Dam Break Flood Inundation Modeling for Mount Coffee Dam

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DAM BREAK INUNDATION MODELING
FOR MOUNT COFFEE DAM

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By

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This is to certify that the work presented in the dissertation entitled *Dam Break Flood Inundation Modeling for Mount Coffee Dam* submitted by *Sherron Brisbane Sherman*, Roll Number *214CE4004*, is a record of original research carried out by her under my supervision and guidance in partial fulfillment of the requirements of the degree of *Master of Technology in Water Resources Engineering*. Neither this dissertation nor any part of it has been submitted earlier for any degree or diploma to any institute or university in India or abroad.

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Declaration of Originality

I Sherron Brisbane Sherman, roll number: 214CE4004 hereby declare that this Master's degree thesis, Entitled Dam Break Flood Inundation Modeling for Mount Coffee Dam" was carried out as a postgraduate student of NIT, Rourkela and to the best of my knowledge, it contains no material previously published or written by another person, nor any material presented for the award of any other degree or diploma of NIT Rourkela or any other institution. Any contribution made to this research by others, with whom I have worked at NIT Rourkela or others elsewhere, is explicitly acknowledged in the dissertation. Works of other authors cited in this dissertation have been duly acknowledged under the section "References". I have also submitted my original research records to the scrutiny committee for evaluation of my dissertation.

I am fully aware that in case of any non-compliance detected in future, the senate of NIT Rourkela may withdraw the degree awarded to me on the basis of the present dissertation.

May 2016

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Abstract

Dam break analysis is crucial for investigating future effects posed to human life and property by a sudden release of water to the inundation area of a dam. Every constructed as well as proposed dams need to be analyze for the possibility of dam break because even with advanced technology, failure cannot be rooted out based on the huge level risks associated with it. This study aims at establishing the worst-case scenario at the Mt. Coffee dam as a result of overtopping. The impacts are determined using numerical 1-dimensional software (MIKE 11). The flood condition is prompted by the Probable Maximum Flood (PMF) of the basin which is inputted as a time dependent external boundary condition into the reservoir. Accuracy in this study is vital to instituting foundation for further investigations on Emergency Action Plan and Risk Management among others. Efficient dam break analysis relies on high precision of breach parameter. To arrive at this result, two widely used and well recommended breach prediction parameter methods are used in this research. The Federal Energy Regulatory Committee (FERC) and Froelich-2008 regression breach prediction methods are compared to yield outflow hydrographs, travel time of flood from the onset of the overtop to downstream locations, travel time from peak outflow to inhabited locations downstream, velocity of flood, water levels, and attenuation in discharge downstream of the dam break. The sensitivity of the breach is also tested by interchanging prediction parameters such as breach width, breach formation time, and breach slope channel. By establishing the inflow design flood, it has been proven that the Mount Coffee dam has a high possibility of failure due to the inadequacy of spillway capacity.

Keywords: ***Dam-breach; Flood impacts; Inundation zone; MIKE11 Software; Mount Coffee Dam, Worst-case scenario.***

Table of Contents

List of Figures

List of Table

List of Abbreviations

A: Active flow area [m^2]

asl: Above sea level [m]

A_0: Inactive storage area [m^2]

B: Breach bottom width [m]

C: Chezy's coefficient [$m^{1/2}/s$]

$C_{orifice}$: Orifice coefficient (0.599769)

C_{slope}: Weir coefficient for slope parts; (0.431856)

C_v : Correction coefficient (cover up for energy loss to the inflow contraction)

C_{weir}: Weir Coefficient for horizontal parts; (0.546430)

D_c : Dam crest height [m]

DHI: Danish Hydraulic Institute

f: Friction factor

FERC: Federal Energy Regulatory Commission

g: gravitational acceleration [m/s^2]

h: Water level upstream (reservoir water level) [m]

h_1: Upstream water level [m]

H_b: Breach level [m]

H_d: Dam crest height [m]

h_{ds}: Downstream water level [m]

h_p: Centerline of pipe; (h_{pt} +h_b)/2 [m]

h_{pt}: Top of pipe [m]

K_o: Constant used in Froehlich's breach width equation

K_s: Submergence correction coefficient

L: Embankment length (crest length) [m]

L_b: Breach length in flow direction[m]

M: Manning number which is equivalent to the inverse of manning's n

Q: Discharge (Flow) [m^3/s]

Q_p: Flow through the pipe [m^3/s]

q : *Lateral outflow [m^3/s]*

q_t: *Sediment transportation rate [m^2/s]*

R: Hydraulic Radius [m]

S: Breach slope

S_c : *Expansion contraction slope*

S_f : *Friction slope*

t: time [s]

u : *Current velocity [m/s]*

uf: *Fiction velocity [m/s]*

$V_{s.}$: *Volume of water behind the dam [m^3]*

W_{ave}: *Average embankment width (perpendicular to the crest) [m]*

x: Distance along the channel [m]

Z_{obv}: *Overt of the pipe [m]*

λ: *Darcy's resistance factor*

Φ: *Sediment transport rate (dimensionless)*

θ: *Total shear stress [P_a]*

ϵ : *Sediment porosity*

Chapter 1

Introduction

1.1 Overview

The construction of dams is highly necessary and is growing rapidly around the world for the purpose of providing electricity, flood control, Water storage, recreation, navigation, etc. It produces low environmental impacts, low operational and maintenance cost (Kaygusuz, 2004). With the numerous benefits of dams, new technologies and designs, the possibility of dam break cannot be eliminated because since the inception of dams, dams have been failing in association with: spillway capacity, landslide, Seismic resistance, Quality of design, Nature of the foundation, Quality of construction, Monitoring, Maintenance and human factors (War, terrorism, etc.). Dam Break is the failure of a dam leading to uncontrollable release of concentrated water to the downstream which can be disastrous to life and property. In the 20th century, approximately 200 dam failures have occurred in the world claiming about 8000 lives and millions of dollars damages.

Vaiont in Italy in 1963 killed about 2000 people, Machhu II dam failure, India in 1979-about 2000 people, Malpasset Concrete dam in France in 1959 led to 433casualties, in Southern Germany the failure of a dam in 1999 caused 4 deaths and damaged properties worth billions of Euro (R. Mathew, 1997). Due to hazard pose by Dam Break, Inundation analysis at every dam is highly relevant in predicting, managing and minimizing the risk to flood zone downstream of a dam.

In this study, a MIKE 11 hydrodynamic unsteady model is setup for the Mount Coffee Dam for the purpose of predicting the outflow and impacts of a dam breach by routing the outflow flood through the stream to determine the water surface profile at different locations along the river network (Harding, 2001). MIKE 11 fully dynamic unsteady model provides a highly accurate hydraulic model involving time series data. It uses the 1-Dimensional implicit difference model for unsteady flow base on the St. Venant continuity and momentum equations. A hypothetical breach at Mount Coffee will facilitate a precise Risk Management and Emergency Action Plans for the downstream. An increase in populations along the St. Paul river stream is expected of an increase after the rehabilitation of the Mount Coffee Hydropower plant.

1.2 Dam Breaching – Theoretical Background

1.2.1 Dam Breaching Mechanisms

Before leading research on dam breaching modeling is discussed in more detail, it is important to understand the main causes of dam breaching. This section explains the main reasons dam failures occur and how they develop. There are three major types of earthen dam failures. They are: overtopping, foundation defects, and piping. According to Costa's statistics in 1985, 34% of all dam failures were due to overtopping, 30% to foundation defects, and 28% to piping; leaving the balance 8% of the dam failures to other miscellaneous acts or processes.

1.2.2 Overtopping Failure

Overtopping is the most common type of dam failure. It occurs when the water levels or waves are higher than the crest of the dam and it usually follows storm events where inflow raises the reservoir level above the spillway capacity. This could be caused due to inadequate design, construction and maintenance, debris blocking the spillway, settlement causing the dam crest to be lowered, or a dam section of the crest is built lower than other (Task committee on Dam/Levee Break, 2010). Dams are constructed with different compaction sediments; therefore their failure processes may be significantly different. In a homogeneous, non-cohesive dam, the mechanism of failure is sediment transport. Sediment began to erode near the crest of the dam at the downstream end causing a steeper slope. The stage is described by the upstream erosion of the downstream slope which narrows the crest width further and eventually the dam crest is lowered due to down cutting and lastly by lateral erosion, the breach widens and the dam collapse (Task committee on Dam/Levee Break, 2010). Wahl (1998) describes the first two stages as one stage and calls it the "breach initiation". The breaching process for a dam constructed of homogeneous, cohesive sediment is significantly different. This is because the erosion mechanism is the head cut or vertical drop erosion. The Task committee on Dam/Levee Break (2010) still describes this breaching process as occurring in four stages. The first stage is when the initial overtopping occurs, which results in sheet and rill erosion. These rills develop into large over falls and eventually cause large head cuts in the downstream crest. The second stage is described by the headcut reaching the upstream part of the crest. The third stage lowers the crest of the dam by down cutting and finally, the fourth stage widens the initial breach and again, the mass failure occurs. The task committee believes that the third and fourth stages are very similar for cohesive and non-cohesive sediments even though the erosion modes and mass failure occur very differently.

The Task Committee on Dam/Levee Break (2010) states that the overtopping failure of dams made out of composite sediments is not the same as dams constructed out of homogeneous sediments. They believe when overtopping occurs on a dam with clay, steel, or concrete core, erosion starts on the downstream slope either by sediment transport or headcut that advances until it reaches the core. This erosion may affect the stability of the core and cause it to fail. Common failures of the core include sliding, overturning and bending. The core would then wash away downstream and the breach would increase until mass failure occurs. If the cover is less erosive than the core, the cover may erode first and the core would only erode at the areas where the cover has eroded.

1.2.3 Piping Failure

Piping is another common type of dam failure. Piping occurs from seepage or leakage through weak layers, structure joints, dead tree roots, and animal burrows in the embankment. For piping to occur, the water level does not need to reach the height of the dam crest. It is possible for seepage to soften the material in the body of the dam and cause large volumes of the dam to slide as slurry. It is most common for a "pipe" to be formed from one end of the dam to another. The erosion within the pipe causes parts of the dam to slump and eventually collapse from the weight and water pressure. After the collapse, the breach acts very much like an overtopping breach. This includes both the down cutting and then widening. The piping failure takes much longer to occur than overtopping failure. Piping failure can take days but overtopping failure takes hours or less.

1.2.4 Foundation Defects

Foundation defect is the last major type of dam failure include differential settlement, sliding and slope uncertainty, high uplift pressure, and unrestrained foundation seepage. Where differential settlement occurs, often cracks and weak layers are found throughout the dam. These cracks and weak layers can lead to internal erosion which often results in piping failure. When there is a lot of seepage passing through the foundation sand boils are possible. Uplift pressure is another major foundation defect that could cause instability to the dam slope and the dam may slide. Sliding defect is crucial and can form an instantaneous failure faster than overtopping and piping failure. Sliding breach is usually rectangular in shape and covers the entire dam height (Singh, 1996).

1.3 Background of Study Area

The Mount Coffee Hydropower Plant (MCHPP) is located on the St Paul River about 25 km upstream of Monrovia with a catchment area of 19,992 Km^2; located in Liberia, West Africa. The climate is tropical with two seasons, six months of rainy season and six months of dry season. Dry season extends from December to April while the rainy season is from May to November. Maximum annual rainfall is 3800mm and minimum annual rainfall is 1768mm. The St Paul River has a length of about 500 km and originates at Diani River in south-eastern Guinea. It flows in a south-westerly direction through Liberia and empties into the Atlantic Ocean. From 1973 to 1990, hydropower generation contributed 98% of the country's electricity until fore bay dam 1 experienced a breach in August 1990 and due to the inability to access the catchment area during the crisis, there were no statistics collected for further analysis of the breaching of the dam. The rehabilitation of the Mt. Coffee hydropower dam is in progress and is expected to be completed in 2018. Mount Coffee Hydropower plant is the largest of the three hydropower plants with an expected upgraded installed capacity from 64-80MW. Notwithstanding, Liberia has a hydropower potential of 2000MW.

Figure 1-1 Location of Mount Coffee Hydropower Plant in Montserrado County

Source: Hatch 2012

1.4 Thesis Research Objectives

Due to hazardous threats pose to human lives, infrastructures, floodplains, and livestock by dam failures, precision of dam break flood magnitude and propagation time at different downstream locations of the dam are essential for mitigation measures. To achieve this, the aim of this research is to accurately:

1. Determine the outflow flood magnitude through the dam as a result of overtopping failure.
2. Simulate the variations in discharge, velocity and water level at downstream locations for the purpose of estimating the effects of the flood wave at these populated locations.
3. Establish an inflow design flood for the Mount Coffee spillway.
4. Illustrate the flood inundation area resulting from routing the flood wave through the downstream.

1.5 Thesis Outline

This thesis commences with a brief description of the importance, effects and causes of dam break. Next, vital contributions of researchers in the field are described and with much emphasis on research that contributes to outflow and breach parameter predictions based on numerical or physical investigations. The thesis then explains the numerical computer model mechanisms in dam break investigation, and two methods of breach parameters (FERC and Froehlich, 2008) to facilitate the result. In chapter 6, the sensitivity of breach parameters are analyzed and various effects are specified. Appendix (a) deals with the evaluation of spillway capacity at the study area. Details of the investigation are outlined and resolutions are made for the future.

Chapter 2

Literature Review

2.1 Introduction

During extreme events, all dams experience added forces on them which increase the risk potential of failure therefore dam breach modeling is conducted to predict the outflow hydrograph due to the breach and to route the hydrograph to the downstream of the channel to get the maximum water level and discharge along with time at different locations downstream of the dam.

There are three techniques followed in analyzing dam break. They are as follows: Regression modeling technique where historical data of dam failures are evaluated using dam and reservoir properties to predict peak outflow and hydrograph shape directly. The next technique is the analytical modeling technique, utilizing physical dam model characteristics to make failure predictions. And the last is the numerical modeling technique which involves routing flood wave by means of computer software.

2.1.1 Regression Model

Regression model technique is the most popularly used for dam break analysis for embankment dam breach peak prediction analysis. Simple regression technique evaluates the relationship between peak outflow through the breach and depth and volume of water behind the dam at failure. Table 2.1 shows different prediction equations, type of statistical curve fit, and number of case study used in the analysis. Variables in relationship to empirical equations include: Q_p = peak outflow (m^3/s), h_w = height of the water behind the dam at failure (m), h_d = height of the dam (m), S = reservoir storage at normal pool (m^3), and V_w = volume of the water behind the dam at failure (m^3). Parameters input for different regression equations by different investigators can be represented slightly differently. I.e. Effective head can be represented differently depending on the investigator, (h_w) height of water behind the dam or (h_d) height of the dam; volume of outflow through the breach can be represented as volume of water behind the dam (V_w) or reservoir storage (S). Time to failure (t_f) of the breach is also analyzed using regression technique. Figure 2-2 from the Department of the Interior Bureau of Reclamation Dam safety shows the Prediction of Embankment Dam Breach Parameters by

Froehlich 1995, Von Thun and Gillette 1990, MacDonald and Langridge-Monopolis 1984 and Reclamation 1988. These regression techniques can be used along with computer models.

Table 2.1 Previous studies of peak-outflow Prediction

	Investigator	Type	R^2	Number of Case Study Real	Number of Case Study Sim.	Equation
Height of water equations	Kirkpatrick (1977)	*Best − fit*	0.790[a]	13	6	$Q_p=1.268(H_w+0.3)^{2.5}$
	SCS (1981) for dam	Envelope[b]	Not available	13		$Q_p=16.6(H_w)^{1.85}$
	USBR (1982)	Envelope	0.724	13		$Q_p=19.1(H_w)^{1.85}$
	Singh and Snorrason (1982)	*Best − fit*	0.488	21	8	$Q_p=13.4(H_d)^{1.89}$
	Pierce et al. (2010)linear	*Best − fit*	0.633	72		$Q_p=0.784(H)^{2.668}$
	Pierce et al (2010)curvilinear	*Best − fit*	0.640	72		$Q_p=2.325\ In(H)^{6.405}$
Storage equations	Singh and Snorrason(1984)	*Best − fit*	0.918		8	$Q_p=1.776(S)^{0.47}$
	Evans (1986)	*Best − fit*	0.836	29		$Q_p=0.72(V_w)^{0.53}$
	Pierce et al (2010)	*Best − fit*	0.805	87		$Q_p=0.00919(V)^{0.745}$
Height of water and storage equations	Hagen(1982)	Envelope	Not Available	6		$Q_p=1.205(V_w.H_w)^{0.48}$

MacDonald and *Langridge* – *Monopolis*(1984)	*Best – fit*	0.805	87	$Q_p=0.00919(V)^{0.745}$
Froehlich (1995)	*Best – fit*	0.934	22	$Q_p=0.607(V_w^{0.295}.Hw^{1.24})$

[a]This R^2 value was calculated using a portion of the writer's original data set.
[b]Wahl (1998) suggested that this is an enveloping equation even though three data points plots slightly above the curve.
[c]This R^2 value was calculated without the five concrete and masonry dams included in the writer's original data set.

