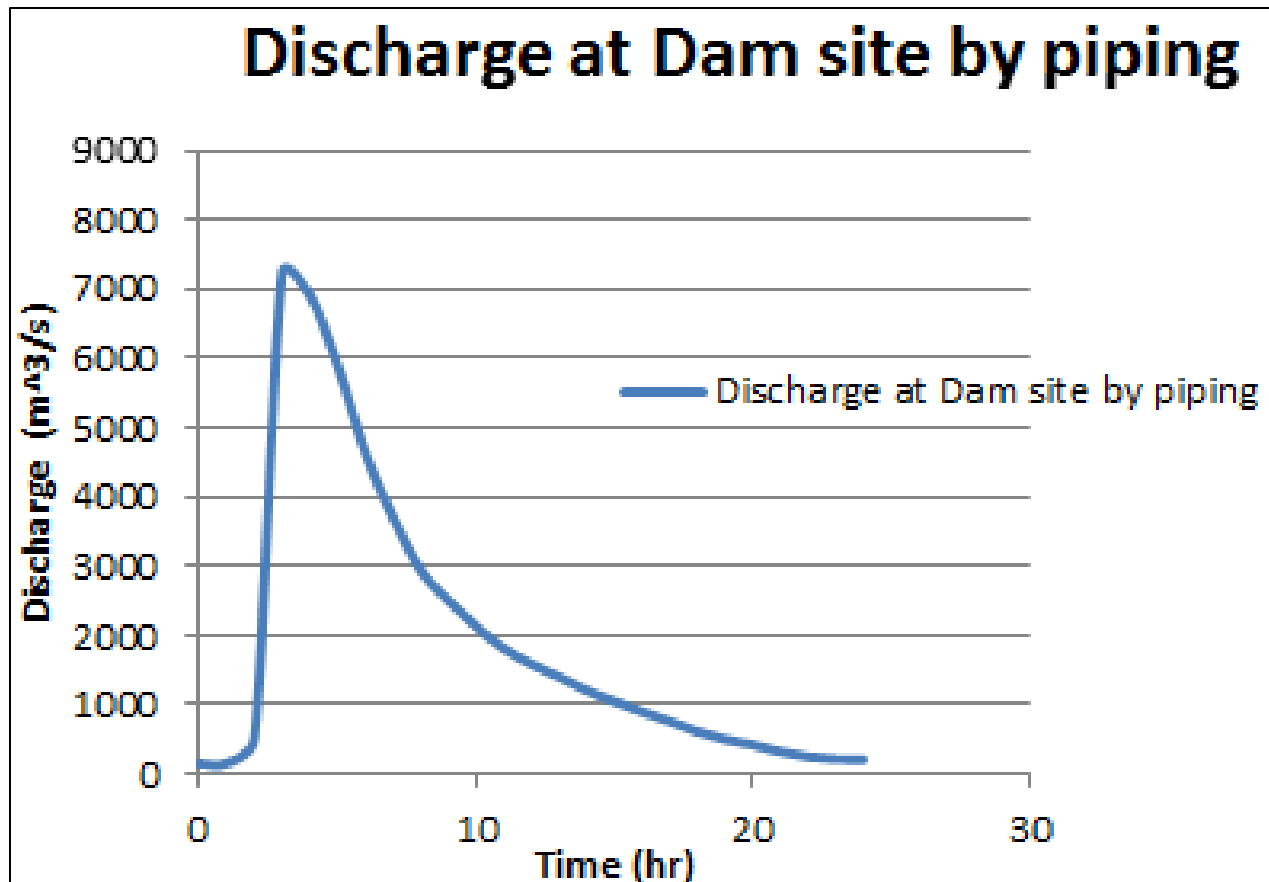


Figure 4-13: Flood routing at dam site by piping

4.3.2 Routing of flood hydrograph at different chainage by piping method

As discussed under overtopping mode of failure the routing of flood hydrograph is analyzed at three chainage points: 6.5km, 24.5 km and 42km downstream of the Fincha'a dam. At 6.5km the peak discharge $6654.29\text{m}^3/\text{s}$ formed at 4hrs which is less than the peak discharge that formed at the dam site. As we go further to 24.5km and 42km downstream of the dam, the corresponding the peak discharge are: $5689.69\text{m}^3/\text{s}$ and $4804.84\text{m}^3/\text{s}$ and flows out at 7:00hrs and 10:00hrs respectively. Figure 4-14 shows routing of flood hydrograph at different chainage points downstream of the dam.

4.3.3 River cross-section at different chainage for piping mode of failure

Figure 4-15 to 4-17 shows the cross sections of each location with their water level and top width. The water depth and top width of dam breach at different downstream locations are: 3.42m, 2.61m, 1.7m and 752.55m, 1909.25m, 2074.01m respectively.

Generally when we analyze the dam breach for Fincha'a dam by overtopping has greater water level and top width. So, by comparing the peak flood discharge the risk that caused by overtopping mode of failure is greater than that of the piping failure mode.

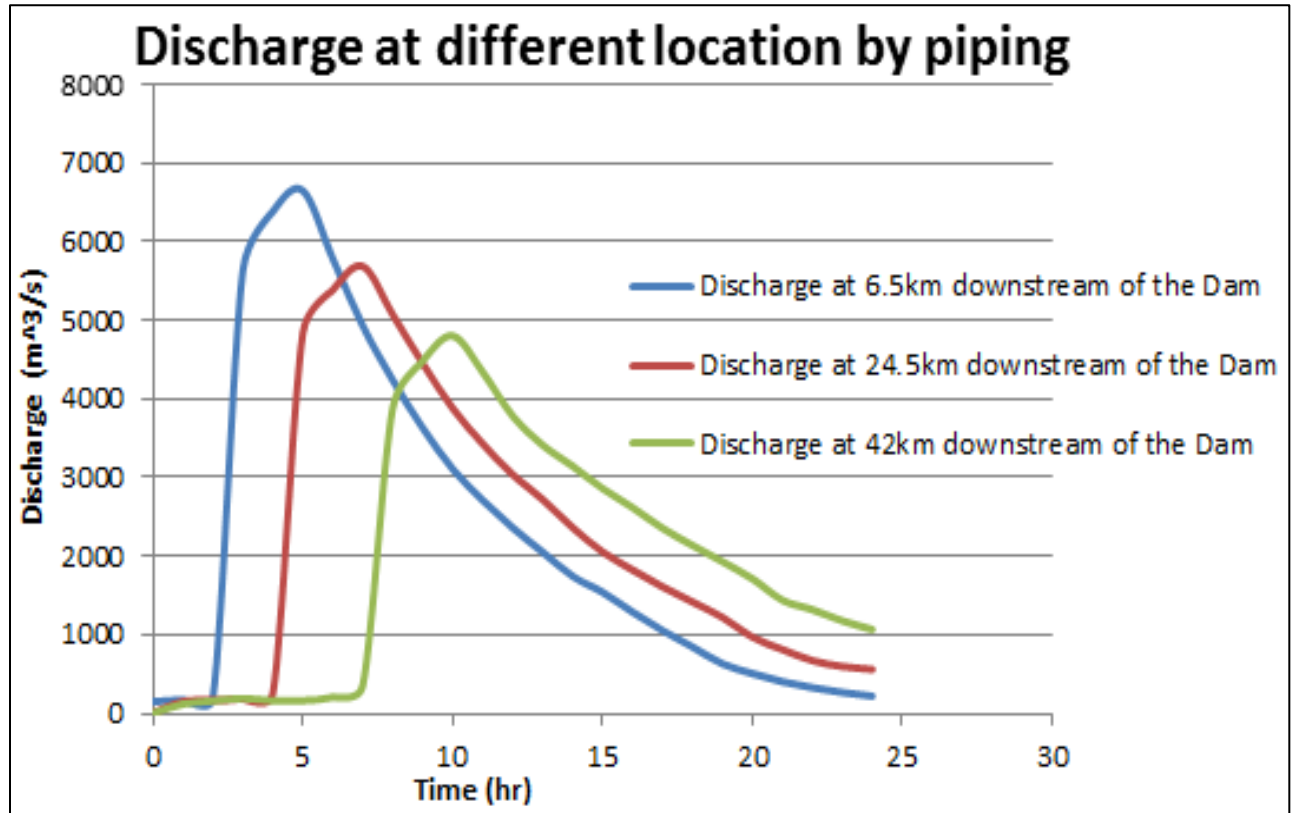


Figure 4-14: Flood hydrograph at 6.5km, 24.5km and 42km downstream of the dam by piping