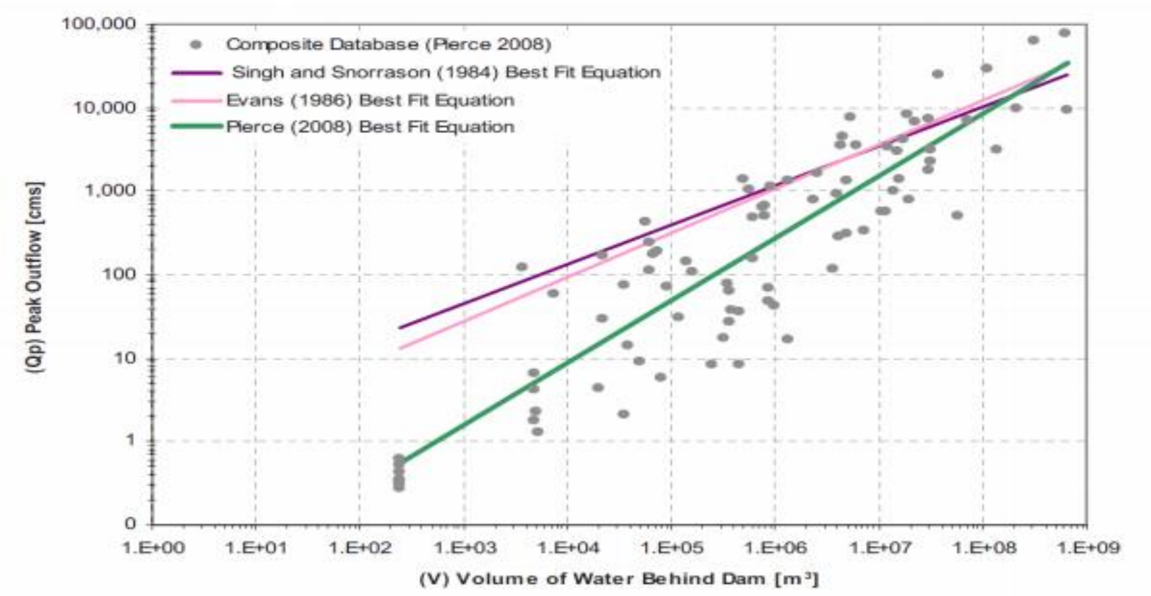
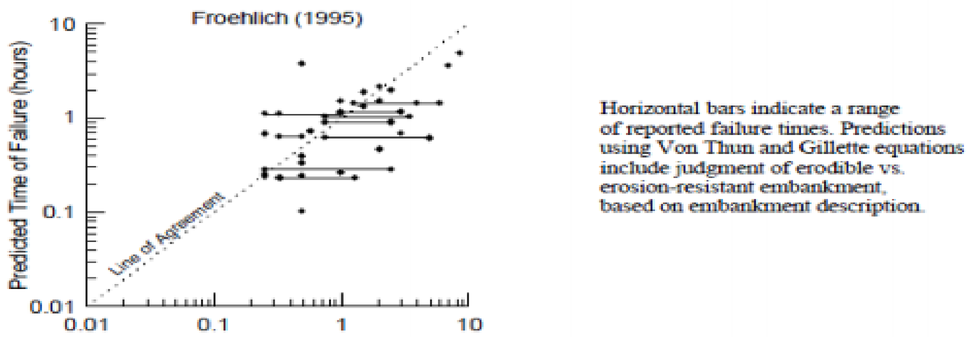


Figure 2.1 Dam failure data sets (Thornton et al. 2011).



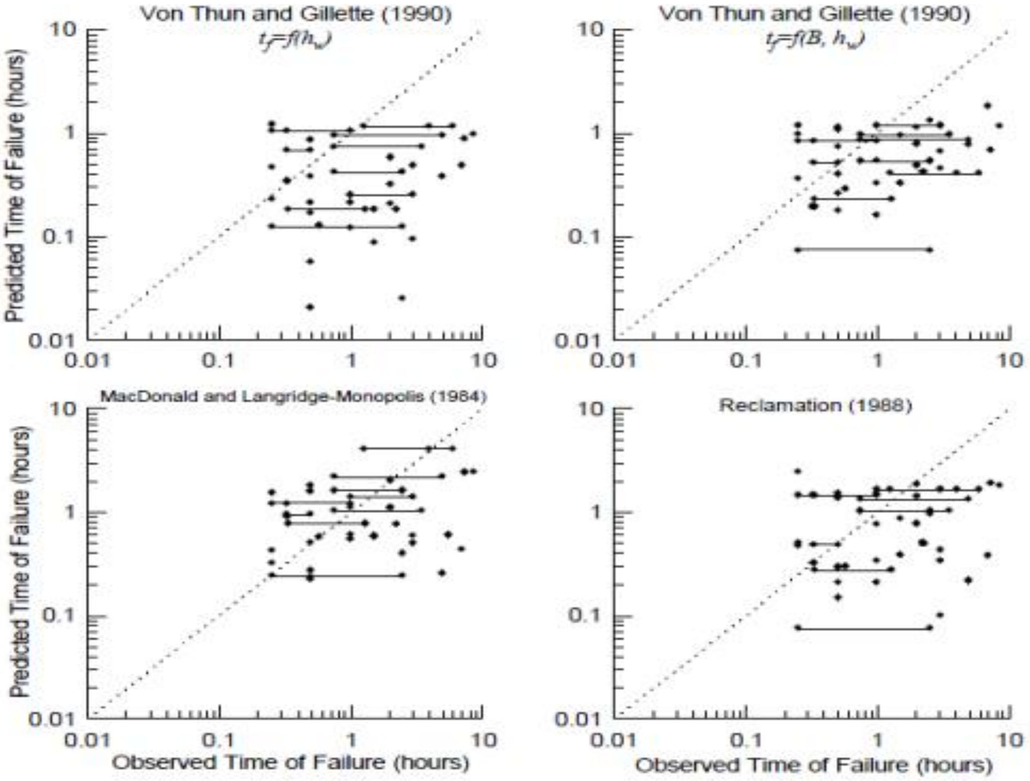


Figure 2-2 Predicted vs. Observed time of failure (Wahl 1998)

The relationship between the dam failure data set in figure 2-2 is multivariate and the peak discharge equations developed for a breach include:

$$Q_p = 0.863(V_s^{0.335}\, H_d^{1.833}\, W_{ave}^{-0.633}) \tag{2.1}$$

$$Q_p = 0.012(V_s^{0.493}\, H_d^{1.205}\, L^{0.226}) \tag{2.2}$$

In Equations 2.1and 2.2: $V_{s.}$ = volume of water behind the dam (m^3)

H_d = dam crest height (m)

W_{ave} = average embankment width (m) (perpendicular to the crest)

L = embankment length(m) (crest length)

When the pertinent dam characteristic variables are up to three as in the equations, the coefficient of variation increased slightly and the main predicted error and the uncertainty bandwidth decreased (Thornton 2011).

In 2004 Wahl investigation found Froehlich (1995a) equation to have the lowest uncertainty of the peak flow prediction equations. The advantage of the regression model is that it's simple and not time consuming making it useful in the analysis of large dam inventories and comparing results estimated from other methods but to its disadvantage, this model do not consider factors related to material erodibility and time parameters prediction even though help to define the shape of the hydrograph but do not evaluate the warning time prior to the peak outflow.

2.1.2. Analytical Model

The analytical model is based on sets of equations formulated of the physics of dam erosion and hydraulics. The discharge through the breach is related to the rate of erosion by using an equation sensitive to shear strength of the soil particles and the force of the flow of water. Using this model, it is assumed that a trapezoidal breach of constant side slope, bottom width of the breach resulting from the angle of repose of the material and bottom slope of the breach channel is equal to the internal angle of friction. Cristofano's (1965) work is known to be the first physically dam base model. A mathematical model for peak discharge was developed by Walder and O'Connor (1997) as a function of reservoir size, material erosion rate, breach shape parameter, breach side slope angle, reservoir shape factor, and the breach depth to dam height ratio (Wahl 2010). See equation below: Table 2.2 shows some physical based embankment dam breach models.

$$Q_p = 1.51\left(g^{1/2} h_d^{5/2}\right)^{0.06}\left(\frac{k_b V_s}{h_d}\right)^{0.94}, t_p \approx 1.24\left(\frac{V_s}{k_b^{2}(g h_d)^{1/2}}\right)^{1/3} \ldots for \left(\frac{k_b}{(g h_d)^{1/2}}\right)\left(\frac{V_s}{h_d^{3}}\right) <\sim 0.6 \qquad (2.3)$$

$$Q_p = 1.94\left(g^{1/2} h_d^{5/2}\right)\left(\frac{D_c}{h_d}\right)^{3/4}, t_p = \frac{h_d}{k_b} \ldots for \left(\frac{k_b}{(g h_d)^{1/2}}\right)\left(\frac{V_s}{h_d^{3}}\right) >> 1 \qquad (2.4)$$

In Equations 2.3 and 2.4:

- g = gravitational acceleration (m/s^2)
- h_d = water level drop in reservoir (m)
- k_b = mean erosion rate of the breach
- V_s = volume of water behind the dam (m^3)
- D_c = dam crest height (m)

Equation 2.3 is used on dams where reservoir volume stored to dam height ratio is small while equation 2.4 is used for where reservoir volume store to dam height ratio is large. The advantage of this model is that it identifies the difference in behavior of small and large reservoirs. In small reservoirs, the peak flow occurs while the breach is still forming and large reservoirs breach occurs when the breach is formed fully and at maximum head. Unlike other techniques, analytical does not initiate breach time only breach formation.

Table 2.2 Analytically based embankment dam breach models

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

2.1.3. Numerical model

Numerical breach model is a process used to determine the outflow hydrograph, duration and dimension of a dam failure. A Dam failure formation can reach the riverbed or stop at the middle of the dam body. The speed formation and dimensions of the breach determine the size, shape and outflow through the breach. A breach dimension is the depth and width of the breach and the speed formation refers to the time it takes for the breach to form. A breach model is based on erosion, hydraulic principles, dam geometry, dam materials, surface mechanics, reservoir properties, and amounts of inflow into the reservoir at a time. The complexity of breach modeling is crucial to all Hydraulic engineers to ascertain an accurate result.

Breach model depends on the dam properties which may likely be distributed. The distribution of dam properties affects the size, shape, duration formation and outflow of the flood through the breach. Therefore, sensitivity analysis and critical dam material assessment are to be carried out by engineers in analyzing a dam breach. What happens if the materials or properties of the dam are not homogeneous? Table 2.3 shows dam properties of outer section and inner core materials and their characteristics are to be considered as well as if the surface of the dam is spouted, the grass quality must be taken into account (Seker, D. Z. et. al, 2003).

Table 2.3 Dam Properties and materials

Properties related to the material	Characteristics related to the structure
a. Internal friction angle	a. Downstream and upstream slope of dam
b. Cohesion stress	b. River bottom slope
c. Mean grain diameter (D_{50})	c. Crest level weight
d. Density	d. Spillway level and capacity level
	e. Inflow hydrograph
	f. Reservoir surface area curve
	g. Initial surface level

Further guidance for predicting breach parameters (e.g., duration of formation, geometry) have been developed by researcher from case study data.

MGS Engineering Consultants Inc. (Rev. 2007):

Outlines Middlebrooks study of 200 earth dam failures, the catalogue of these failures showed that 50 percent of failure occurred within 5 years and 19 percent at the time of first failed. Also, the Guidelines follows the principle used by Wahl from U.S. Army Corps of Engineers (USACE) and Fread to specify empirical procedures and numerical model used to predict embankment dam, Concrete gravity dams breach parameters. Details of MGS research is shown in the table below.

Table 2.4 MGS Breach Parameters

Dam Type	Average Breach width (expressed as dam height)	Side Slope of Breach Z_b (Z_b Horizontal:1vertical)	Failure time (Hours)
Earth fill Dam	Min: 0.4 Max: 13 Mean: 4	Min: 0 Max: 6 Mean: 1	Min: 0.1 Max: 12 Mean: 2
Concrete Gravity Dam	Integer Multiple of Monolith Widths	Vertical	0.1 t0 0.5
Concrete Arch Dam	Entire Valley Width	Valley Wall	0 to 0.1

FERC refers to the U.S Federal Energy Regulatory Commission Guideline. The FERC guideline is widely used and accepted by the National Weather Service guideline (NWS). The FERC guideline is shown in Table 2.5. This guideline is also used as the UK dam break Guidelines.

Table 2.5 FERC and UK Dam Break Guideline

DAM TYPE	**AVERAGE BREACH WIDTH (m)**	**FAILURE TIME (hr)**	**BREACH SIDE SLOPE H:1V**	**AGENCY**
Earthen/ Rock fill	(0.5 to5.0) x HD (1.0 to 5.0) x HD (2.0 to 5.0) x HD	0.5 to 4.0 0.1 to 1.0 0.1 to 1.0	0 to 1.0 0 to 1.0 0 to 1.0	USACE (2007) FERC (1988) NWS(Fread, 2006)
Concrete Gravity	Multiple Monoliths Usually ≤ 0.5 L Usually ≤ 0.5 L	0.1 to 0.5 0.1 to 0.3 0.1 to 0.2	Vertical Vertical Vertical	USACE (2007) FERC NWS (Fread, 2006

Froehlich (2008) developed a model estimating the average breach width (B), average slope (z) and the breach formation time (t_f), with the use of linear regression analysis making use of 74 historic embankment dam failure data. His equations were formulated with various dam and reservoir parameters including: reservoir water elevation (V_w), critical overtopping depth (H_c), and height of breach (H_b).

$$B_{avr} = 0.27K_oV_w^{0.32}H_b^{0.04}$$

Where K_o = 1.3 for overtopping failure and 1.0 for other failure modes

V_w = volume of the reservoir at the time of failure

Z = 1.0 for overtopping failure and 0.7 for other failure modes

The slope relation was formulated from equation 2.6:

In z = - 0.416 + 0.389 X Mode

Breach formation time approximation equation:

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

Figure 2.3 and 2.4 show the comparison of measured and predicted breach width values and breach formation time in Froehlich research. Other equations formulated by other researcher are shown in Table 2.6.

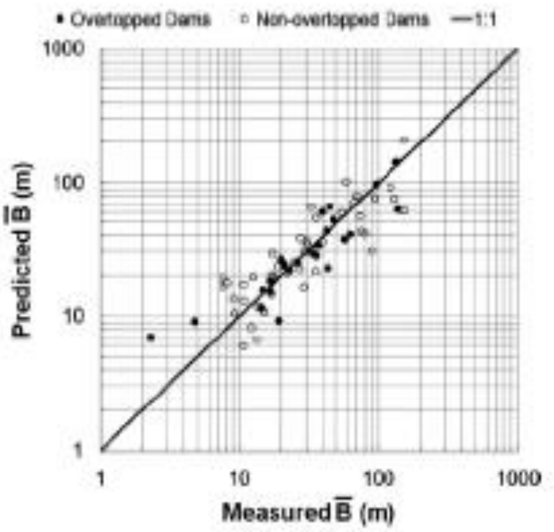


Figure 2-3 Comparison of measured and predicted average breach width

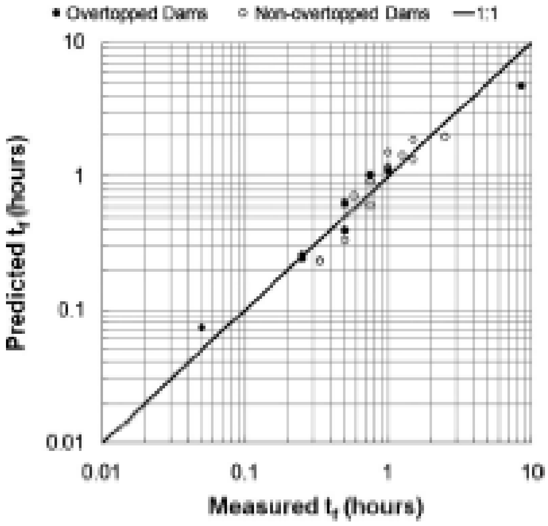


Figure 2-4 Comparison of measured and predicted average breach formation time (Froehlich 2008).

Table 2.6 Breach Parameter relations based on dam failure case studies

Reference	**Number of Case Studies**	**Relations Proposed (S.I. units, meters, m^3/s hours)**
Johnson and Illes (1976)		$0.5h_d \le B \le 3.0h_d$ for earthfill dams
Singh and Snorrason (1982, 1984)	20	$2h_d \le \boldsymbol{B} \le 5h_d$ $0.15m \le d_{overtop} \le 0.61m$ $0.25h_r \le t_f \le 1.0h_r$
MacDonald and Langridge-Monopolis (1984)	42	Earthfill dams $V_{er} = 0.0261(V^*_{out}h_w)^{0.769}$ (best-fit) $t_f = 0.0179(V_{er})^{0.364}$ (upper envelope) Non-earthfill dams $V_{er} = 0.034(V^*_{out}h_w)^{0.852}$ (best-fit)

FERC (1987)		B is normally 2-4 times h_d B can range from 1-5 times h_d Z = 0.25 to 1.0 (engineered, compacted dams) Z = 1 to 2 (non-engineered, slag or refuse dams) t_f = 0.1 to 1.0 hour (engineered, compacted dams) t_f = 0.1 to 5.0 hour (non-engineered, poorly compacted dams)
Froehlich (1987)	43	$B = 0.47K_o(s^*)^{0.25}$ K_o = 1.4 overtopping; 1.0 otherwise $Z = 0.75K_c(h_w^*)^{1.57}(w)^{0.73}$ K_c = 0.6 with corewall; 1.0 without a corewall $t_f = 79(s^*)^{0.47}$
Singh and Scarlatos (1990)	52	Breach geometry and time of failure tendencies B_{top}/B_{bottom} averages 1.29
Von Thun and Gillette (1990)	57	B, Z, t_f guidance (see discussion)
Dewey and Gillette (1993)	57	Breach initiation model; B, Z, t_f guidance
Froehlich (1995b)	63	$B = 0.1803K_oV_w^{0.32}h_b^{0.19}$ $t_f = 0.00254V_w^{0.53}h_b^{(-0.90)}$ K_o = 1.4 for overtopping; 1.0 otherwise

Singh and Snorrason (1982):

Concluded that variation of breach width vary from 2 to 5 times the height of a dam, he stated that generally, complete failure time is 0.25 to 1hour and for overtopping failures, the maximum overtopping depth prior to failure ranged from 0.15 to 0.61 meter from an analysis of 20 dam failures.

T. C. MacDonald and J. Langridge-Monoposis (1984):

Concluded that computer programs (HEC-1and DAMBRK) developed for dam safety analyses are limited by the accuracy of data input for geometric and temporal breach characteristics based on analysis conducted on numerous historical dam failure in relation to breach characteristics.

MacDonald and Langridge-Monopolis (1984) Concluded from a 42 site case study that the side slopes of a trapezoidal or triangular breach formation is 1:2 depending on if the breach reached the base of the dam.

Duke M. Mojid (1999):

Mojid developed a mathematical model for simulating the gradual failure of earthen dams, due to overtopping. The model is based on continuity, sediment transportation equations and a breach shape geometric descriptor.

P. P. Mujumdr (2001):

He explained the propagation of flood wave along an open channel.

F.H. Jaber and S. Shukla (2007):

Suggested that the one dimensional Saint Venant equations are suitable to simulate the standing waves and other degeneration that occurs in the reservoir and that accuracy of the simulations depend on the courant numbers used in the simulation.

Pramanik, N., Panda, R. K., & Sen, D. (2010):

Pramanik et. al. Used Digital Elevation Model (DEM) to extract 40 cross section along the reaches of the Brahmani River for simulating the magnitude of flood which result showed a close agreement between the simulated and observed stage hydrograph.

In 2009, Xu and Zhang applied a multi-parameter nonlinear regression analysis to a very large database of case studies which produced a very significant result on the effects of erodibility.

Gupta, S. K., & Singh, V. P. (2012).Proposed a new equation that could better predict peak discharge through breached dam embankment in a case study of 87 dam breach using the multivariate regression data analysis to incorporation the height of water level (h), water volume at failure time(v) and average embankment length (L) or Width (W) as three independent variables.

$Q_p = 0.02174 \ V \ 0.4738h1.1775 \ (W+L) \ 0.17094$

Chapter 3

Dam Break Modeling

3.1 Computer routing methods for dam break

Dam break modeling is significant in the field of hydraulic engineering. Modeling a dam break includes: a) Outflow hydrograph prediction b) Routing the outflow through the downstream of the channel for estimating maximum water levels, discharge and arrival time along the channel and c) Identifying the flood inundation zone. Studies on dam break flood routing model have advanced over the past decade and can be simulated using computers; several comparative and available computer programs (1-dimansional and 2-dimensional) have been developed for computing outflow hydrograph through a dam break and routing the flood wave downstream of the breach including: DAMBRK (Fread, 1988b), (FLDWAV, Fread, 2000), HEC-RAS (HEC, 2006a), MIKE 11 by DHI, and so forth. One study shows that the National Weather service models, Dam-Break Hood Forecasting Model (DAMBRK) and FLDWAV were the most optimal choice of model used for achieving the most practical level of accuracy in dynamically routing flood waves, however, when compared with HEC-RAS have the same background, numerical solution technique for most conditions and same results when using the same parameter in the models. (Zhou et al, 2005). MIKE 11 model is used in this research to analyze a hypothetical breach at the Mount Coffee Dam. Further details on the MIKE 11 computer program is discussed below.

3.1.1 MIKE 11 by DHI

MIKE 11 is a subset program of packaged software developed by DHI (Danish Hydraulic Institute) for simulating flow; i.e. hydrodynamic, rainfall-runoff, structure operation, dam break, advection dispersion and water quality. It is a 1-dimensional river modeling software, driven by the open channel flow of St. Venant (1971) continuity and momentum equations. Mike 11 takes into account the implicit finite difference scheme for unsteady flow created by Abbott and Ionescu (1967). The 6-point Abbott scheme (see figure 3.1) procedure is organized such that, computational grids are alternating in calculating water level and discharge at each time step, the mass equation (continuity) emphasis on the h-point (water level) while the momentum equation centered on the Q-points (discharge). The software is user friendly and requires user's input choses to set up and run complex 1-D applications. The default iteration of the equations is changeable; therefore, the user has the option of using more iteration in solving the governing equations. From the previous time step result, the first iteration starts and the next is based on the centered value of the first. Also as user oriented, the program requires the following input editors, network editor, cross section editor, boundary condition editor and hydrodynamic editor to simulate a model. MIKE 11 is also programmed to solve any form of the St. Venant equation: Kinematic, diffusive or dynamic. To compute flow passage through structures (dams, bridges, culvert, sluices), the broad crested weir equation is initiated. The software uses the following equations below depending on the user's choose and study scenario.

i. Conservation of mass (continuity) equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \quad \text{-------------------------------------} \qquad (3.1)$$

ii. Conservation of momentum equation

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\frac{Q^2}{A}\right)}{\partial x} + gA\frac{\partial h}{\partial x} + \left(s_f + s_c\right) = 0 \quad \text{------} \qquad (3.2)$$

iii. Kinematic equation

$$gA + \left(s_f + s_c\right) = 0 \quad \text{----------------------------} \qquad (3.3)$$

iv. Diffusive equation (backwater evaluation)

$$gA\frac{\partial h}{\partial x} + \left(s_f + s_c\right) = 0 \quad \text{--------------------} \qquad (3.4)$$

v. Broad crested weir equation

$$Q = C_v K_s\left[C_{weir} b\sqrt{g(h-h_b)}(h-h_b) + C_{slope} s\sqrt{g(h-h_b)}(h-h_b)^2\right] \qquad (3.5)$$

Where:
Q = Discharge
A = Active flow area
q = Lateral outflow
x = Distance along the channel
t = time
g = gravitational acceleration
A_0= Inactive storage area
S_f = Friction slope
S_c = Expansion contraction slope

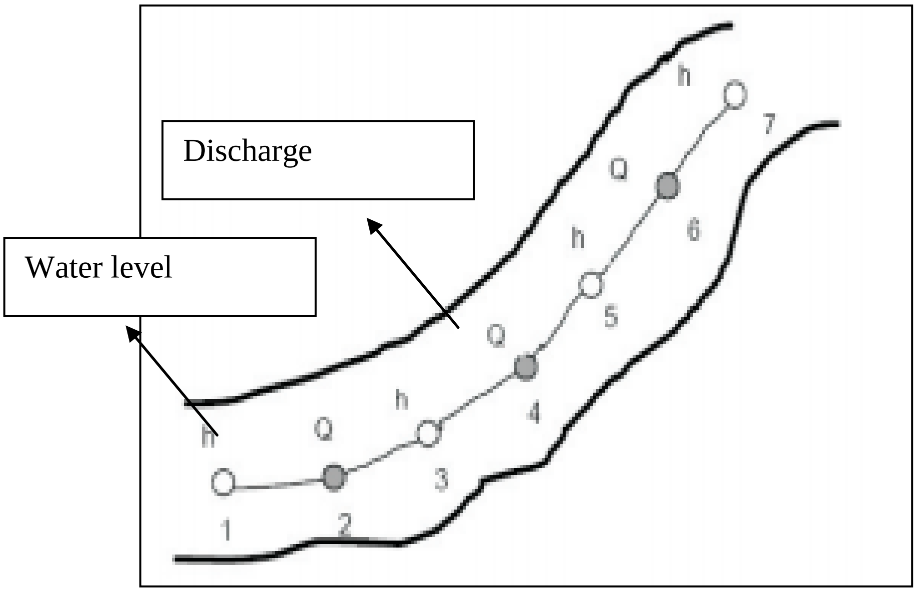


Figure 3-1 Point Abbott Ionescu Scheme

MIKE 11 presupposition

i) The flow is incompressible

ii) The wave length is large compared to water depth, assuming that flow everywhere is parallel to the bed

iii) The bottom slope is small.

The failure mode has to be specified as one of the following:

After the start of the simulation

Date and time

Reservoir level

3.1.1.1 Bed Resistance

The flexibility of the software makes calculated bed resistance diversely. The bed resistance of a channel can be calculated using Chezy's, Manning's or Darcy's equations and the hydrodynamic editor makes it possible to insert single or multiple bed resistance parameters within the channel as applicable to the study area. See bed resistance equations below: equations 3.6, 3.7 and 3.9

Chezy's bed resistance equation

$$\frac{g|Q|Q}{C^2AR} \qquad (3.6)$$

Manning's bed resistance equation

$$\frac{g|Q|Q}{C^2AR^{4/3}} \qquad (3.7)$$

The relationship between Chezy's coefficient and Manning's number

$$C = \frac{R^{1/6}}{n} = M^2R^{1/6} \qquad (3.8)$$

The Darcy-Weisbach coefficient

$$\lambda = \frac{8g}{C^2} \qquad (3.9)$$

Where: ` g = gravitational acceleration

Q = flow

A = cross sectional area of the river

R = Hydraulic Radius

M= manning number which is equivalent to the inverse of manning's n.

λ = Darcy's resistance factor

C = Chezy's coefficient

3.1.1.2 Boundary Condition

In MIKE11, boundary conditions are categories as external and internal boundary conditions. Internal boundary condition considers links at nodal points, structures, internal inflows, and wind friction. On the other hand, External boundary condition considers time varying values for water level (h) or discharge (Q) and relations between h and Q. Model boundaries are to be chosen at points where water level or discharge measurements are available to be used for a predictive reason. Depending on the stream situation and data available a boundary condition can be chosen. An inflow hydrograph or constant inflow into the reservoir upstream, and constant water level or a rating curve downstream are typical set-up for MIKE 11 boundary editor file.

3.1.1.3 HD (Hydrodynamic) Coefficients

The HD editor in MIKE 11 is built with multiple defaults parameters; these parameters can be changed by the user to best fit their study scenario.

Alpha Coefficient: Velocity distribution coefficient in the momentum equation. (Default = 1.0)

DELH Coefficient: During low flow conditions, the top elevation and depth of the slot is controlled by the DELH where the DHLH is off the river bottom up to the depth of 5*DELH. (Default =0.1m)

DELHS Coefficient: DELHS helps prevent instabilities and establish water level difference across a weir or structure when the surface gradient of water changes direction. (Default = 0.01m)

DELTA Coefficient: defines the dissipating influence of the forward center scheme of the term dh/dx. (Default = 0.5 no dissipative effect; maximum value 1.0 has a maximum influence)

EPS Coefficient: With the approximation of the diffusive wave, if the water surface slope is larger than the EPS, the stream becomes upstream centered. (Default = 0.0001)

Froude Exp: Is used in suppression of convective terms in the momentum equation for supercritical flow. Default is applied if there is negative value for Froude Exp. or Froude Max.

Froude Max: Suppression of the convective terms in the momentum equation. By default suppression occurs if a negative value for Froude max inserted.

Inter 1 Max: Stipulates the completed maximum number of iteration in a time step around a structure. (Default = 10)

Max IterSteady: Stipulates the maximum number of iteration for steady initial condition of the water profile. (Default =100)

NODE Compatibility: determines whether water level compatibility or energy level compatibility is calculated at each node. (Default: water level compatibility)

NoITER: States the number of iteration in a time step do derive at a solution (default value = 1).

Theta: The default value of theta is 1; it’s used in the momentum equation to represent the resistance term.

ZetaMin: Stipulates minimum sum of head loss factors around a structure (optional)

3.1.1.4 Cross sections

The cross section is indicated by a cut in the channel perpendicular to the flow which is defined by x and z coordinates. The x coordinate measures the horizontal distance of the cross-section while the z coordinate measures the vertical corresponding elevation of the channel at that cross section. It is advisable to input as many cross sections as possible to adequately detect changes in channel slope or topography.

Depending on the nature of the channel bed, different Manning's coefficient can be allocated at each cross section if needed.

3.1.1.5 Dam-break Structure

In MIKE 11, dam break structures are structures at which the breach is simulated. The simulation of the breach at a dam break structure takes into account all hydraulic occurrences over, and through the structure. There are two failure modes provided by MIKE 11; breach (overtopping) failure and piping failure. The breach development through the dam break structure can be described using either of the two methods: NWS DAMBRK or energy equation.

3.1.1.5.1 The NWS DAMBRK method

This method uses a weir type equation to determine the flow through the breach failure and an orifice type equation to determine the flow through the Piping failure.

Equation for Breach failure

$$Q = C_v K_s \left[C_{weir} b \sqrt{g(h-h_b)}(h-h_b) + C_{slope} s \sqrt{g(h-h_b)}(h-h_b)^2 \right] \qquad (3.10)$$

Where: C_v = Correction coefficient (cover up for energy loss to the inflow contraction)

K_s = Submergence correction coefficient

C_{weir} = Weir Coefficient for horizontal parts; (0.546430)

b = Breach bottom width

g = Gravitational acceleration

h = Water level upstream (m) (reservoir water level),

h_b = Breach bottom level

S = Breach slope

C_{slope} = weir coefficient for slope parts; (0.431856)

3.1.1.5.2 Equation for piping failure

Piping failure usually starts with a circular hole formed through the body of the dam that eventually results into a collapse of the dam. Through a piping failure, the discharge of flow can be calculated using given equation 3.11.

$$Q = C_{orifice} A \sqrt{2g(h - \max(h_p, h_{ds}))} \qquad (3.11)$$

Where: $C_{orifice}$ = Orifice coefficient (0.599769),

A = Flow area in pipe; b ($h_{pt} - h_b$) + S (h_{pt} - h_b)^2

h_{pt} = Top of pipe

h_b = Bottom of pipe

h_p = centerline of pipe; (h_{pt} +h_b)/2

h_{ds} = Downstream water level

The possibility is considered that the pipe collapse may be from the top of the pipe to the top of the dam crest or there may not be enough water upstream of the dam to maintain the pipe however, this condition is computed using equation 3.10.

$$h < \frac{3}{2}(h_{pt} - h_b) + h_b \tag{3.12}$$

During pipe failure, the orifice equation is used until the dam collapses after which the flow is now calculated using the breach equation.

3.1.1.5.3 Energy equation (Erosion based Breach Development)

This breach development method uses a theory similar to the broad crested weir but with some exceptions; the changes in the dam are time oriented i.e. with time, the dam crest decreases and the breach increases; flows over the crest are not the same as flows over the breach due to the height difference and so these flows are computed separately. Please denote figure 3-2.

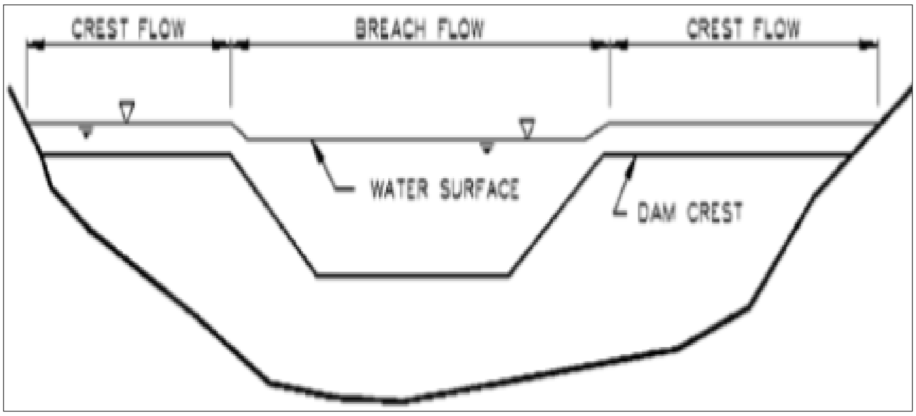


Figure 3-2 Combined flow over dam (DHI Water and Environment, 2009).

Breach development using the energy equation is overtopped through a trapezoidal breach or piping mode. If the failure mode is time dependent, the user specifies the initial breach shape, breach level, breach bottom width and breach side slope. With these input parameters, the breach is developed based on sediment transport time function. But if the mode of failure is erosion based, the user must input the initial and the final breach shape of the breach into the model. In this instance, the Engelund-Hansen's sediment transport formula is used to calculate the sediment transport in the breach.

$$\Phi = 0.1\frac{\theta^{5/2}}{f} \tag{3.13}$$

$$\Phi = \frac{q_t}{\sqrt{(s-1)gd^3}} \tag{3.14}$$

$$\Phi = 2\frac{uf^2}{u^2} \tag{3.15}$$

Where: Φ = Sediment transport rate (dimensionless)

θ = Total shear stress

q_t = Total bed material transported per unit width

f = friction factor

uf= fiction velocity

u = current velocity

The Engelund-Hansen equation calculates the sediment transport only in m^2/s per m width of pure sediment therefore there's a need to evaluate how the transportation of sediments have affect the level of the breach. This can be analyzed using equation 3.16.

$$\frac{dH_b}{dt} = \frac{qt}{L_b(1-\epsilon)} \tag{3.16}$$

Where: H_b= Breach level

q_t = Sediment transportation rate m^2/s

ϵ = Sediment porosity

L_b = Breach length in flow direction

t = Time

3.1.1.5.4 Erosion based piping failure

Similar to the NWS DAMBRK method, piping failure starts with flow through the body of the dam and due to the transport of sediments from the dam body that gradually enlarge the pipe until the dam collapses. In MIKE 11 it is assumed that the pipe through the dam is circular and always below the water level. Therefore; the pipe is always full, and the pipe center line is located in the final breach area of the dam. Illustration of the pipe failure development is shown in figures 3-3 and 3-4 respectively. At the collapse of the dam, the breach bottom elevation will be equal to the inverted portion of the pipe and materials settling on the breach bed will be computed using f_{lost}.

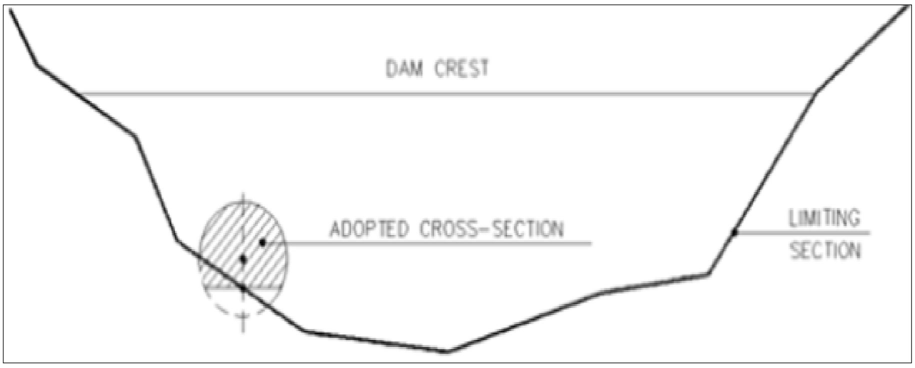


Figure 3-3 Piping failure cross section (DHI Water and Environment. 2009)

f_{lost} is used to evenly distribute a friction of the sediment that will not be washed away over the bottom of the breach. See figure 3-4.

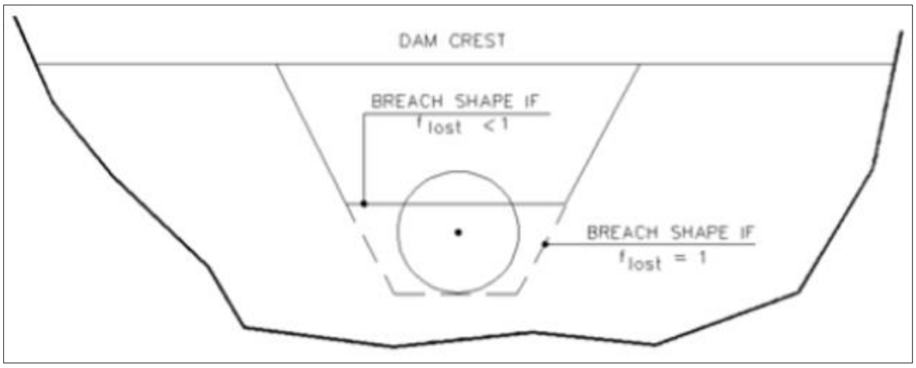


Figure 3-4 The collapse after piping failure (DHI Water and Environment, 2009).