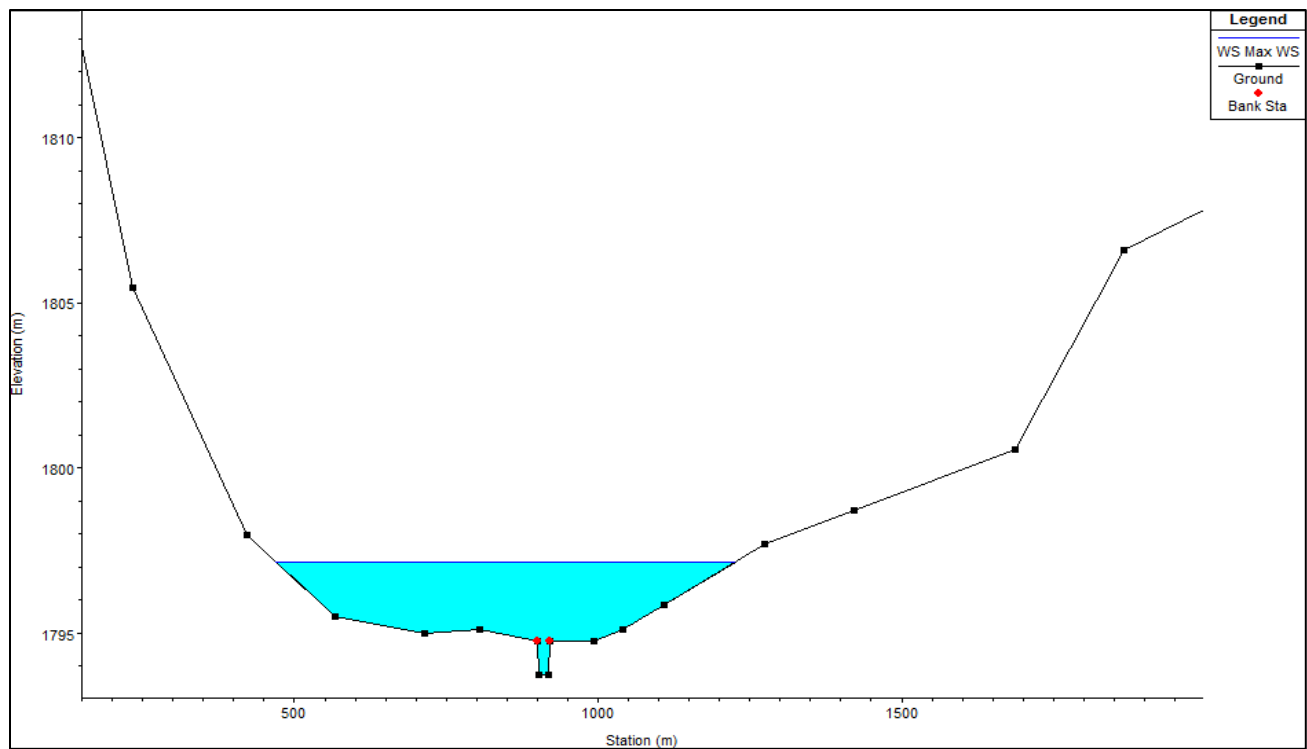


Figure 4-15: Fincha'a River cross-section water level at 6.5km from the dam

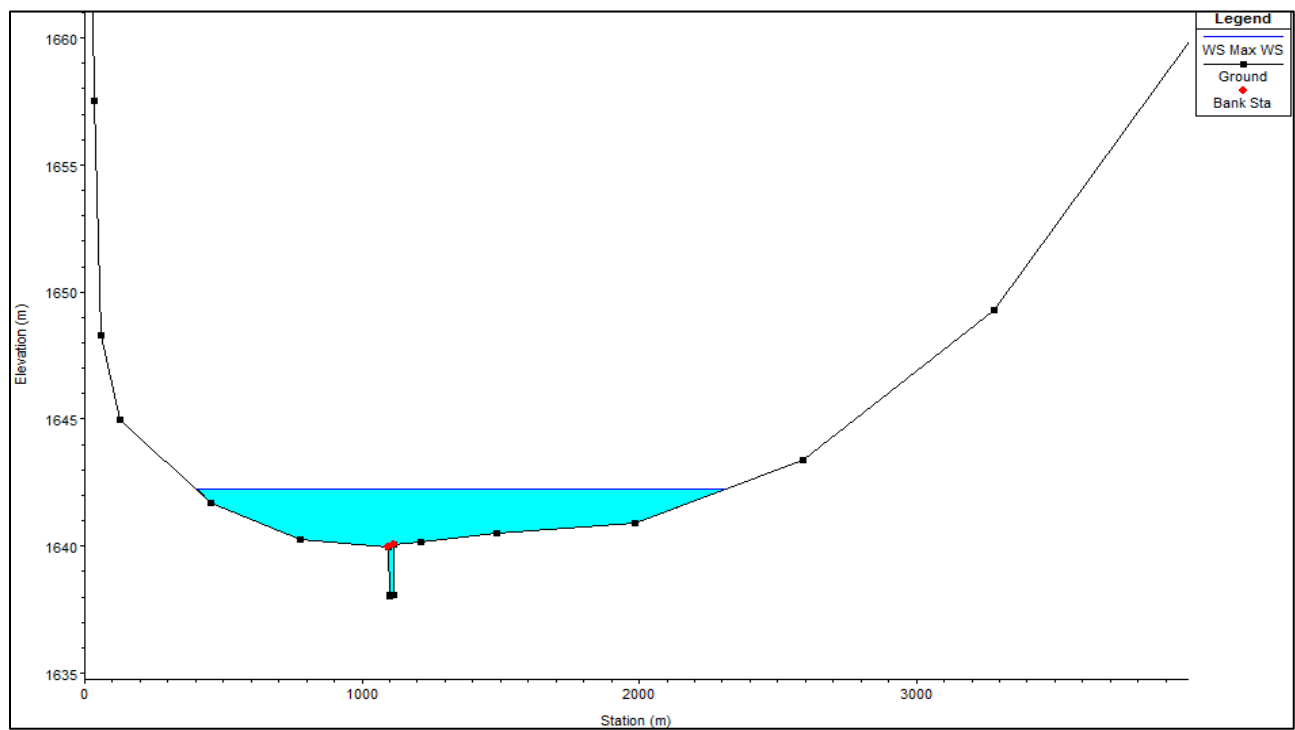


Figure 4-16: Fincha'a River cross-section water level at 24.5km from the dam

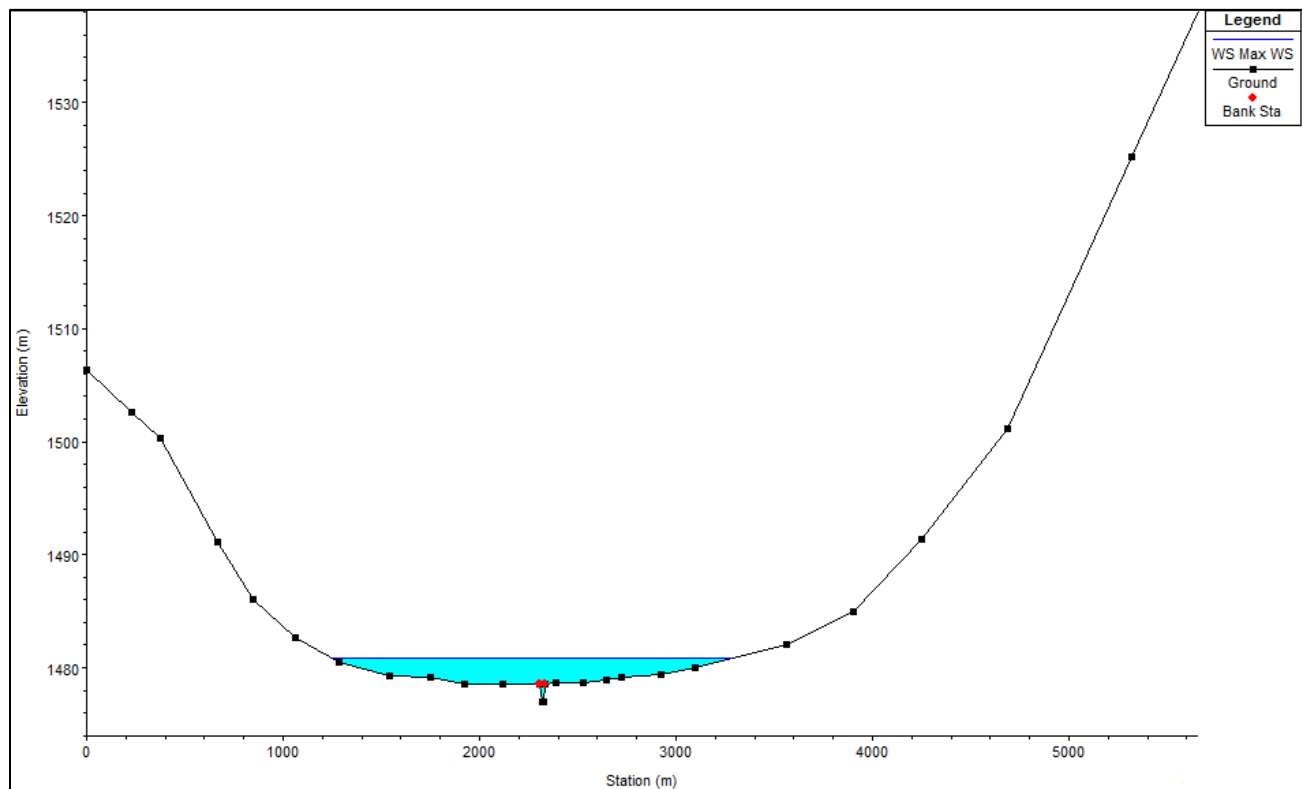


Figure 4-17: Fincha's River cross-section water level at 42km from the da

4.4 Sensitive parameters for input parameters in terms of peak discharge and water level

Input parameters selected for the dam break model are very important to do the analysis. The sensitivity analysis of input breach parameter is undergone by changing the values of these parameters to model dam break and analyzing what is the effect of each breach parameters on discharge values and water level. So, input parameters which are considered for the sensitivity analysis are: breach time, breach width, side slope, manning's roughness coefficient and breach Weir coefficient.