The following equations below are used to calculate flow through the Pipe.

$$Q_p = A\sqrt{\frac{2g\Delta H}{1.5+\frac{fL}{4R}}} \tag{3.17}$$

$$\frac{1}{\sqrt{f}} = 2log_{10}(\frac{12R}{k_s}) \tag{3.18}$$

$$\Delta H = h_1 - \max(h_s, Z_{obv}) \tag{3.19}$$

Where: Q_p = Flow through the pipe

$\mathbf{A}$ = Pipe cross sectional area

g = Gravitational acceleration

f = Darcy's friction factor

R = Hydraulic radius

Z_{obv} = Overt of the pipe

h_1 = Upstream water level

Equation 3.18 is used to calculate the water depth for sediment transport. The more sediment passes through the pipe, the larger it gets, therefore, to calculate the change in pipe's radius equation 3.21 is used.

$$y_p = \frac{\Delta H}{2} + D \qquad (3.20)$$

$$\Delta R_p = C_{cal} \frac{qt}{2L_p(1-\epsilon)} \Delta t \qquad (3.21)$$

Chapter 4

Methodology

4.1 Data Collection

Data collection is paramount to any research, therefore, researchers are to be definite about the legality of data being collected; the credibility of any analysis depends on the accuracy of collected data. For the dam break flood inundation modeling of the Mount. Coffee Dam in MIKE 11, the below-listed data and processes are used to simulate the effects of breach parameter on the outflow hydrograph, velocity, water level, flood wave travel time at the dam and different downstream locations of the dam. Required data for MIKE 11model analysis are found in Table 4.1, 4.2, 4.3 and figure 4.1.

Table 4.1 Salient features of Mount Coffee Dam.

GENERAL DATA	
Catchment Area	19,992 Km^2
Maximum Annual Precipitation	3,800mm/year
Annual Mean Flow	1,768mm/year
Maximum Reservoir Water Level	29.56m
Maximum Reservoir Capacity	62.6 x $10^6 m^3$
Maximum Reservoir Area	8.1Km^2
Minimum Reservoir Operation Level	27.43m
Minimum Reservoir Operation Area	7.19Km^2
Minimum Reservoir Operation Capacity	54.3 x $10^6 m^3$
Probable Maximum Flood	21,184.2m^3/s
Dam Type	Earth filled
Dam Crest Level	31.09m
Dam Crest Width	5.5m
Dam Crest Length	466m
Spillway Type	Reinforced Concrete

Spillway Height	8.2m
Spillway Crest Length	121.2m
Spillway Crest Level	31.09m

Table 4.2 Probable Maximum Flood for Mount Coffee Dam.

TIME (hr.)	DISCHARGE (m^3/s)
0	0
10	485
20	5259
30	5250
40	19400
50	2700
57	27814
60	25000
70	19000
80	8000
90	5250
100	5260
110	3000
120	2000
130	1000
140	485

As established by Hatch investigation, the Mt. Coffee Probable Maximum Flood (PMF) duration is 5 days with a peak flood of 27184m^3/s. The peak of the flood is calculated to occur 57 hours after the beginning of the flood event. (Hatch, 2012). Inflow probable maximum flood hydrograph is shown in figure 4-1.

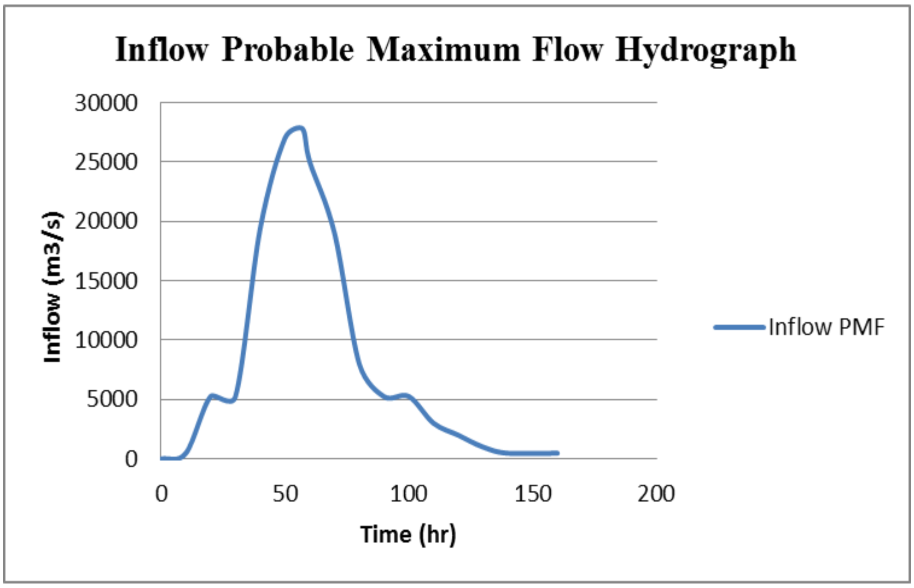


Figure 4-1 Inflow PMF Hydrograph for Mount Coffee Dam.

(Source: Hatch, 2012)

Table 4.3. Stage-Area Capacity Curve of Mount Coffee Reservoir.

STAGE	AREA (10^6 Km)	CAPACITY (10^6 m^3)
15.24	1.9	0.9
15.85	2.0	2.4
16.46	2.15	3.7
17.07	2.29	5.2
18.29	2.46	6.7
18.9	2.63	8.3
19.5	2.8	10
20	3.0	11.7

20.7	3.2	13.6
21	3.44	15.9
21.9	3.67	17.8
22.5	3.93	20.1
23	4.2	22.6
23.7	4.5	25.2
24	4.81	28.0
25	5.14	31.1
25.6	5.5	34.3
26	5.88	37.8
26.8	6.72	41.5
27	7.19	45.5
28	7.69	49.7
28.7	8.22	54.2

4.1.1 Network Editor

The Study area map is generated from MIKE Hydro's base map using the coordinate map projection for Mount Coffee, X= (-1220000, -1140000), and Y= (660000, 780000). The map is then exported to MIKE 11 for river alignment and chainage connections along the channel. This research model considers 32 Km of the St. Paul River, 28 km downstream of the Mount Coffee dam and 4Km upstream of the dam.

The network editor in MIKE 11 allows users to create and connect chainage points along the channel to form branches in flow directions. This editor allows the user to define data like, flow direction, the maximum distance between chainage points, structures (Dam break) locations, dam height, dam length, head loss factors, failure mode and moment, and breach calculation method. Based on FERC Guidelines chapter 2, it is assumed that the dam collapse at the peak reservoir level after the inflow of PMF into the reservoir. The Mount Coffee model network is similar to a typical network layout where the dam is used as an inland structure separating the network into two branches; the reservoir and the branch downstream see a typical layout in Figure: 4-2.

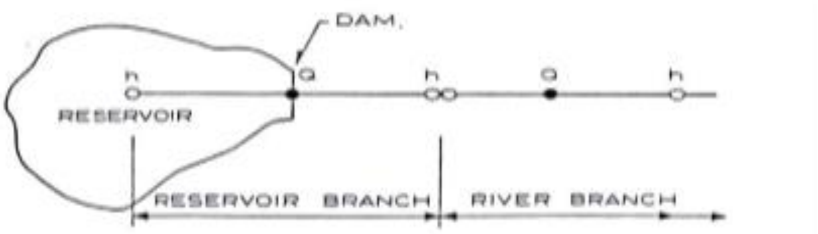


Figure 4-2 Typical Dam Breach Model Layout for Simulation.

4.1.2 The Dam

At the inland dam structure Q-point, the momentum equation is replaced by the broad crested weir flow equation describing the subcritical or critical flow through the structure (see equation 4.1). The Mount Coffee Dam is an earthen embankment of length 466m with a reinforced concrete spillway of length 121m and radial gates 10. The dam elevation is at 31.09m asl, River bed at 15.2m asl, sill level of the spillway at 18.3m asl and spillway height 8.2m. Calculation method and mode of failure used in this study for flow through the dam is NWS DAMBRK and overtopping respectively. Details of methods and modes are discussed in chapter 3.

Eqn. 4.1. Broad Crested Weir Formula

$$Q = C_v K_s \left[C_{weir} b \sqrt{g(h-h_b)}(\mathrm{h}-h_b) + C_{slope} s \sqrt{g(h-h_b)}(\mathrm{h}-h_b)^2 \right]$$

4.1.3 Cross Section Editor:

All cross sections in the model are defined using MIKE Hydro, a packaged product of MIKE by DHI. By using the tasks bar in MIKE HYDRO, cross sections are generated from the base map with corresponding topography. This information is transferred to

MIKE 11 and stage-area curve is inserted into the reservoir. Figures 4-3 and 4-4 show cross sections from the reservoir and the downstream area respectively.

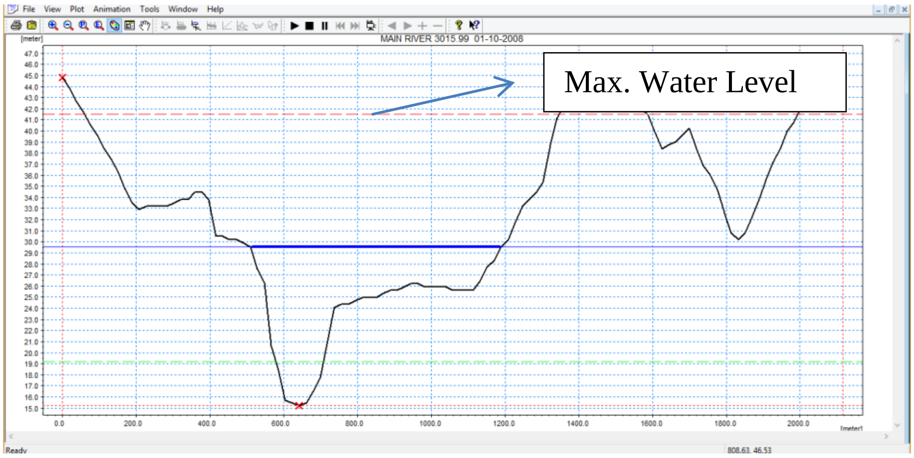


Figure 4-3 Cross Section at Reservoir.

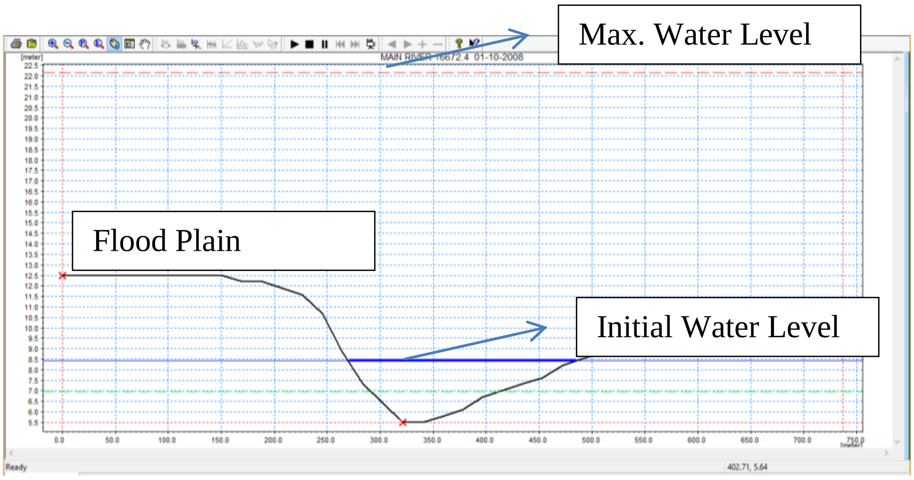


Figure 4-4 Cross Section at Foffee Town (12Km) Downstream.

4.1.4 The Reservoir

The reservoir in the network file is represented by the first chainage h-point (water level) and the h-point after the dam structure that connects with the downstream branch. These h-points are water level indicator as specified in the 6-point Abbott Ionescu scheme (See chapter 3 for details). In the model, the first h-point is accurately placed at the beginning of the reservoir; therefore; it serves as the upstream boundary of the model where the inflow probable maximum flood and the storage area capacity curve are inputted for inflow and water surface level.

4.1.5 Boundary Editor:

The inflow hydrograph enters the model at the upstream end of the model as an external boundary condition and the structure (i.e. the main dam) is inserted as an inland structure between the upstream and the downstream where breach parameters (width, level, and slope) are internal boundary conditions. The initial condition at the reservoir is that the reservoir is at its maximum water level (29.56m) prior to the inflow PMF while at the downstream end of the model; an automatic Q-h relationship is generated by the software through digitized chainage points along the river network. Mount Coffee channel and floodplain roughness coefficients were established by Hatch, 2012 “Dam Safety Report” by using ortho imagery, site photos, and Open-Channel Hydraulics (Chow, 1959). The report states that the channel is clean, winding with some pools and shoal and the flood plain, medium dense with vegetated brush. So they specify the channel and floodplain as 0.04 and 0.09 respectively.

4.2 Estimating Dam Breach Parameters

Estimating breach parameter of a dam is crucial since the outflow hydrograph depends on these parameters to determine its impacts (timing and sizing) to the downstream inundation area. Prediction equations obtained by other researchers as a result of regression analysis of historical dams can be used to predict breach parameters considering the hydraulic and geometrical characteristics of the dam to be investigated. Two sets of equations are compared in this paper in order to determine the worst-case scenario of overtopping the Mount Coffee Dam. These sets of equations are FERC and Froehlich, 2008. They are based on historical dam break data and are widely used and accepted in dam break investigations by individuals as well as agencies. The sets of equations formulated by these researchers are shown in Table 4.3.

FERC (Federal Energy Regulatory Commission Guidelines) is prepared by a United States Federal Agency. This guideline is widely used and accepted by National Weather Service (NWS) guidelines and UK dam break guidelines. The FERC guideline also deals with many other hydraulic issues such as; Breach Prediction Parameters, Inflow Design, Probable Maximum Flood, Freeboard Allowances etc. See Breach Prediction table in Chapter 2, Table 2.5.

Froehlich (2008) developed a model estimating the average breach width (B_{avr}), average slope (z) and the breach formation time (t_f), with the use of linear regression analysis making use of 74 historic embankment dam failure data. His equations were formulated with various dam and reservoir parameters including, reservoir water elevation (V_w), critical overtopping depth (H_c), and height of breach (H_b).

In Table 4.4. the two sets of breach parameter equations are outline; Table 4.5 shows the properties of the Mount Coffee Dam and Table 4.6 shows the results from both sets of equations using Mount Coffee Dam properties.

Table 4.4 Breach prediction parameter equations.

Breach Parameter	**FERC**	**Froehlich (2008)**
Average Breach width B_{avr} (m)	$H_d \leq B_{avr} \geq 5H_d$	$B_{avr} = 0.27K_o V_w^{0.32} H_b^{0.04}$ Ko = 1.3 for overtopping
Breach formation time t_f (hr.)	$0.1 \leq t_f \geq 1.0''$	$t_f = 63.2\sqrt{\frac{V_w}{gH_b^2}}$
Breach side slope Z:H	$\frac{1}{4} \leq t_f \leq 1$	z = 1:1 for overtopping

Table 4.5 Mount Coffee Dam Properties.

Properties	Maximum Reservoir Level
V_w, (m^3)	62.6 x 10^6
H_b (m)	15.2
H_w (m)	29.56
H_d (m)	15.2

Table 4.6 Predicted Breach Values for Mount Coffee Dam.

Breach Parameter	FERC	Froehlich (2008)
B_{avr} (m)	46	122
t_f (hr)	0.5	2.9
Z:H	0.5:1	1:1

Where: B_{avr} = average breach width (m)

t_f = breach formation time (hr)

K_o = Constant

z = side slope of breach

H_d = dam height (m)

H_b = breach height (m)

V_w = reservoir volume (m^3)

H_w =height of water

g = acceleration due to gravity (m/s^2)

4.3 Establishing Inflow Design Flood

To establish the appropriate Inflow Design Flood (IDF) the incremental hazard evaluation effects through the dam and the downstream must be identified above which the consequences of failure become acceptable. According to FERC guidelines the threshold of incremental increase at an inhabited area is 0.6m or more. Flood flow condition above which additional incremental increase in elevation due to a dam failure is no longer considered to present an unacceptable threat to the downstream life and property can be stated as the IDF. Since probable maximum flood (PMF) is the upper limit of IDF, smaller flood events are analyzed for hazard classification in two cases: case I: Routing the flood through a normal reservoir level with no-break condition and case II: Routing the same flood through a dam break condition.

Mount Coffee full PMF which is 27184.2m^3/s is broken down into smaller fractions for the purpose of establishing the flood hazard condition above which the breach is no longer unacceptable or hazardous to life and property. To calculate the incremental increment of the flood flow downstream, the difference between case I and case II divided by case I. Fraction of the Mount Coffee PMF is specified in Table 4.7. Base on the guideline set by FERC on inflow design floods, the result of incremental increase is shown in Table 4.8 and Figure 4.5. This incremental increment has satisfied that fraction 0.3PMF - 8155.3m^3/s is suitable as inflow design flood since it poses acceptable flood hazard to the downstream inhabitants. The recent spillway capacity at the Mount Coffee Dam is 9910m^3/s which is inadequate to any flood more than 0.3PMF.

Table 4.7 smaller flood events

Flood Events	Discharge(m^3/s)
0.3PMF	8,155.3
0.4PMF	10,873.7
0.7PMF	19,028.9
0.8PMF	21,747.4
Full PMF	27,184.2

Table 4.8. Max incremental to the downstream

Distance D/S of Dam (Km)	**0.3PMF**	**0.4PMF**	**0.7PMF**	**0.8PMF**	**FULL PMF**
0.5	0.68651	0.56826	0.40073	0.37285	0.17317
1	0.68908	0.5709	0.41923	0.39276	0.17825
2	0.60995	0.50103	0.36795	0.3439	0.15823
3	0.60547	0.48988	0.35611	0.33333	0.14979
5	0.53683	0.43605	0.3426	0.32531	0.14083

6	0.51647	0.42161	0.32991	0.31301	0.13383
7	0.4506	0.37451	0.29837	0.28406	0.12464
8	0.469	0.38548	0.31072	0.29674	0.13047
9	0.42343	0.35089	0.28505	0.27394	0.1256
10	0.4244	0.35019	0.28639	0.27451	0.12413
11	0.41486	0.34248	0.28123	0.27041	0.12267
12	0.40501	0.33141	0.27778	0.26846	0.11301
14	0.40532	0.32615	0.27846	0.24854	0.1163
15	0.42218	0.32845	0.27746	0.23744	0.11467
16	0.44388	0.34465	0.28997	0.22809	0.11128
17	0.4436	0.33168	0.27938	0.21818	0.10848
18	0.42134	0.33031	0.28391	0.2039	0.09771
18.5	0.41957	0.32242	0.27498	0.19415	0.10181
20	0.44989	0.34089	0.28929	0.19514	0.10434
21	0.40871	0.34255	0.2949	0.1958	0.10951
22	0.40111	0.32915	0.25536	0.18672	0.1016
24	0.39603	0.31	0.23921	0.18911	0.09925
25	0.36621	0.29874	0.22464	0.18508	0.09454
26	0.36095	0.28397	0.21521	0.16934	0.08916
27	0.31418	0.26113	0.20435	0.12129	0.09734
28	0.24348	0.21027	0.17231	0.09065	0.08174

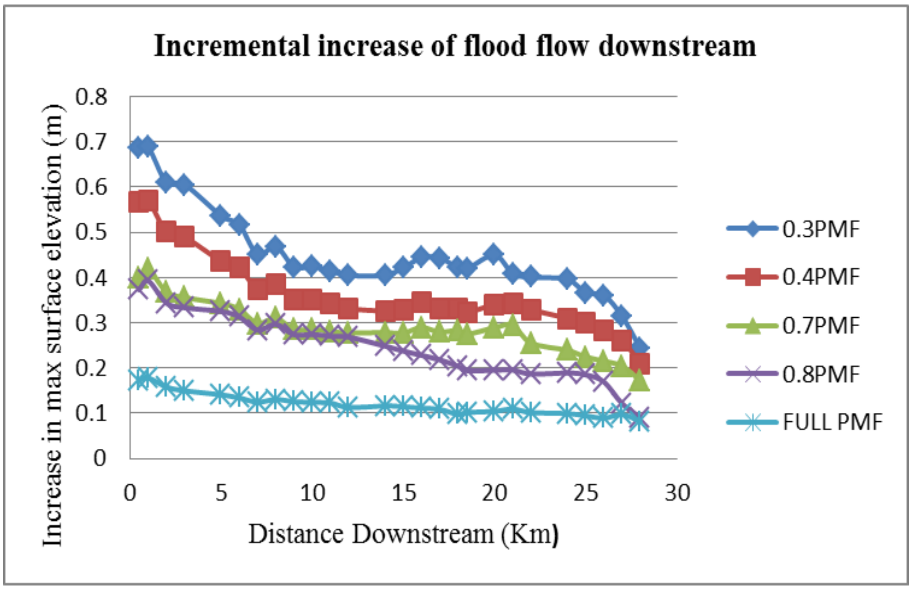


Figure 4-5 Flood flow incremental increase downstream of dam.

Chapter 5

RESULTS AND DISCUSSION

So far, selection and comparison of breach parameters established by FERC and Froehlich have been studied and selected based on their efficiencies for earthen dams. The two methods are widely used in dam break analysis by both individuals and agencies. (See details in chapter 4). In this study, these methods of breach prediction are compared to predict the most critical or worst case scenario (outflow hydrograph and flood routing of flood wave downstream) of dam breaching at the Mt. Coffee.

To reach the most critical or worst-case failure scenario, it is assumed that dam failure during normal operation would result to low risk to the flood plains in that water would be restricted to the river channel meanwhile worst case can be expected if dam is to fail during a flood condition (FERC, Chap. 2). Based on FERC's reference, this paper's evaluation considered the flood flow condition or the overtopping dam failure mode with the reservoir at maximum water level (29.56m) and inflow of the probable maximum flood (27184m3/s) into the reservoir. It is also assumed that the dam collapsed at the peak reservoir level. (Chapter 1, FERC Guidelines).

After running the two methods of breach prediction parameters in MIKE 11, the results are seen below. Table 5.1 shows the output breach statistics from the FERC breach parameter at the dam and Table 5.2 shows the output breach statistics of Froehlich's breach parameter at the dam.

5.1 FERC Dam Breach Result:

Inflow into the reservoir until the peak reservoir level 41.5m, it takes 58.267 hours. It is assumed in the model set-up that the breach of the dam begins to occur at the time of the reservoir peak level, therefore failure of the dam starts at 58.267 hrs. The first outflow through the breach 539.9m3/s starts at a level of 30.031m; approximately 1 m less than the dam crest level (31.09 m). From the start of the breach to the bed level (15.2 m) and maximum discharge (12936.1m3/s) it takes 28 minutes. After the breaching of the dam, Flood water last up to 61.7 hours (2.57days) to drain through the breach. The breach velocity gradually decreased from 10m/s at the beginning of the breach to 8.785 at the maximum discharge and then to 3.376 m/s at base flow over the period of 2.57 days ending with a breach bottom width and a crest width of 45.72 m and 61.16 m respectively. Statistical details of breach outputs are shown in Table 5.1.

Table 5.1 FERC Dam Breach Statistics.

Time (hr)	Q in Breach (m)	V in Breach (m)	Reserv. Water Level (m)	Level of Breach (m)	Depth in Breach (m)	Breach Bottom Width (m)	Breach Width@ Crest (m)
58.26	539.9	10.84	41.509	30.031	11.479	3.328	4.387
58.43	3009.5	7.609	41.475	24.734	16.747	18.468	24.824
58.56	6486	8.059	41.392	20.497	20.91	30.58	41.173
58.73	12936.1	8.785	41.168	15.2	25.997	45.72	61.61
58.9	12752.3	8.749	40.933	15.2	25.753	45.72	61.61
59.06	12590.5	8.707	40.747	15.2	25.565	45.72	61.61
59.23	12443.3	8.669	40.577	15.2	25.393	45.72	61.61
59.43	12285.3	8.628	40.393	15.2	25.208	45.72	61.61
59.7	12103.2	8.58	40.18	15.2	24.992	45.72	61.61
60	11930.5	8.534	39.976	15.2	24.786	45.72	61.61
60.3	11785.3	8.495	39.803	15.2	24.612	45.72	61.61
60.6	11662.7	8.462	39.656	15.2	24.464	45.72	61.61
61	11527.2	8.426	39.493	15.2	24.3	45.72	61.61
65.6	10791	8.224	38.589	15.2	23.392	45.72	61.61
69	10224.4	8.063	37.872	15.2	22.676	45.72	61.61
76.3	8575.1	7.576	35.658	15.2	20.463	45.72	61.61
83.3	7474.5	7.233	34.06	15.2	18.864	45.72	61.61
87.6	6787.5	7.01	33.001	15.2	17.806	45.72	61.61
91.3	6135.6	6.792	31.947	15.2	16.752	45.72	61.61
92.8	5841.8	6.691	31.454	15.2	16.26	45.72	61.61
94.5	5460.4	6.559	30.794	15.2	15.602	45.72	61.61
100.2	3762.9	5.872	27.579	15.2	12.389	45.72	61.61
105.5	2331.4	5.077	24.362	15.2	9.172	45.72	61.61
109.4	1539.5	4.465	22.237	15.2	7.045	45.72	61.61
113.6	992.2	3.889	20.51	15.2	5.316	45.72	61.61
115.3	849.2	3.702	20.004	15.2	4.809	45.72	61.61
119.96	635.7	3.376	19.186	15.2	3.988	45.72	61.16

5.2 Froehlich Dam Breach Result:

From the start of the Froehlich dam breach, all statistics of the breach (time, velocity, Reservoir water level, breach level, depth in breach, breach bottom width, and breach width at the crest), except initial discharge through the breach are similar to the FERC dam breach statistics however, the time taken for the breach to reach bed level of the channel is much longer than that of the FERC's. It takes 7hours and 20minutes from the start of the breach to reach the bed of the channel and the maximum discharge (26668.1m3/s). The flood through the breach lasts for 2.57day as the FERC method predicted. The breach velocity dramatically decreased from 28.165m/s at the beginning of the breach to 8.411 at the maximum discharge and then gradually to 2.276 m/s at base flow over the period of 2.57 days ending with a breach bottom width and a crest width of 122 m and 153.78 m respectively.

Table 5.2 Froehlich Dam Breach Statistics.

Time (hr)	Q in Breach (m)	V in Breach (m)	Reserv. Water Level (m)	Level of Breach (m)	Depth in Breach (m)	Breach Bottom Width (m)	Breach Width@ Crest (m)
58.26	615.4	28.165	41.507	30.907	10.601	1.699	2.064
58.43	1224.5	9.88	41.489	29.994	11.496	8.693	10.885
58.56	1802.9	8.354	41.476	29.264	12.214	14.289	17.941
58.73	2642.9	7.702	41.451	28.35	13.103	21.283	26.762
58.9	3616.7	7.47	41.415	27.437	13.982	28.277	35.583
59.06	4726.7	7.403	41.366	26.524	14.848	35.271	44.403
59.23	5973.6	7.416	41.302	25.611	15.699	42.266	53.224
59.43	7649.9	7.484	41.201	24.515	16.696	50.659	63.809
59.7	10184	7.621	41.019	23.054	17.979	61.849	77.922
60	13424	7.8	40.739	21.41	19.347	74.439	93.799
60.3	17041	7.987	40.366	19.766	20.623	87.029	109.67
60.6	20990	8.173	39.888	18.122	21.795	99.618	125.55
61	26668	8.411	39.067	15.931	23.175	116.40	146.72
65.6	21436	7.658	35.081	15.2	19.886	122	153.78
69	19769	7.446	34.144	15.2	18.949	122	153.78
76.3	15514	6.873	31.556	15.2	16.362	122	153.78
83.3	11685	6.303	28.898	15.2	13.706	122	153.78
87.6	9028.1	5.822	26.839	15.2	11.647	122	153.78
91.3	6652.4	5.293	24.784	15.2	9.594	122	153.78
92.8	5701.8	5.042	23.884	15.2	8.695	122	153.78
94.5	4592.6	4.708	22.759	15.2	7.571	122	153.78
100.2	1846.7	3.517	19.397	15.2	4.202	122	153.78
105.5	1013.5	2.893	18.043	15.2	2.847	122	153.78
109.4	703.1	2.568	17.443	15.2	2.244	122	153.78
113.6	534.5	2.303	17.008	15.2	1.808	122	153.78
115.3	504	2.276	16.965	15.2	1.765	122	153.78
119.96	485.8	2.276	16.965	15.2	1.765	122	153.78

In table 5.3, The dam breach statistics illustrates that the Froehlich method of breach produces more formation time, peak discharge, initial velocity, breach bottom width and breach crest width than the FERC method of breach prediction. The percentage difference is given in Table 5.3 of the two methods of breach prediction parameters.

Table 5.3 Comparing Dam Breach Statistics of FERC to Froehlich.

Parameter	FERC	Froehlich	% of difference
Formation time (hr)	0.733	2.733	115
Peak Discharge (m3/s)	12936.1	26668.1	69
Initial velocity (m/s)	10.84	28.165	88
Peak velocity (m/s)	8.785	8.411	4
Breach bottom width (m)	45.72	122	90
Breach crest width (m)	61.16	153.78	88.6

Tables 5.1 and 5.2 statistics are limited to passage of flood through the breach. However, Table 5.4 and figure 5-1, show statistics of outflow both through the breach and overtopping the dam. The formation time and the outflow hydrograph resulting from the FERC method are lesser in peak than the Froehlich's methods.

Table 5.4 Simulated outflow using FERC and Froehlich methods of breach parameters.

TIME(hr.)	FERC OUTFLOW (m^3/s)	FROEHLICH OUTFLOW (m^3/s)
0	0	0
14.61	0.43	0.94
24	4773.87	4781.20
48	21523.02	21532.70
58.73	35079.91	27712.72
61.13	26770.11	39879.71

72	19314.45	19522.72
96	5038.35	3596.36
120	634.84	485.74

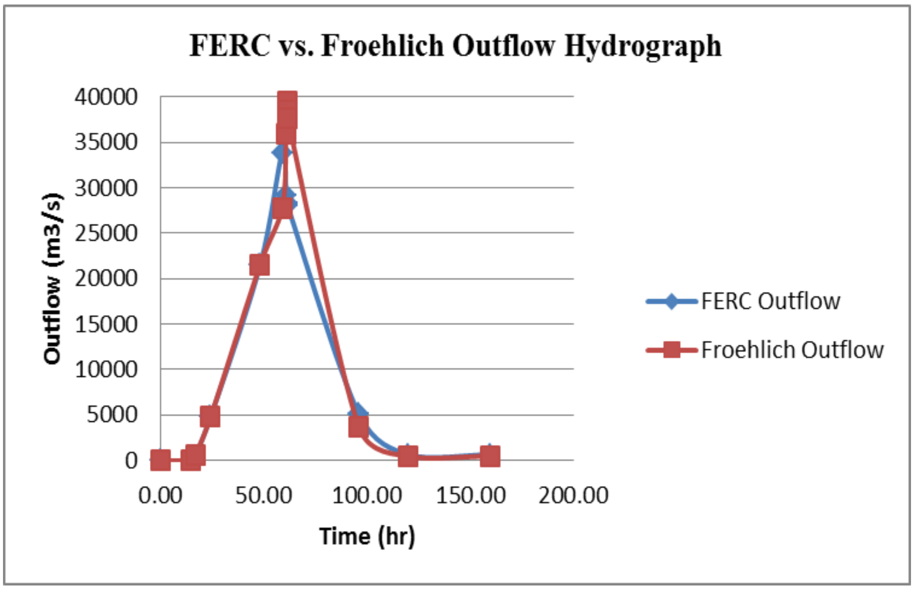


Figure 5-1 Simulated outflows at Dam, FERC and Froehlich Prediction parameters.

5.3 Routing of Total simulated outflow hydrograph downstream using FERC and Froehlich results

Six downstream locations are sited in this analysis; however, three of these chainage locations are emphasized due to the inhabitants that might be at risk in these areas. As flood wave is routed downstream, the following characteristics, discharge, velocity, water level, arrival time, their attenuations are compared at 1Km, 7Km, 12Km, 18Km, 23Km, and 28Km. The peak outflow hydrographs simulated of FERC and Froehlich breach parameter are 35079.91m3/s at 58.73hrs and 39879.71 at 61.13hrs respectively after the inflow of the PMF into the reservoir at the peak level of 41.168m. Overtopping of the dam begin 14.62 hours from the time of inflow of the PMF into the reservoir.

5.3.1 FERC Outflow Hydrograph Routing

From the primary time of overtopping the dam, 14.62 hr, the first flood wave reached 1Km at 14.68 hr, 7Km at 16.86 hr, 12Km at 17.76hr, 18Km at 18.71hr, 23Km at 19.61hr and 28Km at 21.21hr. Also from the time of peak discharge at the dam to peak discharge downstream of the dam, 1Km, 7Km, 12Km, 18Km, and 28Km, it takes 6mins, 27mins, 43mins, 66mins, 1hr 45mins, and 1hr 55mins respectively. The water levels from 1Km to 28Km downstream ranged from 28.49m to 921m and velocity from 3.93m/s to2.97m/s. These parameters are shown is Table 5.5 and figure 5-3.

5.3.2 Froehlich Outflow Hydrograph Routing

From the time the flood starts to overtop the dam at 14.62 hr, the first flood wave reached the downstream distance of 1Km at 14.68 hr, 7Km at 16.5 hr, 12Km at 17.3hr, 18Km at 18.8hr, 23Km at 19.37hr and 28Km at 20.21hr. Also from the time of peak discharge at the dam to various peak discharge downstream of the dam, the travel time for 1Km, 7Km, 12Km, 18Km, and 28Km, are as follows 1.2mins, 10.2mins, 19.2mins, 31.2mins, 56mins, and 1hr respectively. The water levels from 1Km to 28Km downstream ranges from 27.58m to 8.64m and velocity from 4.94m/s to 3.81m/s. Further details are seen in Table 5.5 and Figures 5-2, 5-3 and5-4.

In this study, the FERC and the Froehlich prediction results obtained is evaluated for the method that would produce the worst case or critical flood wave condition at inhabited locations downstream of the dam breach. There are three specific areas highlighted since it has been observed that the population has been increasing over the last two years; they include Fofee Town- approximately 12Km downstream, the Township of Caldwell- 18Km downstream and OAU Village/New Kru Town-28Km downstream of the Mt. Coffee Dam. Effects of flood wave caused by a dam break at these locations are tantamount to creating a hazard to human lives and properties. Comparison of the two prediction methods discharge, velocity and water level of the flood wave is shown in Table 5.5 and figures 5-2, 5-3 and 5-4.

Table 5.5 Comparison of FERC vs. Froehlich Flood Wave Discharge, Travel time and Water level of the Mount Coffee Dam breach.

Distance (Km)	FERC Peak Q (m^3/s)	Froeh-lich Peak Q (m^3/s)	FERC Flood Peak Travel Time from Dam Breach (hr)	Froeh-lich Flood Peak Travel Time From Dam Breach (hr)	FERC Peak H_2O Level (m)	Froeh-lich Peak H_2O Level (m)	FERC Flood Vel. (m/s)	Froehlich Flood Vel. (m/s)
1	33842.24	39401.03	0.1	0.02	28.49	27.58	3.93	4.94
7	32605.55	38523.66	0.45	0.17	23.48	22.6	2.82	3.59
12	31667.11	37534.67	0.72	0.32	22.17	21.26	3.45	4.38
18	30822.7	36467.48	1.6	0.52	19.64	18.89	2.45	3.09
23	30357.74	35614.23	1.75	0.93	17.83	17.13	2.82	3.5
28	30342.21	35580.58	1.92	1.0	9.21	8.64	2.97	3.81

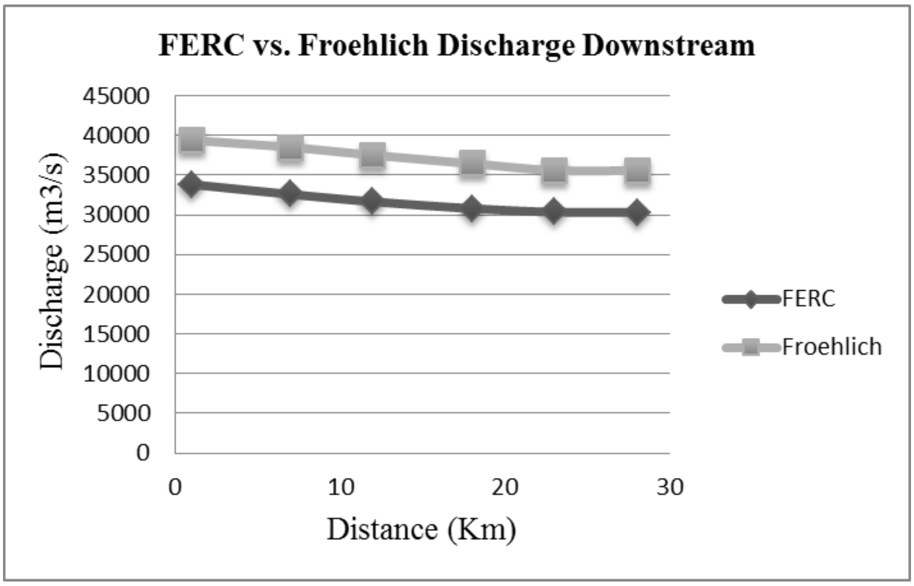


Figure 5-2 Comparison of FERC and Froehlich discharge.