4.4.1 Effect of breach time

In this section the increased and decreased breach time by 15% are compared with the calculated breach time. Breach time has more impact on peak discharge than the other breach parameters. When breach width is constant (87m) as far the present study then with the 15% increase in

breach time (0.667hr) there was decrease in peak discharge through cross section and with 15% decrease in breach time (0.493hr) there was increase in peak discharge.

Breach time is the time of development of breach fully in the dam structure. The more magnitude of peak discharge will result in more peak water level in the downstream. When the breach time is increased or decreased almost the same peak water level along the downstream location is observed. Figure 4-18 and 4-19 shows the effect of breach time on peak discharge and water level.

Table 4-1: Dam break modeling for different breach time for Fincha’a River

Breach time (hr)	Bottom width (m)	Side slope (H:V)
0.667	87	2:1
0.58	87	2:1
0.493	87	2:1

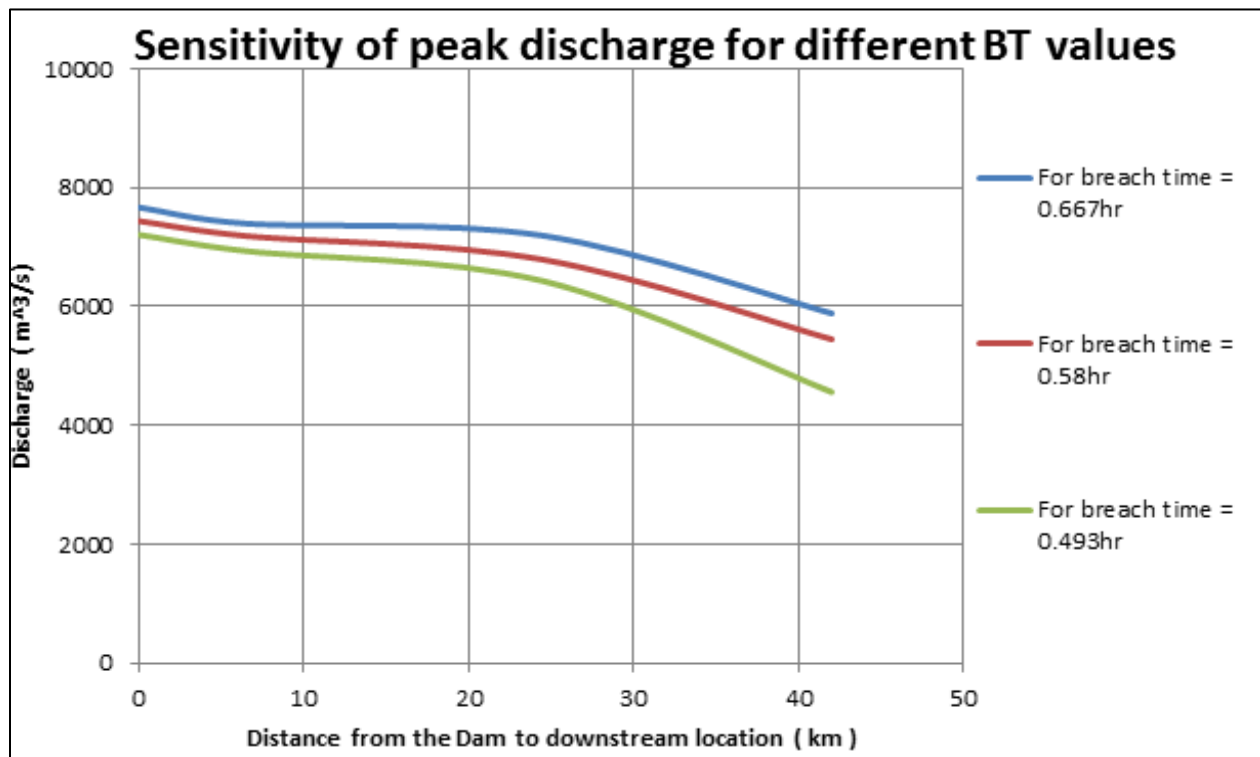


Figure 4-18: Sensitivity of breach time on peak discharge of Fincha’a River

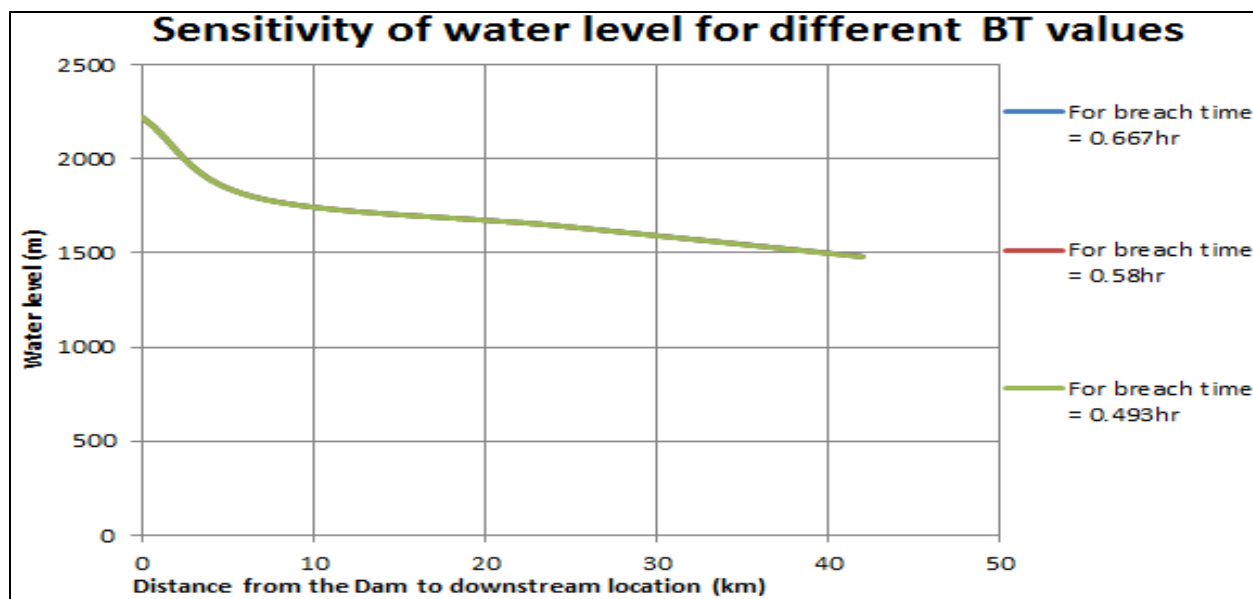


Figure 4-19: Sensitivity of breach time on maximum water level of Fincha'a River

4.4.2 Effect of bottom breach width

In this sensitive parameter dam break model is analysed by changing breach bottom width and keeping constant the other parameters. Increasing and decreasing bottom breach width by 15% and keeping breach time constant and the results obtained is analysed as how much the breach width will affect the peak discharge and water level downstream of the valley comparing with the calculated bottom width by Von Thun and Gillette regression equation. When bottom breach width is increased by 15% from 87m to 100.05m the peak discharge is increased and when it decreased by 15% then decrement of peak discharge is noticed through downstream cross section. So, with the change of bottom breach width there is slightly increment and decrement of peak discharge and almost the same peak water level along the downstream location is observed. Fig. 4-20 and 4-21 shows the effect of bottom breach width on discharge and water level.

Table 4-2: Dam break modeling for different breach width of Fincha'a River

Bottom breach width(m)	Breach time (hr)	Side slope (H:V)
100.05	0.58	2:1
87	0.58	2:1
73.95	0.58	2:1