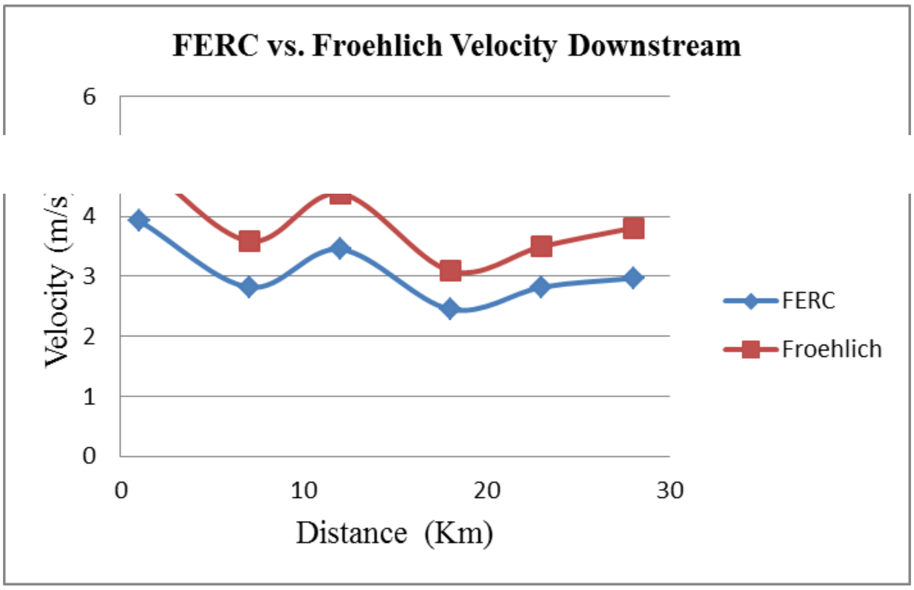


Figure 5-3 Comparison of FERC and Froehlich velocity.

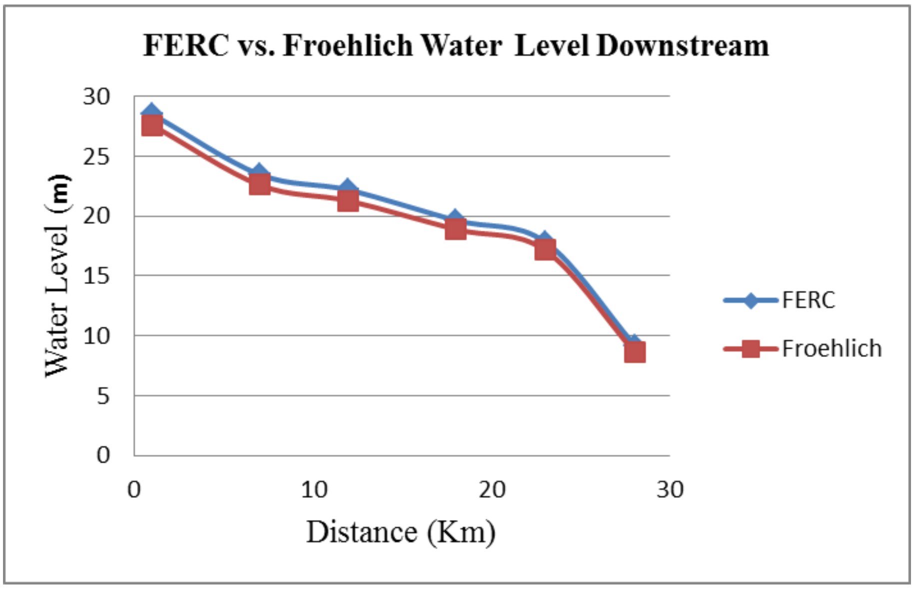
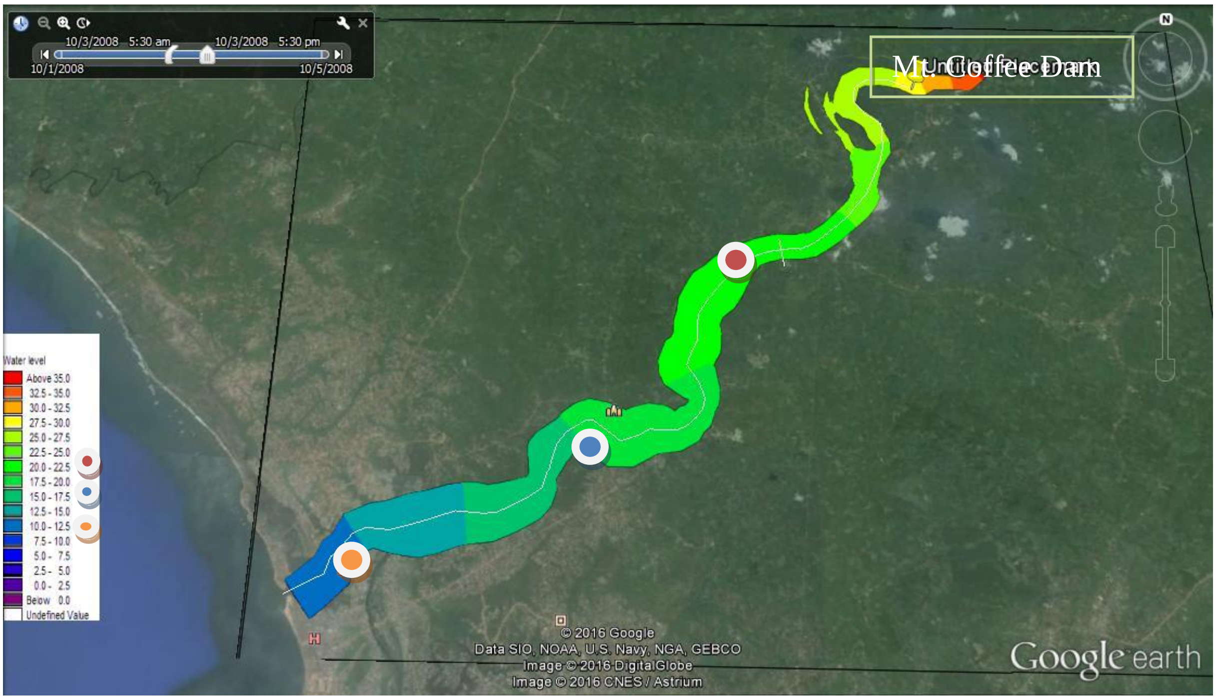


Figure 5-4 Comparison of FERC and Froehlich Water level.

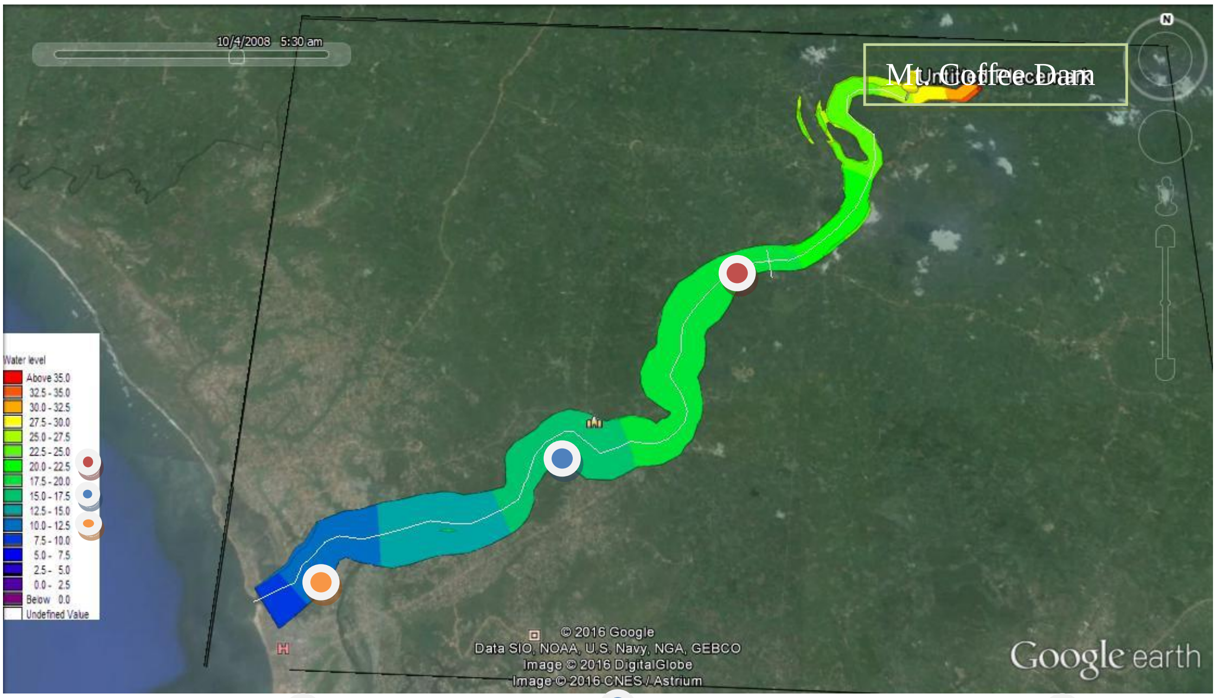
5.4 Flood Inundation Map

Figures 5-5, 5-6 and 5-7 show the effect of flood wave on the downstream of the dam. As mentioned in earlier in this chapter, the PMF duration is five days therefore below flood inundation maps show the variation in water levels over the period of time from the start of the peak outflow on the 3 days to the fifth day. Figure 5-5 shows the flood effects downstream on the 3rd day 4hr 22minutes after the collapse of the dam. Figure 5-6 shows the water level along the downstream zone on the 4th day at time 5:30 a.m. and figure 5-7 shows the flood effect on the 5th day at 5:30 p.m. Observing the three flood inundation maps generated over a period of 12hrs each, Areas like Fofee Town-12Km downstream, Caldwell-18Km downstream, and New Kru Town-28Km downstream have been highlighted and water levels compared in succession of the 36hrs period.



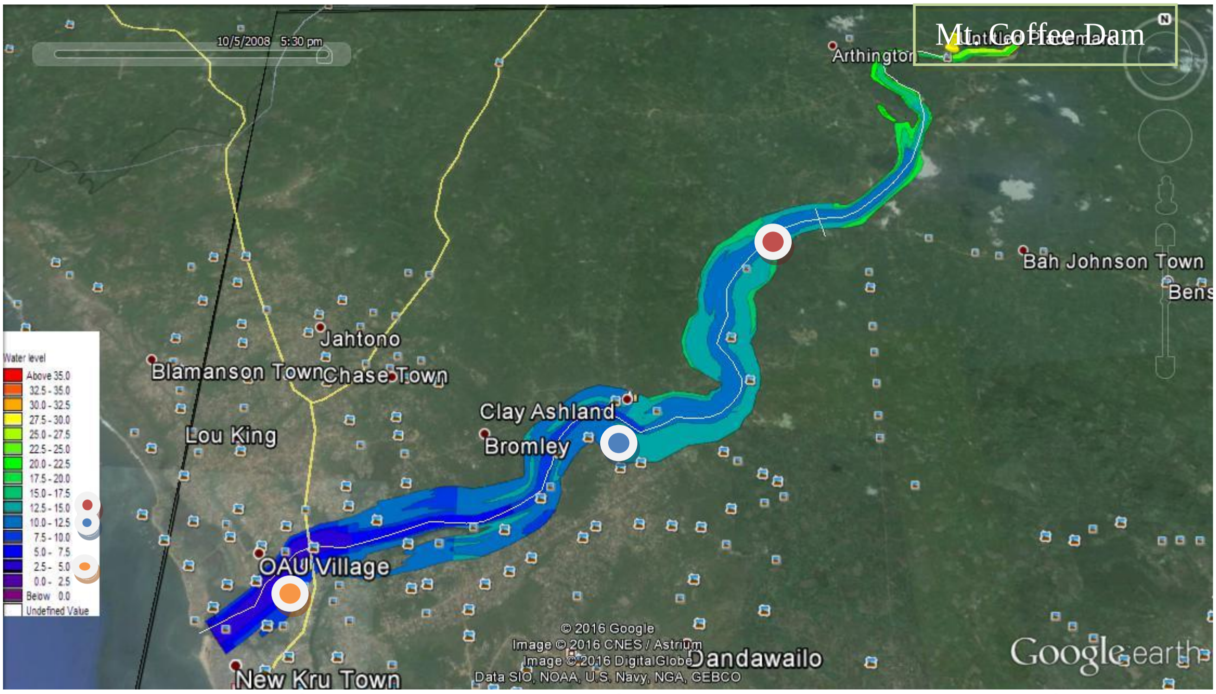
Fofee Town-12Km **Caldwell – 18Km** **New Kru Town- 28Km**

Figure 5-5 Flood inundation map on day 3 @ 5:30 pm (i.e. 4hr 22 min. after breach)



Fofee Town-12Km **Caldwell – 18Km** **New Kru Town- 28Km**

Figure 5-6 Flood inundation map on day 4 at 5:30am (i.e. 16hr 22 min. after breach)



Fofee Town-12Km **Caldwell – 18Km** **New Kru Town- 28Km**

Figure 5-7 Flood inundation map on day 5 at 5:30pm (i.e. 28hr 22 min. after breach)

5.5 Discussion

Based on Simulated results in MIKE 11, it is observed that the breach produced by the FERC breach prediction parameters, is smaller in shape than that of the breach produced by the Froehlich breach prediction parameters. In the same light, peak outflow hydrograph of Froehlich's is 13.6% more than FERC's. As a result of this outflow at the dam, routing the flood wave through the downstream has proven higher peak discharge at every downstream location along the channel. The Froehlich's acquired flood wave being routed through the channel moves faster in time and velocity after the breach of the dam, which is alarming for inhabited areas downstream of the dam. Even though FERC's outflow, velocity and travel time of flood wave are not as compared to Froehlich's, the water level is higher at all downstream locations due to the decrease in velocity of flood wave.

Based on the above analogies, results achieved from the Froehlich breach prediction parameter method is considered for further investigation for flood inundation mapping and sensitivity analysis of the hypothetical breach at Mt. Coffee Dam.

Chapter 6

Sensitivity Analysis

The sensitivity Analysis of a dam break evaluates changes in breach parameters and effects of these changes on the flood wave as they propagate through the model. Since the Froehlich breach parameter produced a more critical outflow hydrograph and flood routing conditions for the Mount Coffee Dam Break, the sensitivity analysis is evaluated using its simulated results. In six different tests, breach parameters are interchanged with respect to percentages. The dam breach parameters (breach formation time, breach width, and breach slope) are increased and decreased in percentage setups for effect evaluation: 20%Setup, 40%Setup, 60%Setup and 80%Setup. In these setups, a parameter is increased or decreased according to its header percentage while the other parameters in that setup remain constant as in the original setup as seen in Table 4.6. All six tests along with their percentage setups arrangements are illustrated in Table 6.1.

Table 6.1 Sensitivity Analysis Tests Setup.

Test I. [Increase in Breach Formation Time]				
Parameter	**20% Setup**	**40% Setup**	**60% Setup**	**80% Setup**
$B_{avr\ (m)}$	122	122	122	122
$t_{f\ (hr)}$	3.48	4.06	4.64	5.22
Z:H	1:1	1:1	1:1	1:1
Test II. [Decrease in Breach Formation Time]				
Parameter	**20% Setup**	**40% Setup**	**60% Setup**	**80% Setup**
$B_{avr\ (m)}$	122	122	122	122
$t_{f\ (hr)}$	2.32	1.74	1.16	0.58
Z:H	1:1	1:1	1:1	1:1
Test III. [Increase in Breach Width]				

Parameter	**20% Setup**	**40% Setup**	**60% Setup**	**80% Setup**
$B_{avr\ (m)}$	146.4	170.8	195.2	219.6
$t_{f\ (hr)}$	2.9	2.9	2.9	2.9
Z:H	1:1	1:1	1:1	1:1
Test IV. [Decrease in Breach Width]				
Parameter	**20% Setup**	**40% Setup**	**60% Setup**	**80% Setup**
$B_{avr\ (m)}$	97.6	73.2	48.8	24.4
$t_{f\ (hr)}$	2.9	2.9	2.9	2.9
Z:H	1:1	1:1	1:1	1:1
Test V. [Increase in Breach Slope]				
Parameter	**20% Setup**	**40% Setup**	**60% Setup**	**80% Setup**
$B_{avr\ (m)}$	122	122	122	122
$t_{f\ (hr)}$	2.9	2.9	2.9	2.9
Z:H	1.2:1	1.4:1	1.6:1	1.8:1
Test VI. [Decrease in Breach Slope]				
Parameter	**20% Setup**	**40% Setup**	**60% Setup**	**80% Setup**
$B_{avr\ (m)}$	122	122	122	122
$t_{f\ (hr)}$	2.9	2.9	2.9	2.9
Z:H	0.8:1	0.6:1	0.4:1	0.2:1

6.1 Test I: Increase in breach formation time

In this setup, the breach formation time is interchanged with an increase by 20, 40, 60 and 80 percent in the model with all other breach parameters remaining constant. The outcome of test I with regards to discharge, velocity and water level and travel time of flood wave at selected locations downstream of the dam are seen in tables and figures below.

Discharge: With the Increase in breach formation, the peak outflow of percentage setups are: 20%setup-38543.51 m^3/s, 40%setup-37344.63 m^3/s, 60%setup-36373.47 m^3/s and 80%setup-35452.49 m^3/s. Discharge at all locations downstream experience decrease in discharge ranging between 1.3 to 10% when compared to the original setup of Froehlich breach prediction parameter; see table 6.2.and figure 6.1.

Table 6.2 Discharge at Downstream Locations for Test I.

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1Km	38190.06	37118.54	35989.48	35276.85	39401.03
7Km	37496.33	36554.89	35701.99	34910.94	38523.66
12Km	36710.59	35926.8	35194.78	34500.18	37534.67
18Km	35848.07	35228.51	34625.95	34035.04	36467.48
23Km	35116.26	34605.38	34094.48	33583.05	35614.23
28Km	35084	34576.01	34067.23	33556.8	35580.58

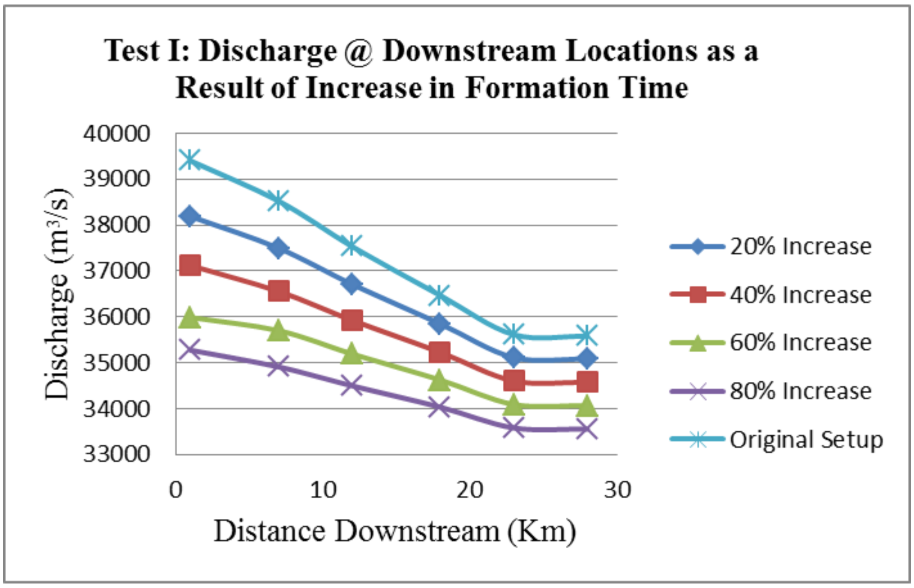


Figure 6-1 Effects of increase in formation time on discharge.