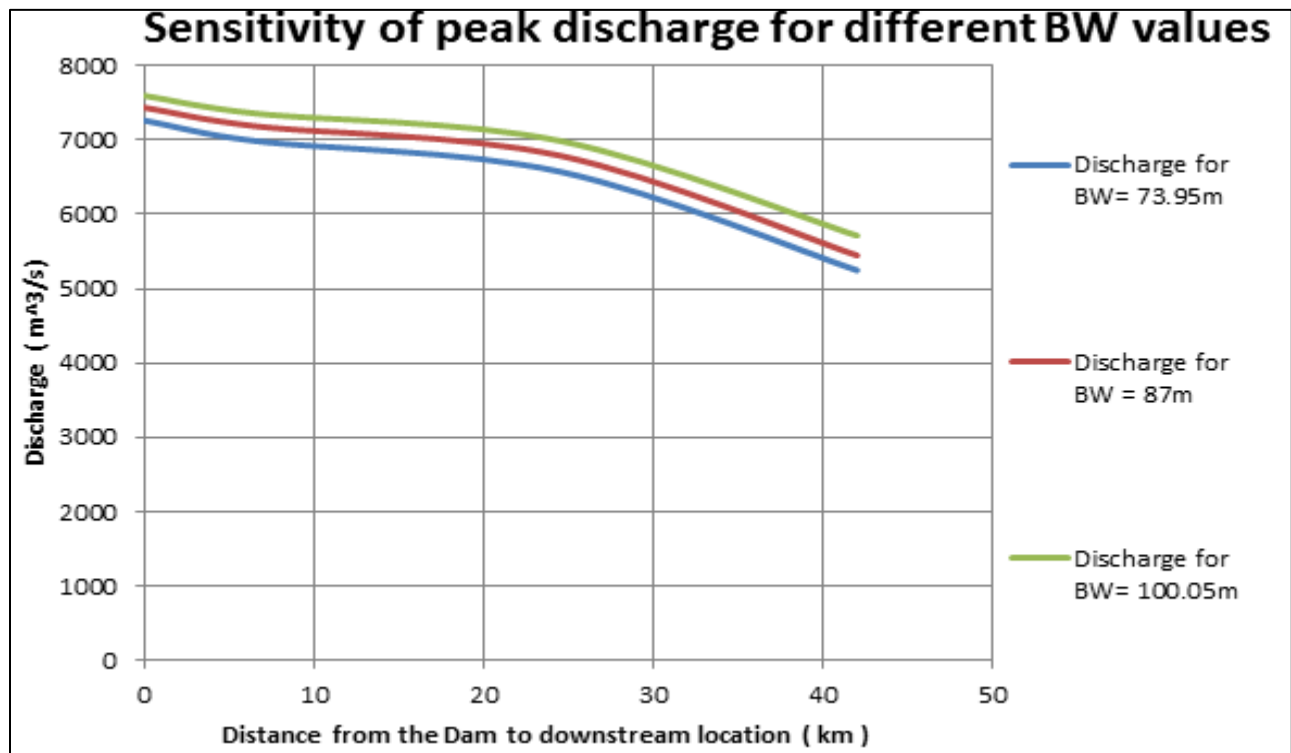


Figure 4-20: Sensitivity of breach width on peak discharge of Fincha'a River

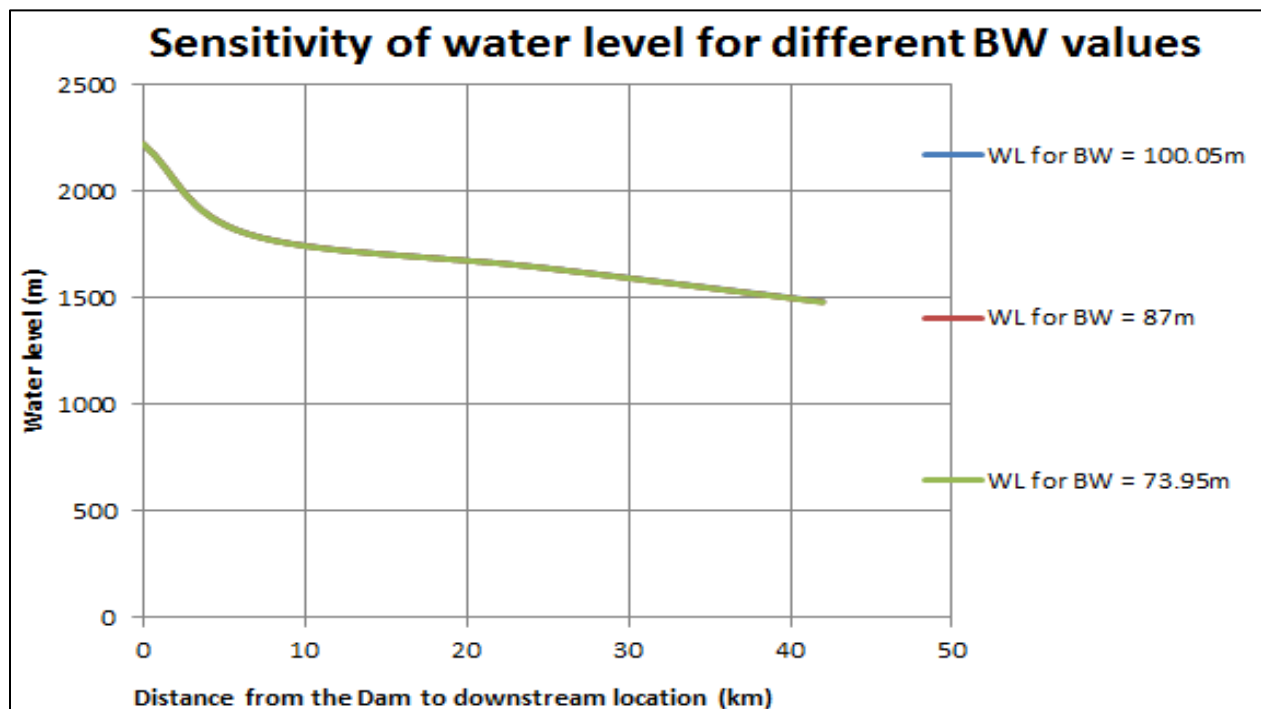


Figure 4-21: Sensitivity of breach width on maximum water level of Fincha'a River

4.4.3 Effect of side slope

The side slope is the lateral slope of trapezoid of the breach section. The model is test for the side slope of 0.4, 0.5 and 0.6 (which mean decreasing and increasing side slope by 10% respectively). Results obtained from these models shows not much change in the values of discharge and water level for the downstream location. So, the sensitivity of this parameter has insignificant effect on the peak values of water level and discharge.

4.4.4 Effect of breach weir coefficient

In this sensitive parameter dam break model is analysed by changing breach weir coefficient and keeping constant the other parameters. When breach weir coefficient is increased by 10% from 1.8m to 1.98m the peak discharge is slightly increased and when it decreased by 10% from 1.8m to 1.62m then peak discharge is slightly decreased. So, with the change of breach weir coefficient there is slightly increment and decrement of peak discharge and almost the same peak water level along the downstream location is observed. Fig. 4-22 and 4-23 shows the effect of breach weir coefficient on discharge and water level.

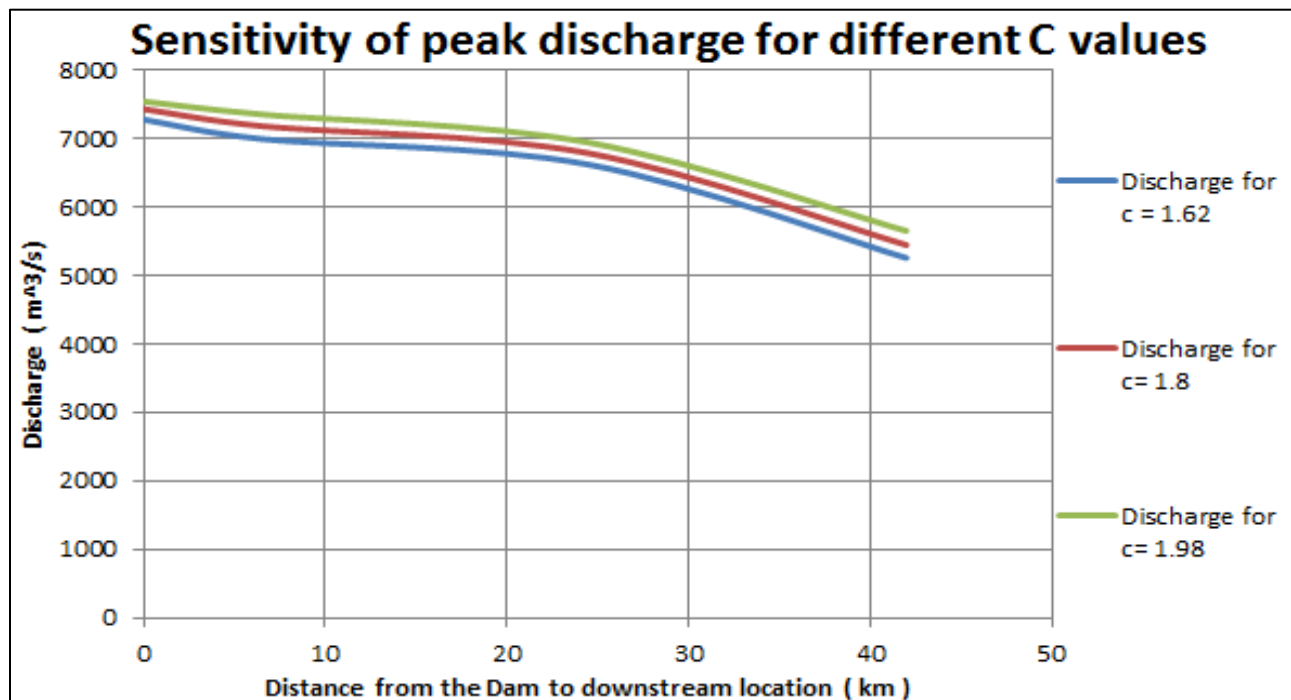


Figure 4-22: Sensitivity of breach wear coefficient on peak discharge of Fincha'a River