Velocity: A Change in formation time produces a decrease at most of the locations except at location 18Km; there is an increment in three percentage setups at 18Km: 40%, 60% and 80%setups these increment in velocity ranges from 20-22%. And the decrease in other locations ranges from 0.5 to 5.7% in comparison to the original setup of breach parameters. See results in Table 6.3. and figure 6.2.

Table 6.3 Velocity at Downstream Locations for Test I.

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1Km	4.87	4.81	4.76	4.71	4.94
7Km	3.52	3.46	3.42	3.38	3.59
12Km	4.31	4.24	4.19	4.13	4.38
18Km	3.05	3.8	3.76	3.72	3.09
23Km	3.48	3.46	3.43	3.41	3.5
28Km	3.79	3.77	3.76	3.74	3.81

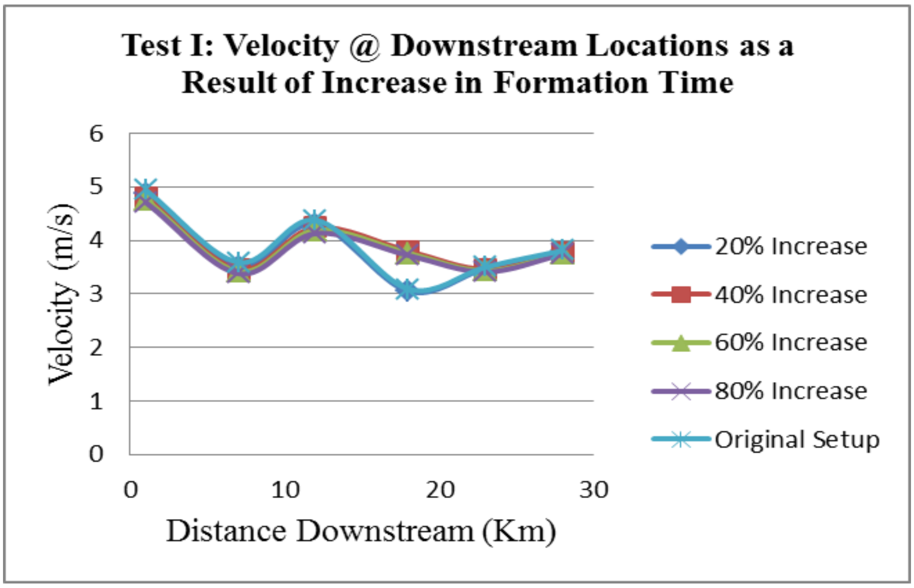


Figure 6-2 Effects of increase in formation time on velocity.

Travel Time: The peak outflow which begins at 61.07hrs after the inflow into the reservoir takes the following time to travel to the end of the model: 20%setup -1.2hrs, 40%setup -1hr, 60%setup - 0.92hr and 80%setup - 0.82hr as compared to the original setup where the travel time from the dam to the end of the model takes 1.0hr. Travel time increases in 20% setup by 20% and decreases by 0 to 18% in other percentage setups.

Water Level: Change in formation time at different percentages cause a decrease in Water level ranging from 0.4 to 2.2% at all downstream locations in the model. Details are shown in Table 6.4 and figure 6.3.

Table 6.4. Water Level at Locations Downstream for Test I

Distance	20% Setup	40% Setup	60%s Setup	80% Setup	Original Setup
1Km	27.41	27.25	27.11	26.97	27.58
7Km	22.49	22.4	22.29	22.19	22.6
12Km	21.17	21.09	21	20.91	21.26
18Km	18.81	18.72	18.62	18.53	18.89
23Km	17.04	16.95	16.86	16.76	17.13
28Km	8.59	8.52	8.46	8.4	8.64

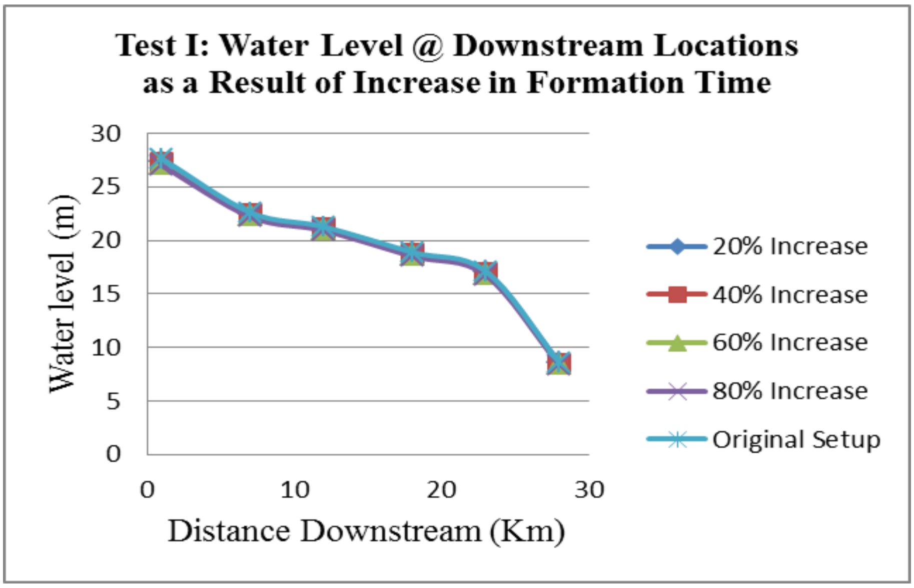


Figure 6-3 Effects of increase in formation time on water level.

6.2 Test II: Decrease in breach formation time

In this setup, the breach formation time is interchanged with a decrease by 20, 40, 60 and 80 percent in the model and all other breach parameters remain constant as in the original setup in Table 4.4 (Froehlich 2008). The outcome of test II. with regards to discharge, velocity, water level and travel time of the flood wave at selected downstream locations of the dam are illustrated below.

Discharge: Decrease in breach formation time, has much more effect on Discharge as compared to the effects caused by the increase in formation time in the study model. The outflow hydrograph is higher. i.e. 20%setup - 41292.93 m^3/s, 40%setup - 43027.09 m^3/s, 60%setup - 45329.63 m^3/s, and 80%setup - 47720.35 m^3/s. Routing the flood through the channel downstream, the increment in discharge ranged from 1.1 to 5.3% when compared with the original setup. See decrease details in Table 6.5 and 6.4

Table 6.5 Discharge at Downstream Locations for Test II

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1Km	40767.66	42261.59	44012.47	45582.25	39401.03
7Km	39624.63	40768.81	41918.64	42821.66	38523.66
12Km	38373.2	39182.24	39940.97	40509.71	37534.67

18Km	37066.1	37613.97	38093.83	38422.84	36467.48
23Km	36081.96	36488.88	36826.12	37041.11	35614.23
28Km	36045.81	36451.12	36786.66	37000.4	35580.58

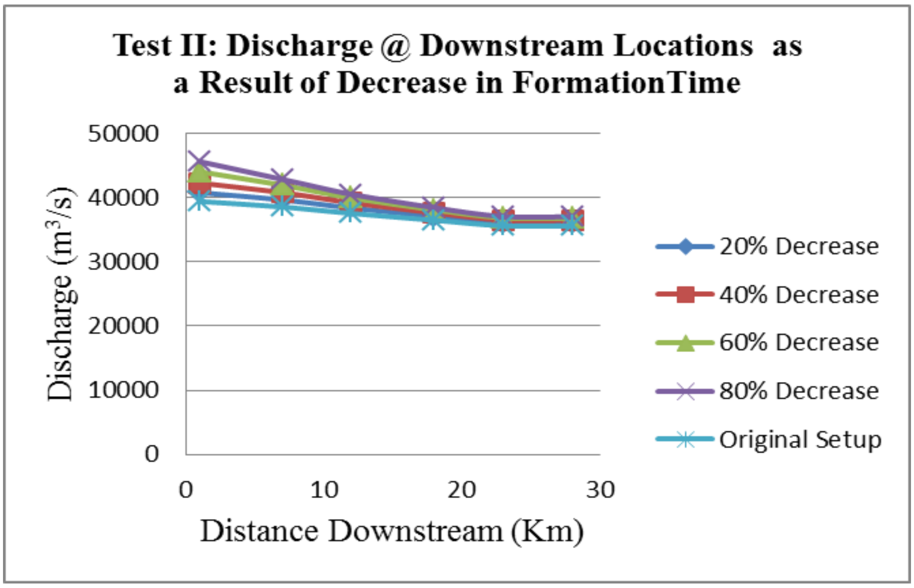


Figure 6-4 Effects of decrease in formation time on discharge.

Velocity: Decrease in formation time increases the velocity of flow by 0.2 to 10.7% in most percentage setups however there are a decrease in velocity at location 23 and 28Km of the 40% setup. The percentages of decrease in velocity at these locations are 3.7% and 4% respectively. See Table 6.6 and figure 6.5. for details.

Table 6.6 Velocity at Downstream Locations for Test II

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1Km	5.02	5.07	5.27	5.47	4.94
7Km	3.67	3.78	3.91	4.07	3.59
12Km	4.46	4.51	4.64	4.72	4.38
18Km	3.14	3.12	3.24	3.29	3.09
23Km	3.53	3.37	3.56	3.57	3.5
28Km	3.82	3.66	3.84	3.85	3.81

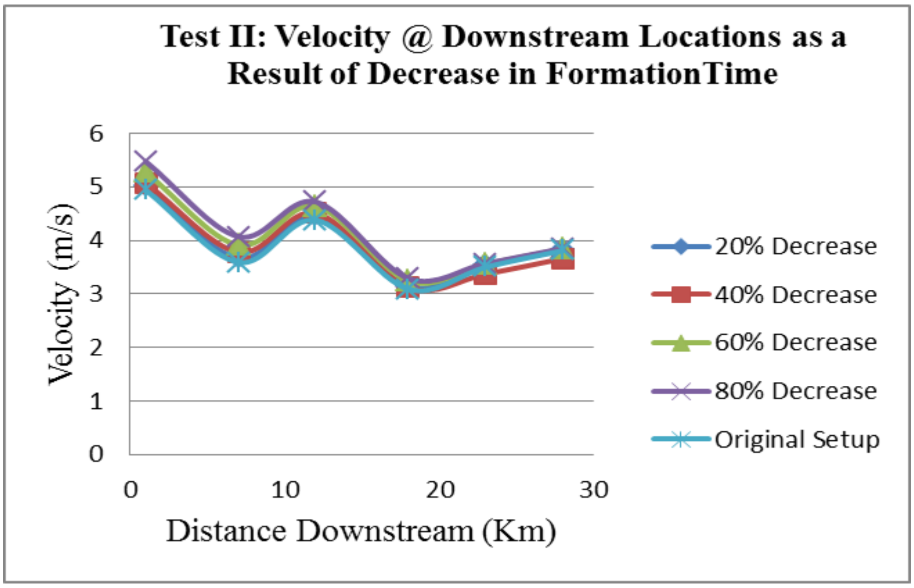


Figure 6-5 Effects of decrease in formation time on velocity.

Water Level: With the Decrease in formation time it is observed that there is an increase in water level at downstream locations ranging from 0 to 2% and also a decrease in water level at 18Km and 23Km of the 60% setup of 0.7% and 1.9% decrease respectively. Water level details are shown both in Table 6.7 and figure 6.6.

Table 6.7 Water Level at Locations Downstream for Test II

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1Km	27.75	27.92	28.04	28.04	27.58
7Km	22.69	22.78	22.27	22.9	22.6
12Km	21.34	21.41	20.84	21.5	21.26
18Km	18.98	19.05	18.75	19.15	18.89
23Km	17.21	17.28	17.13	17.38	17.13
28Km	8.7	8.75	8.64	8.82	8.64

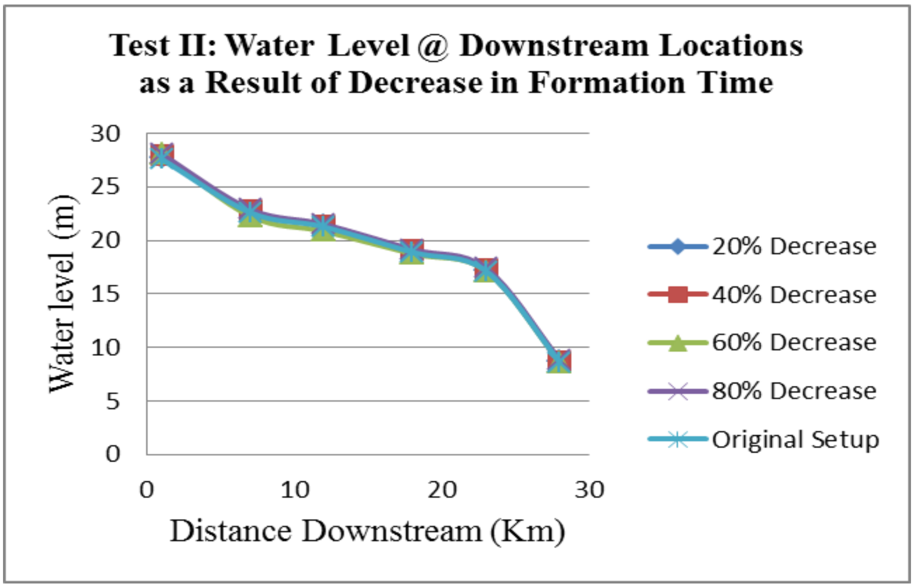


Figure 6-6 Effects of decrease in formation time on water level.

Travel Time: The peak outflow which begins at 60.55hrs after the inflow into the reservoir takes the following time to travel to the end of the model.: 20% - 1.2hrs, 40% - 1.25hr, 60% - 1.37hr and for 80% - 1.52hr as compared to the original setup where the travel time from the dam to the end of the model takes 1.0hr. Travel time increases by 25 to 50%.

6.3 Test III: Increase in breach width

In this setup, the breach width is increased by 20, 40, 60 and 80 percent in the model and all other breach parameters remain constant. The results of Test III. with regards to discharge, velocity and water level and travel time of the flood wave at selected downstream locations of the dam are shown below.

Discharge: Increase in breach width causes an increase in discharge at all the percentage setups. Its effect on the outflow hydrograph from the original peak outflow of 39879.71 m^3/s and the discharge of flood wave routed to the downstream area are as follows: The Peak outflow hydrograph at 20%setup - 42161.35 m^3/s, 40%setup - 44423.18 m^3/s, 60%setup - 46674.30 m^3/s, and 80%setup - 48913.82 m^3/s. Routing the flood through the channel downstream, the increment in discharge ranges from 4.3 to 22.2% when compared with the original setup. Details are illustrated in Table 6.8 and figure 6.7.

Table 6.8 Discharge at Downstream Locations for Test III

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	41610.89	43799.59	45979.77	48151.24	39401.03
7	40592.22	42641.48	44687.28	46733.46	38523.66
12	39443.63	41343.37	43248.62	45139.6	37534.67
18	38188.77	39903.73	41618.48	43322.78	36467.48
23	37171.96	38719.09	40264.78	41785.15	35614.23
28	37131.35	38673.6	40212.34	41726.63	35580.58

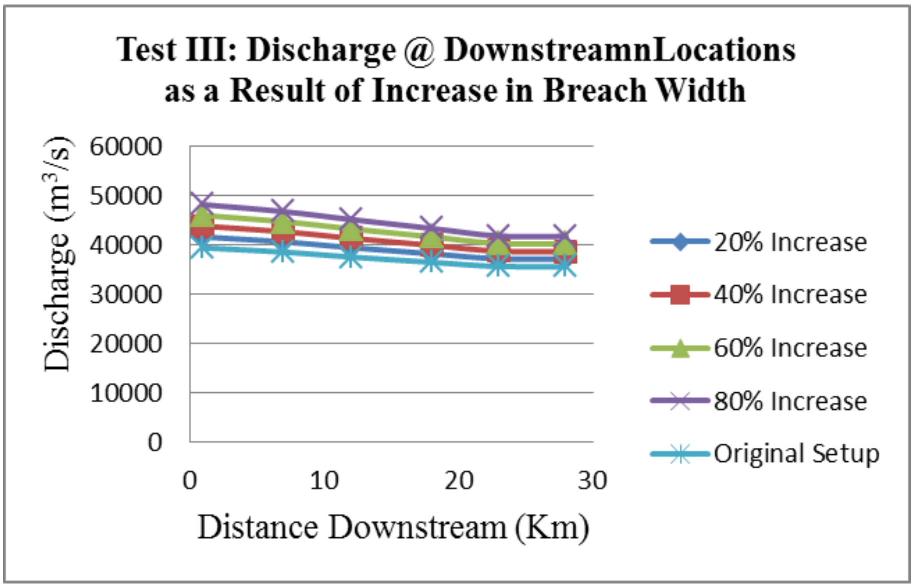


Figure 6-7 Effects of increase in breach width on discharge.

Velocity: Increase in breach width in the model causes an increment in velocity along the channel, ranging from 1 to 10%. See Table 6.9 and figure 6.8. for details.