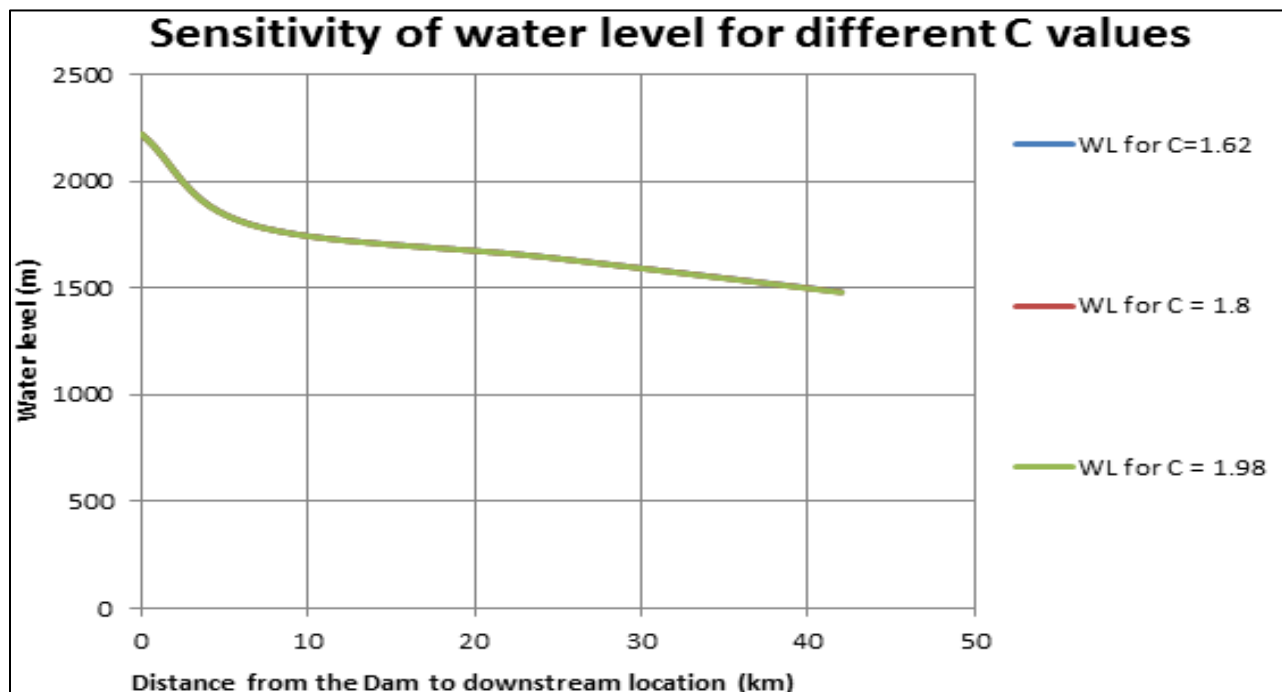


Figure 4-23: Sensitivity of breach weir coefficient on maximum water level of Fincha'a River

4.5 Emergency action plan

Development of effective emergency action plans require accurate prediction of inundation levels, top width and the time of flood wave arrival at a given location where there is infrastructure and population at risk. The arrival time for flood wave to reach the critical location along the flood path required for emergency action plans is determined by calculated velocity and water surface elevation in the HEC-RAS model. Since the peak discharge produced during dam break is high from dam site to 24.5km downstream of the dam, it needs effective EAP and when we go further to downstream location the requirement of emergency action plan is significant. Figure 4-24 and 4-28 shows flood inundation map of Fincha'a dam break by overtopping and piping mode which used for planning effective and urgent emergency action. Figure 4-25 to 4-27 shows inundation map of Fincha'a dam break by overtopping with description of risk measurement parameters. Depending on this condition dam owners and emergency management authorities give urgent protective measures. Figure 4-29 to 4-33 shows different images like falls, power house, towns, irrigation area and sugar factory. Those images are affected if dam breach occurs either by

overtopping or piping mode of failure. Table 4-3 shows the maximum water level, arrival time flood and top width of peak discharge at specified location downstream of the dam.

Table 4-3: Maximum water level, top width and arrival time at different location downstream of the dam when dam break

Downstream distance (km)	Max. water depth (m)		WSE (m)		Top width (m)		Time of arrival (hr)	
	OTM	PM	OTM	PM	OTM	PM	OTM	PM
6.5	3.63	3.42	1798.68	1797.87	752.69	752.55	4	4
24.5	2.67	2.61	1642.24	1642.06	1909.76	1909.25	7	7
42	1.74	1.7	1480.38	1480.1	2074.51	2074.01	10	10

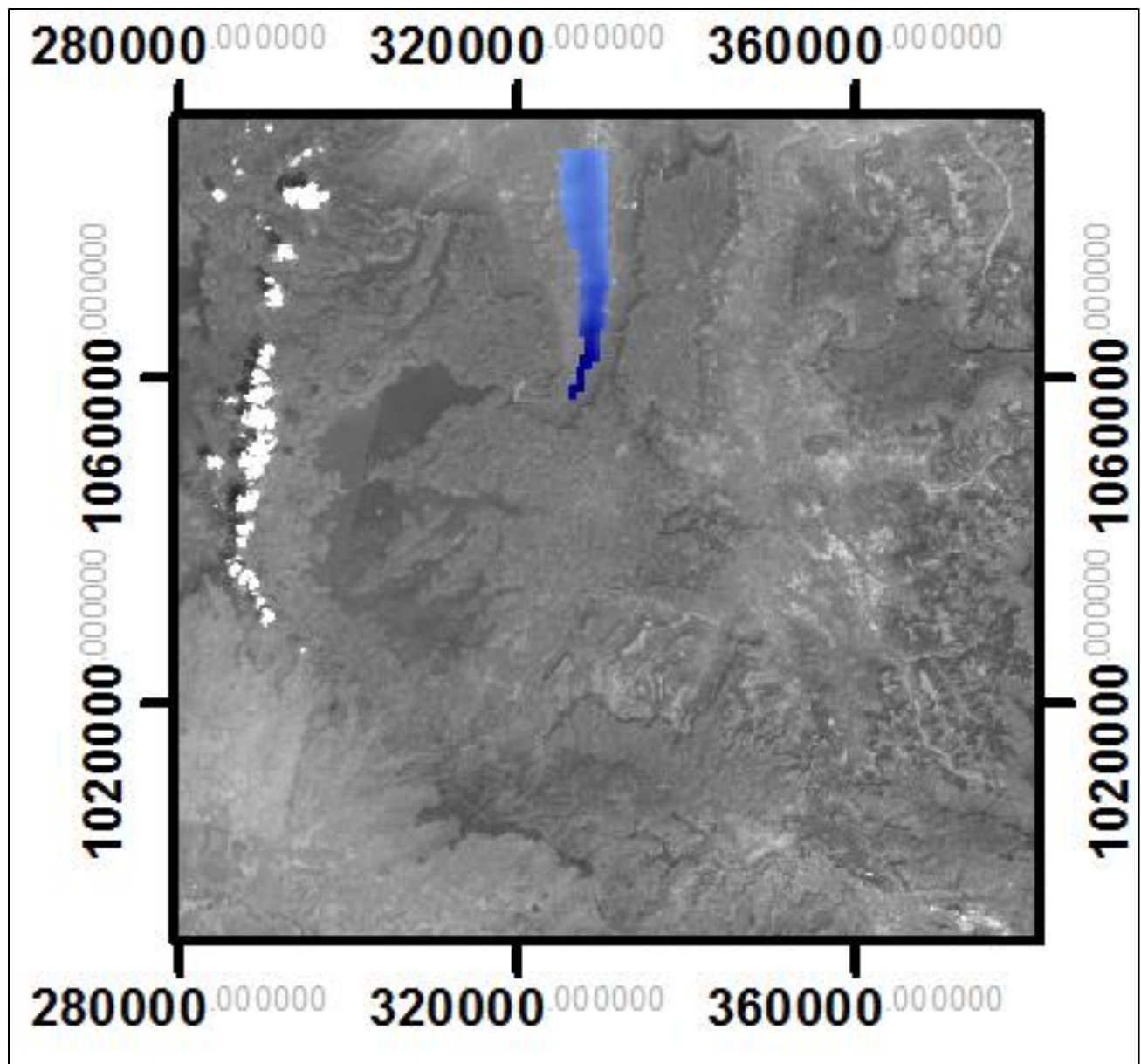


Figure 4-24: Flood inundation map of Fincha'a dam break by overtopping mode between the dam and 42km to downstream location

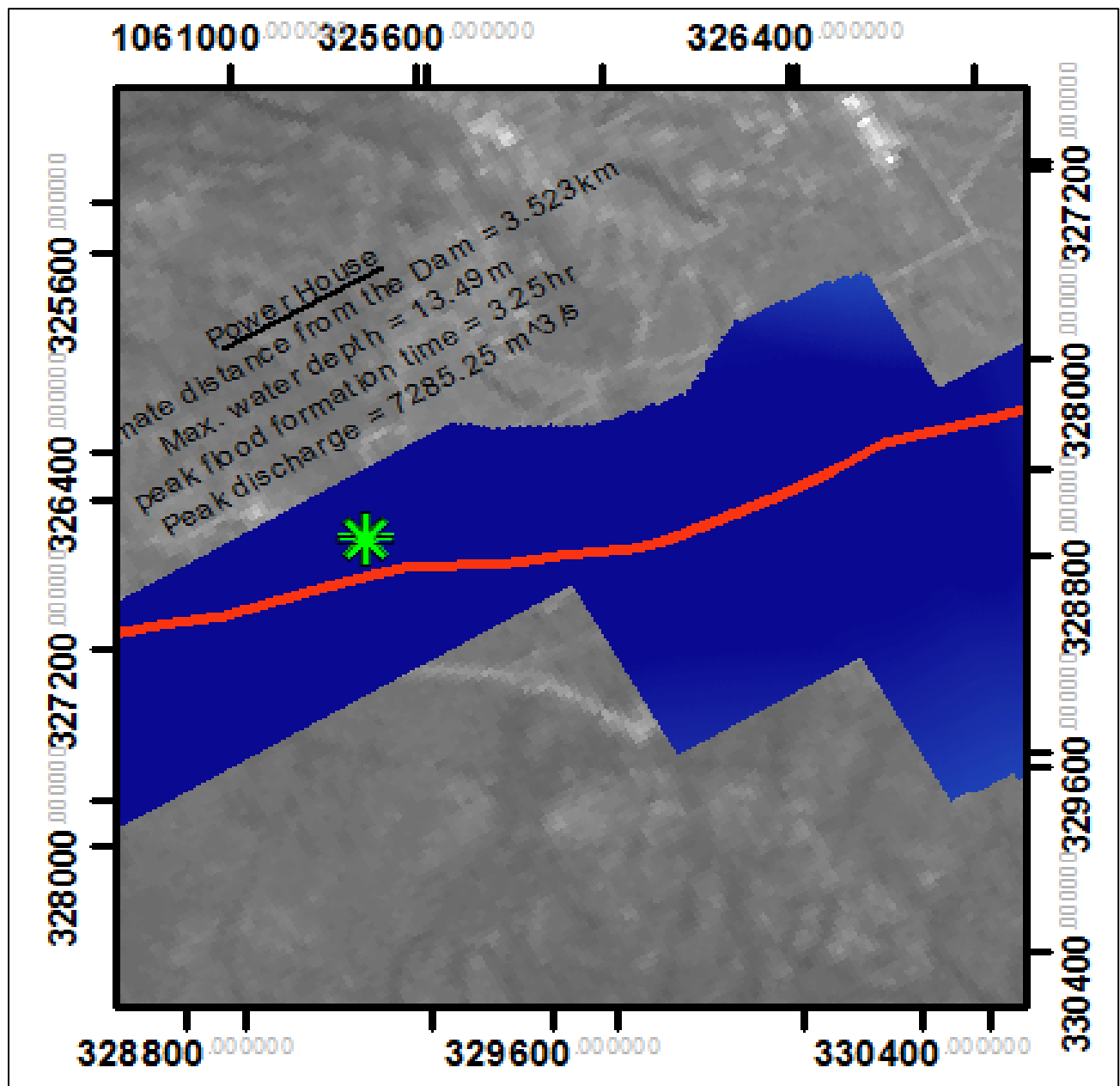


Figure 4-25: Flood inundation map of Fincha's dam break by overtopping that shows flood coverage over power house

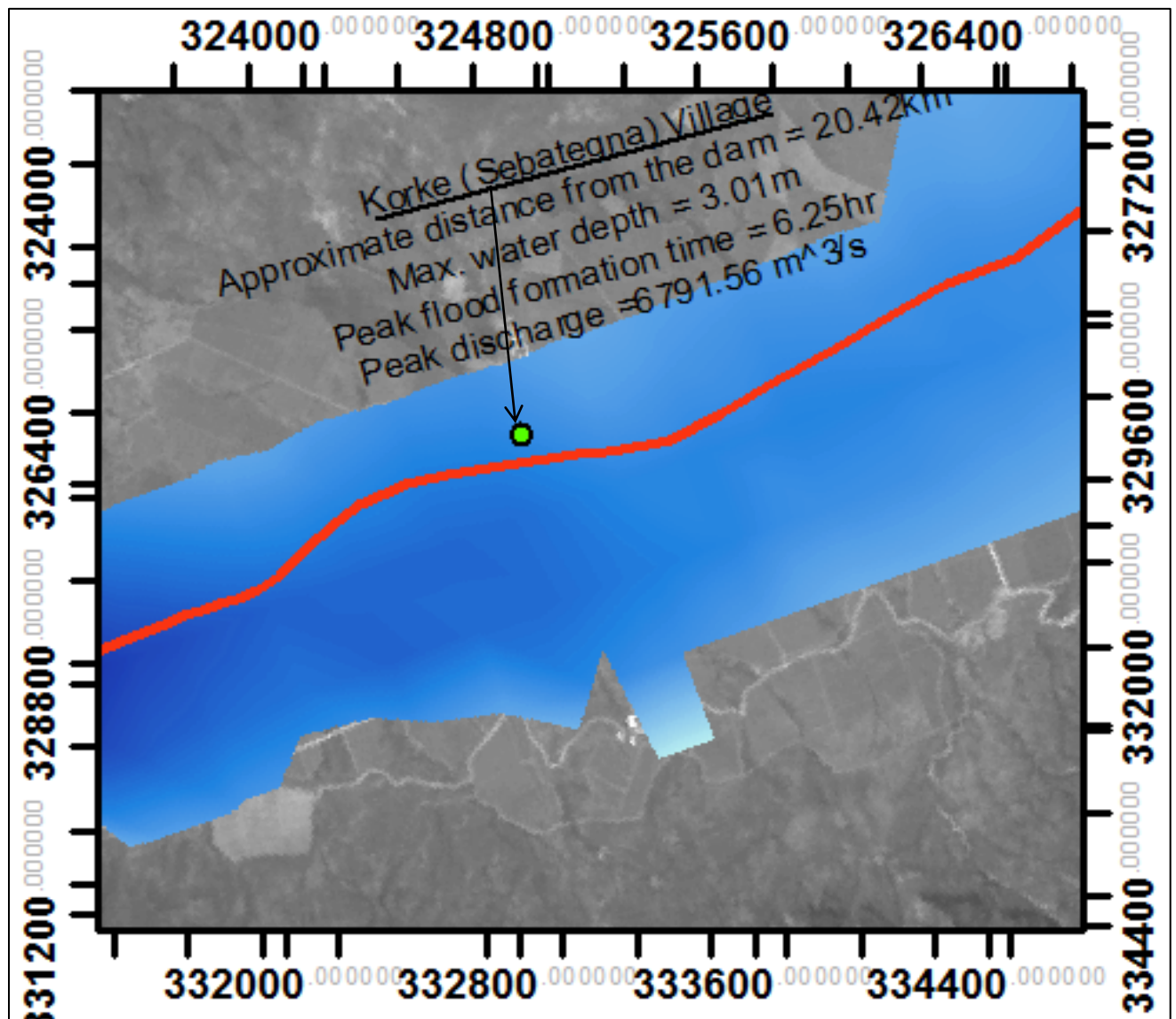


Figure 4-26: Flood inundation map of Fincha's dam break by overtopping that shows flood coverage over Korke (Sebategna) village

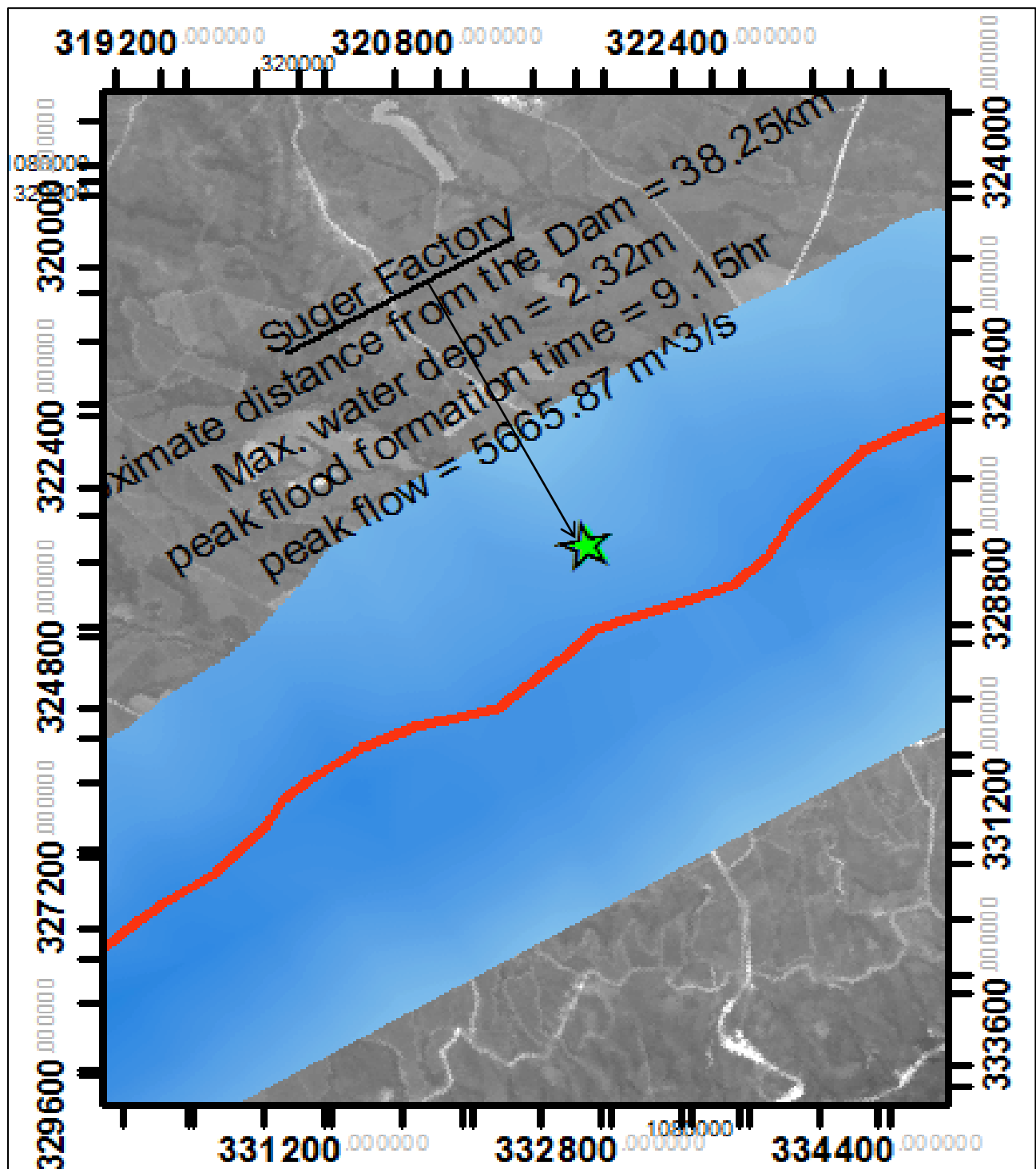


Figure 4-27: Flood inundation map of Finch'a dam break by overtopping that shows flood coverage over Finch'a sugar factory (Hagamsa town)

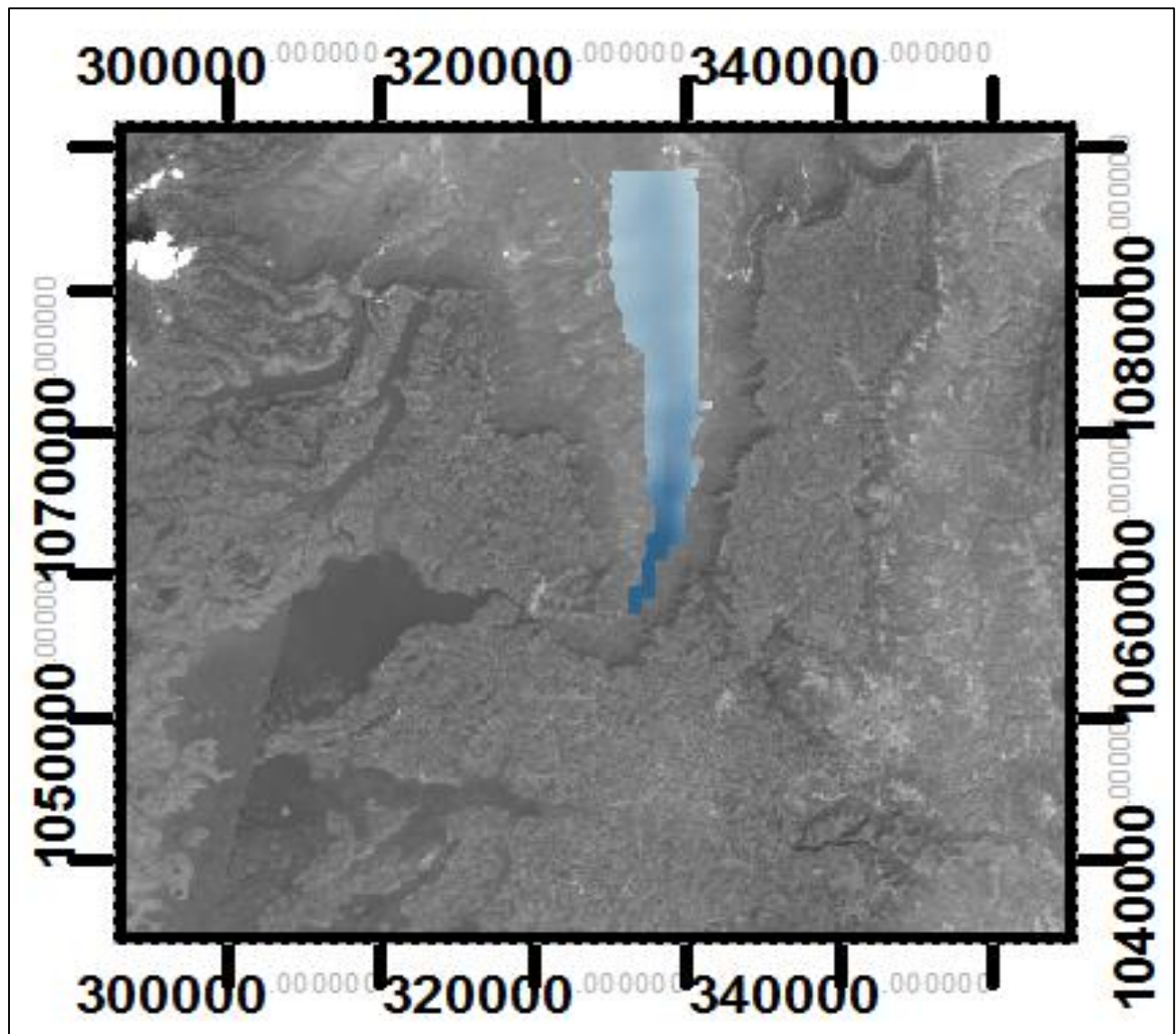


Figure 4-28: Flood inundation map of Fincha'a dam break by piping mode between the dam and 42km to downstream location



Figure 4-29: Fincha'a river fall downstream of the dam



Figure 4-30: Fincha'a Hydroelectric Power House downstream of the fall

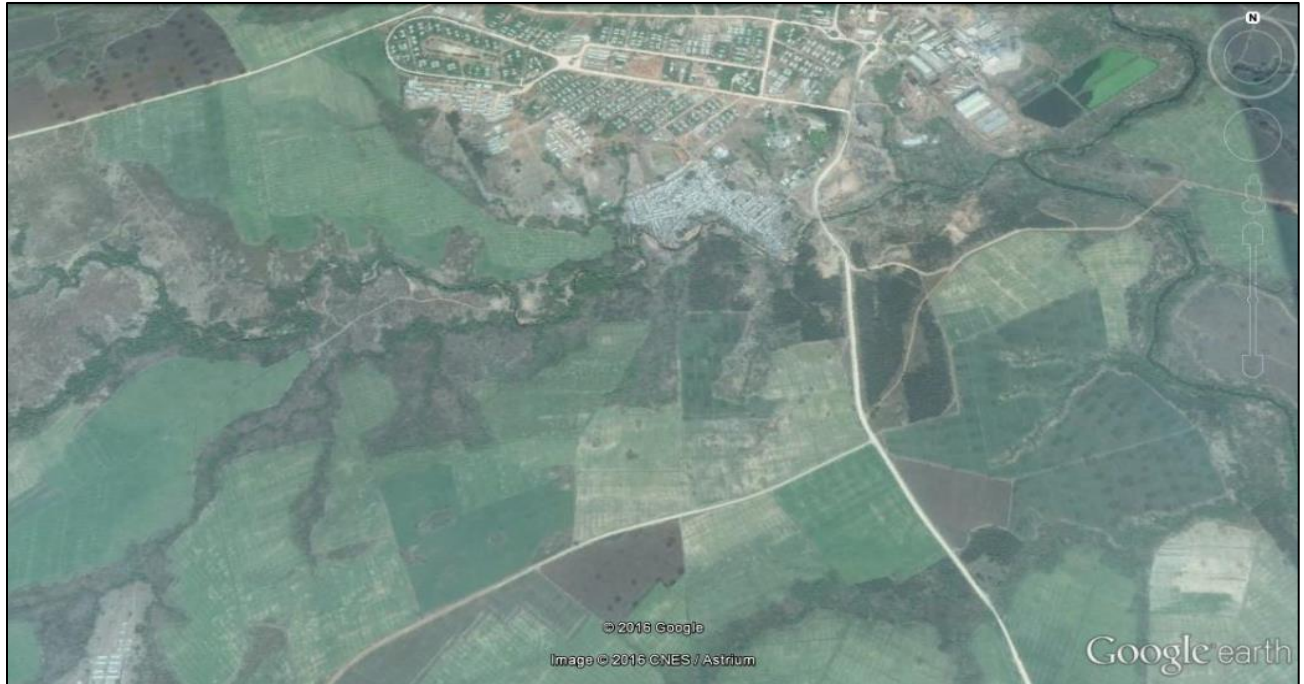


Figure 4-31: Korke town and some of irrigation area downstream of power house

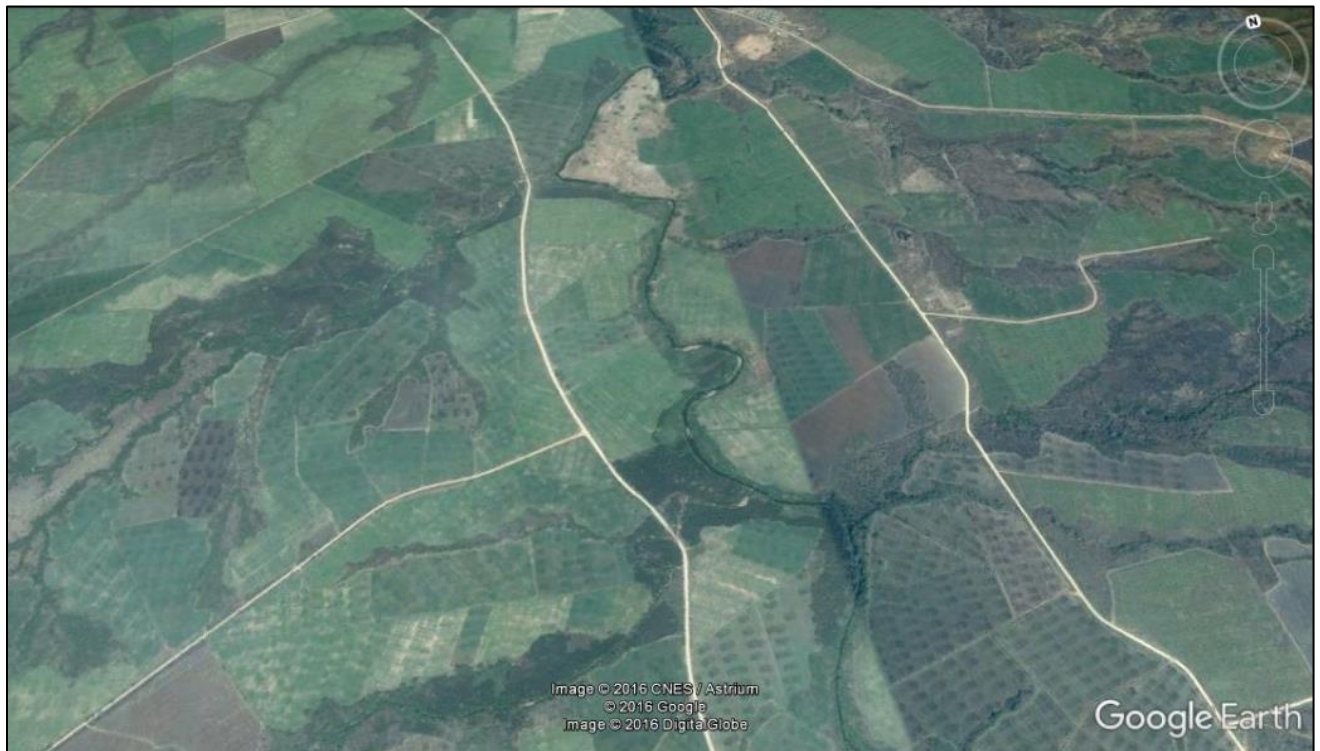


Figure 4-32: Some of irrigation area downstream of korke town



Figure 4-33: Fincha’a sugar factory and Hagamsa town

5 CONCLUSIONS AND RECOMENDATIONS

5.1 Conclusions

The impact of dam break in the downstream area is observed in terms of flood hydrograph, flood arrival time, water level, velocity and flood map. Further the sensitivity analysis of breach time, bottom breach width, manning roughness and breach weir coefficient is carried out for overtopping mode since the result measured in terms of flood hydrograph and water level is greater than piping mode. In accordance with sensitivity analysis, the change of breach time, bottom breach width, manning roughness and breach weir coefficient had no much influence on the downstream maximum water surface elevation. As input parameter data are different therefore the result obtained are different. So, conclusions are drawn by comparing the results as written below.

- ❖ In case of Fincha'a dam the peak discharge by overtopping mode of failure is $7436.8\text{m}^3/\text{s}$ which is greater than peak discharge $7241.22\text{m}^3/\text{s}$ which produced by piping mode of failure.
- ❖ Since the peak discharge resulted from dam break by overtopping mode of failure is greater than that of piping mode of failure we can conclude that the dam break by overtopping mode will develop more risk.
- ❖ The power house, different downstream villages, towns, irrigation areas and Fincha'a sugar factory are affected if the dam failed either by overtopping or piping mode of failure.
- ❖ From sensitivity analysis we conclude that the effect of breach time on discharge is more pronounced than the water level.
- ❖ The increment of water level is insignificant since the surface is flat and water flows to both sides with increasing top width.
- ❖ From sensitivity analysis the effect of breach width on discharge is more than that of the water level.
- ❖ The degree of sensitivity of breach time is more pronounced than that of other breach parameters on discharge than water level.

- ❖ If the dam break unfortunately it affects the population those live downstream of the dam especially around the river. Because of this condition Finchaa dam is classified under high hazard dam for loss of human life is expected to result from the failure of the dam.
- ❖ Since peak discharge and maximum water depth resulted from dam break is great for 24.5km downstream of the dam, effective and urgent action plan is required for the area nearer to the river.
- ❖ Since the probable maximum flood of Fincha'a River is greater than the spillway capacity the probability of occurrence of dam failure is high.
- ❖ Developed inundation map guides the dam owners and emergency management authority to give emergency action plan for the highly affected area by flooding and used for planning future economic development area.
- ❖ The breach development time calculated by Von Thon and Gillette regression equation is the same for overtopping and piping mode of failure. Therefore, the arrival time of flood at different chainage points downstream of the dam is the same for similar chainage points of overtopping and piping mode of failure.

5.2 Recommendations

The economic development activities including power house, irrigation area and residential house downstream of the Fincha'a dam should be developed with minimum distance of 450m from both river banks.

The dam owners and emergency management authority should guide communities those living and farming downstream of the dam near the river by using map of flood prone areas.

For overtopping mode of failure increasing depth of freeboard is the solution to protect the dam failure.

Increasing depth of freeboard and designing adequate spillway capacity are some of the solutions to protect overtopping mode of failure.

Using selected material with sufficient compaction during dam construction is also the solution to protect piping mode of failure.

The data used for the dam break analysis should be primary data to obtain more accurate results.

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APPENDIX

Table appendix-A-1 Flood hydrograph at different chainage points downstream of the dam by overtopping

Time (hr)	Discharge at the Dam and at different location from the Dam			
	At Dam	6.5km	24.5km	42km
0	150	145	48.26	39.48
1	250	175.12	150.25	120.23
2	6556	188.71	163.53	157.92
3	7436.8	6037.67	185.43	191.26
4	7122.16	7187.45	242.15	160.75
5	6041.94	6887.45	5935.98	162.02
6	5027.6	6287.45	6489.69	205.02
7	4225.79	5336.56	6589.69	350.76
8	3477.14	4541.27	5624.57	4884.55
9	2862.61	3834.63	4755.26	5333.1
10	2390.68	3311.1	4182.98	5448.86
11	2051.02	2912.89	3731.66	4844.81
12	1688.49	2562.3	3339.93	4284.32
13	1442.52	2192.26	2924.29	3815.14
14	1242.94	1885.23	2563.98	3447.53
15	1059.31	1652.36	2155.21	3062.07
16	882.16	1428.23	1825.8	2720.32
17	785.71	1205.24	1611.55	2456.14
18	616.84	1015.23	1418.01	2236.01
19	528.08	821.23	1201.63	2030.67
20	428.13	647.35	960.28	1812.42
21	359.58	543.13	812.18	1639.67
22	329.15	416.35	641.77	1419.74
23	296.41	345.65	575.56	1276.57
24	273.29	320.23	560.1	1165.87

Table appendix- A-2 Flood hydrograph at dam and at different chainage points downstream of the dam for piping mode of failure

Time (hr)	Discharge at the Dam and at different location from the Dam			
	At Dam	6.5km	24.5km	42km
0	150	150	52.45	34.89
1	156.23	175	150	120
2	524.12	188.71	163.53	157.92
3	7241.22	5637.67	185.43	191.26
4	6921.15	6387.45	242.15	160.75
5	5900.55	6654.64	4835.98	162.02
6	4645.93	5787.45	5389.69	205
7	3712.93	4936.56	5689.69	350
8	2945.91	4241.27	5074.57	3884.55
9	2512.55	3634.63	4455.26	4483.1
10	2134.46	3111.1	3882.98	4804.84
11	1816.09	2712.89	3431.66	4344.81
12	1589.64	2362.3	3039.93	3784.32
13	1405.06	2052.26	2724.29	3415.14
14	1207.75	1745.23	2363.98	3147.53
15	1052.48	1542.36	2055.21	2862.07
16	902.53	1288.23	1825.8	2620.32
17	763.03	1055.24	1611.55	2356.14
18	615.03	845.23	1418.01	2136.01
19	502.12	631.23	1221.63	1930.67
20	423.83	507.35	970.28	1712.42
21	322.84	403.13	812.18	1439.67
22	252.21	326.35	671.77	1319.74
23	216.56	265.23	595.56	1176.57
24	206.89	220	560.1	1065.87