Table 6.9. Velocity at Downstream Locations for Test III

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	5.03	5.11	5.19	5.27	4.94
7	3.67	3.75	3.82	3.9	3.59
12	4.51	4.63	4.75	4.86	4.38
18	3.16	3.23	3.3	3.36	3.09
23	3.57	3.64	3.69	3.77	3.5
28	3.85	3.9	3.95	4	3.81

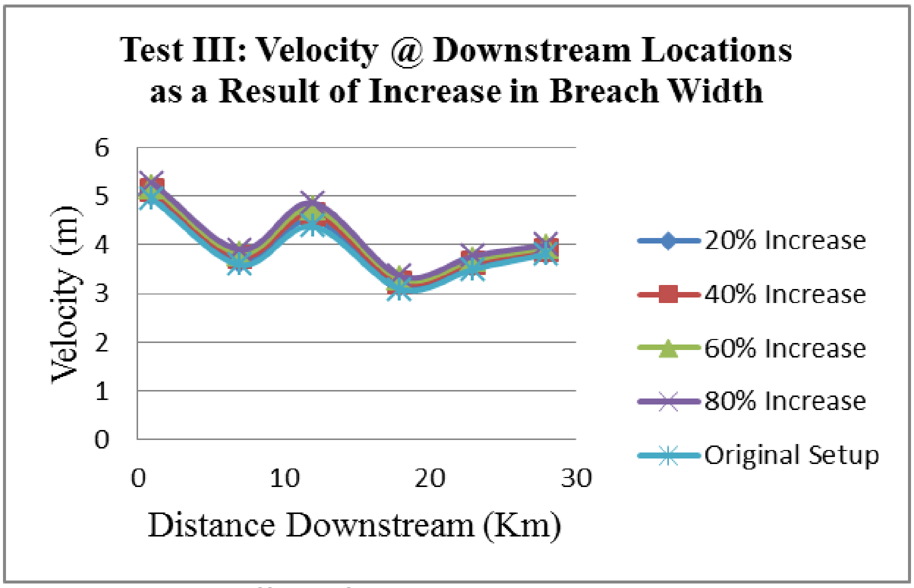


Figure 6-8 Effects of increase in breach width on Velocity.

Water Level: When observing the change of water level caused by the increase in breach width it is realized that there is an increment in the levels, compared to the original setup. The increment ranges from 0.2 to 8.3% at the different percentage setups of increments. See detail in Table 6.10 and figure 6.9.

Table 6.10 Water Level at Downstream Locations for Test III.

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	27.92	28.25	28.3	28.89	27.58
7	22.89	23.17	23.43	23.7	22.6
12	21.52	21.77	21.96	22.26	21.26
18	19.17	19.44	19.7	19.95	18.89
23	17.4	17.67	17.93	18.18	17.13
28	8.84	9.02	9.19	9.36	8.64

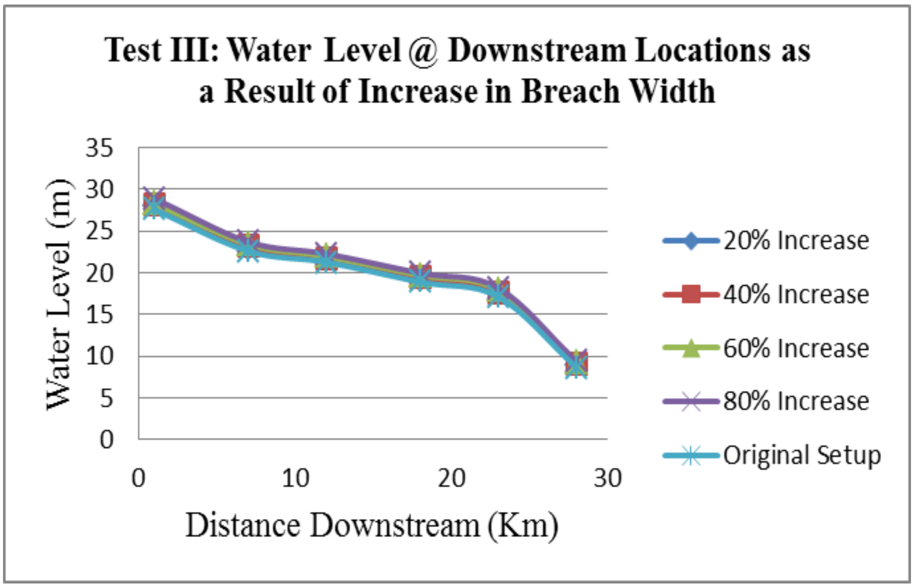


Figure 6-9 Effects of increase in breach width on water level.

Travel Time: The peak outflow which begins at 61.1hrs after the inflow PMF into the reservoir takes the following time to propagate to the end of the model: 20%setup -1.0hrs, 40%setup -1.0hr, 60%setup - 0.96hr and for 80% - 0.96hr as compared to the original setup where the travel time from the dam to the last location takes 1.0hr. Travel time is the same as the original setup for 20%setup and 40%setup, decrease at 60 and 80%setups by 4%.

6.4 Test IV: Decrease in breach width

In this setup, the breach width is decreased by 20, 40, 60 and 80 percent in the model and all other breach parameters remain constant as in the original setup (Table 4.5

Froehlich, 2008). The results of Test IV. with regard to discharge, velocity and water level and travel time of the flood wave at selected downstream locations of the dam are shown below.

Discharge: The decrease in breach width has resulted in a decrease in peak outflow from the original setup peak outflow of 39879.71 m^3/s to the following at different percentage setups: The Peak outflow hydrograph at: 20%setup - 37569.36 m^3/s, 40%setup - 35227.34 m^3/s, 60%setup - 32847.39 m^3/s, and 80%setup - 30429.28 m^3/s. Routing the flood through the channel downstream, the decrease in discharge ranges from 4.4 to 23.1% when compared with the original setup. Details can be seen in Table 6.11 and figure 6.10.

Table 6.11 Discharge at Downstream Locations for Test IV.

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	37170.05	34911.54	32619.15	30289.04	39401.03
7	36430.56	34323.86	32189.58	30030.98	38523.66
12	35604.54	33666.52	31716.62	29748.22	37534.67
18	34733.18	32989.21	31232.64	29460.8	36467.48
23	34041.76	32451.09	30843.81	29222.36	35614.23
28	34014.26	32428.55	30826.86	29211.08	35580.58

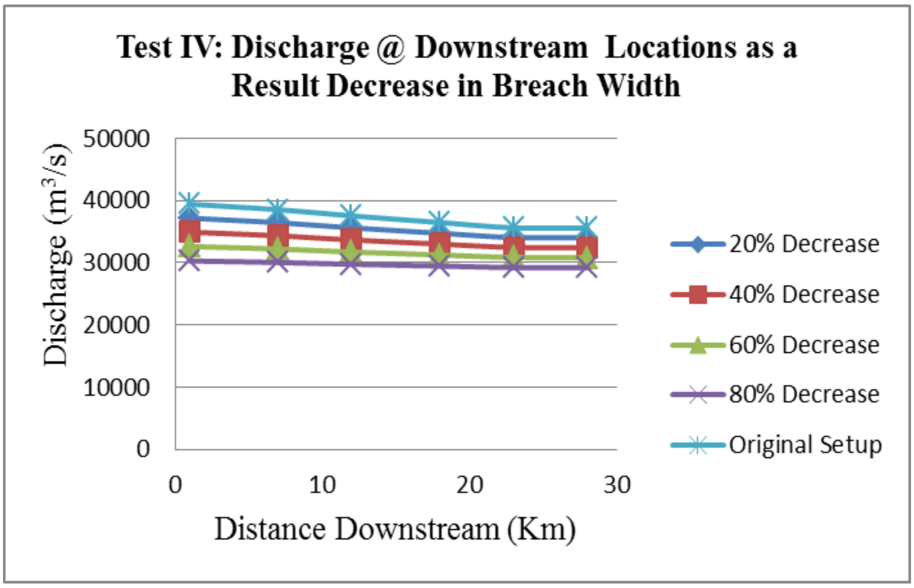


Figure 6-10 Effects of decrease in breach width on discharge.

Velocity: At all downstream locations, velocity decreased at the result of percentage decrease of breach width in the model. The range of decrease is from 2.5 to 10% in comparison to the original setup. Details can be seen in Table 6.12 and figure 6.11.

Table 6.12 Velocity at Locations Downstream for Test IV.

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	4.8	4.7	4.6	4.5	4.94
7	3.5	3.4	3.31	3.21	3.59
12	4.25	4.12	3.98	3.83	4.38
18	3.01	2.94	2.86	2.78	3.09
23	3.38	3.31	3.25	3.18	3.5
28	3.69	3.65	3.61	3.57	3.81

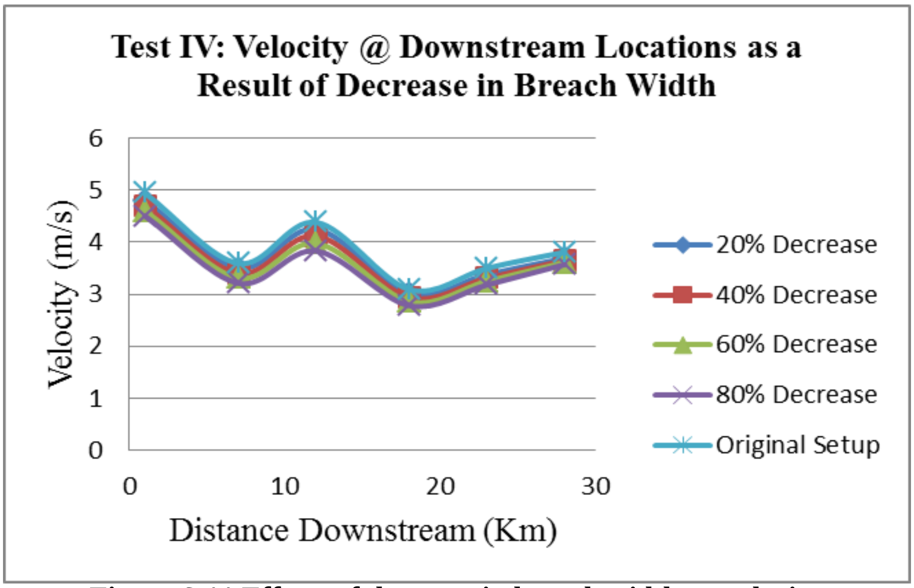


Figure 6-11 Effects of decrease in breach width on velocity.

Water Level: As a result of the changes in the breach width in the model, the model experiences decrease in outflow as well as water level when routed along the downstream of the channel. The percentage of a decrease observed when compared to the original setup, ranged from 1.2 to 9.4%. Details of decrease in percentages are shown in Table 6.13 and figure 6.12.

Table 6.13 Water Level at Locations Downstream for Test IV.

Distance	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	27.22	26.85	26.46	26.05	27.58
7	22.3	21.99	21.68	21.34	22.6
12	20.99	20.71	20.42	20.12	21.26
18	18.51	18.32	18.02	17.71	18.89
23	16.83	16.54	16.25	15.94	17.13
28	8.45	8.24	8.04	7.82	8.64

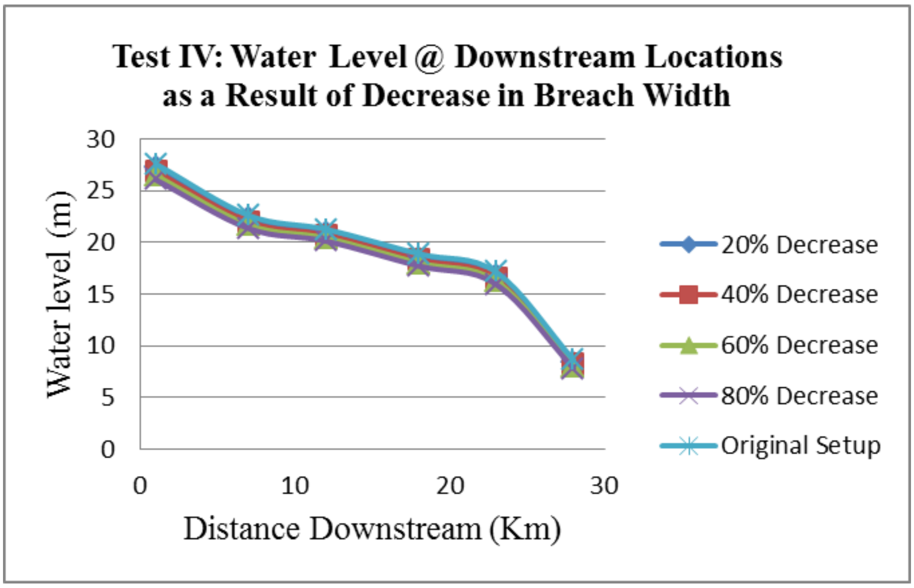


Figure 6-12 Effects of decrease in breach width on water level.

Travel Time: The peak outflow which begins at 61.1hrs after the inflow PMF into the reservoir takes 1.1hr to propagate to the end of the model causing 10% increment in travel time for all percentage decrease setups: 20%setup-1.1hrs, 40%setup-1.1hr, 60%setup-1.1hr and for 80%-1.1hr when compared to the original setup.

6.5 Test V: Increase in breach side slope

In this setup, the breach side slope is increased by 20, 40, 60 and 80 percent in the model and all other breach parameters remain constant as in the original setup (Table 4.5 Froehlich, 2008). The results of Test V. with regard to discharge, velocity, water level and travel time of the flood wave at selected downstream locations of the dam are shown below.

Discharge: Breach side slope increment effects on peak discharge in percentage setup are as follow: The Peak outflow hydrograph at: 20%setup - 40126.65m^3/s, 40%setup - 40371.86m^3/s, 60%setup - 40615.47m^3/s, and 80%setup - 40857.57m^3/s. The increment in percentage setup peak outflow from original setup peak outflow of 39879.71m^3/s ranged from 0.6 to 2.4%. Routing the flood through the downstream of the channel, discharge is observed to increase by 0.4 to 2.4%. See Table 6.14 and figure 6.13 for illustrations of increment along the channel.

Table 6.14 Discharge at Locations Downstream for Test V.

Distance D/S	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	39642.33	39882	40120.06	40356.58	39401.03
7	38751.91	38978.48	39203.46	39427.3	38523.66
12	37748.84	37961.47	38172.61	38382.5	37534.67
18	36664.55	36860.17	37054.38	37247.16	36467.48
23	35792.49	35970.2	36146.5	36321.44	35614.23
28	35757.92	35933.8	36109.2	36283.84	35580.58

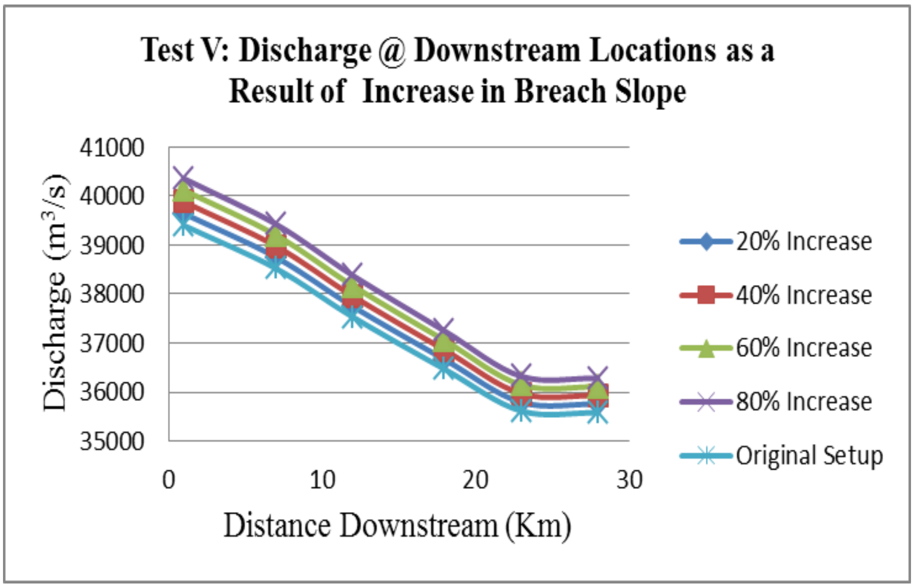


Figure 6-13 Effects of increase in breach side slope on discharge.

Velocity: The increase in breach side slope in the model caused an increase in velocity at most downstream locations with the exception to 20% setup at distance 12 Km and 40%setup at 7Km; these locations experienced 29% and 23% decrease in velocity respectively as compared to the original setup velocity at this location. However, other locations velocities increase by 0.2 to 13.5% in comparison to the original setup velocity. Details of velocity can be seen in Table 6.15 and figure 6.14.

Table 6.15 Velocity at Locations Downstream for Test V.

Distance D/S	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	4.95	4.96	4.97	4.94	4.94
7	3.6	2.74	3.61	2.75	3.59
12	3.1	4.41	4.42	4.44	4.38
18	3.51	3.11	3.11	3.12	3.09
23	3.81	3.52	3.53	3.53	3.5
28	3.82	3.82	3.82	3.83	3.81

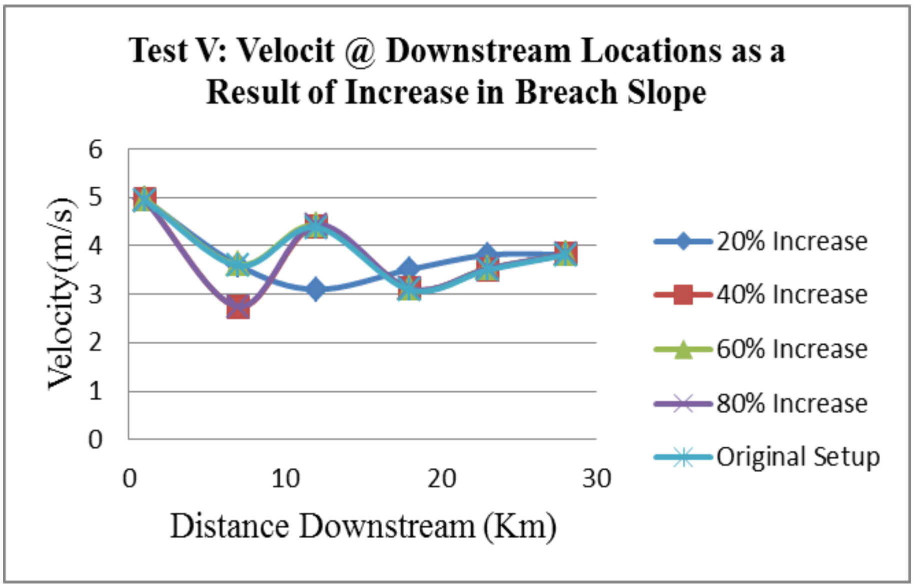


Figure 6-14 Effects of increase in breach side slope on velocity.

Water Level: As a result of the increase in breach side slope in the model, outflow increased as well as water level when routed along the channel. The percentage of increase observed when compared to the original setup, ranged from 0.1 to 1.0%. Details of increase in percentages are shown in Table 6.15 and figure 6.15.

Table 6.16 Water Level at Locations Downstream for Test V.

Distance D/S	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	27.61	27.65	27.69	27.73	27.58
7	22.63	22.67	22.7	22.73	22.6
12	21.29	21.32	21.35	21.38	21.26
18	18.93	18.96	18.99	19.02	18.89
23	17.16	17.19	17.22	17.26	17.13
28	8.67	8.68	8.71	8.73	8.64

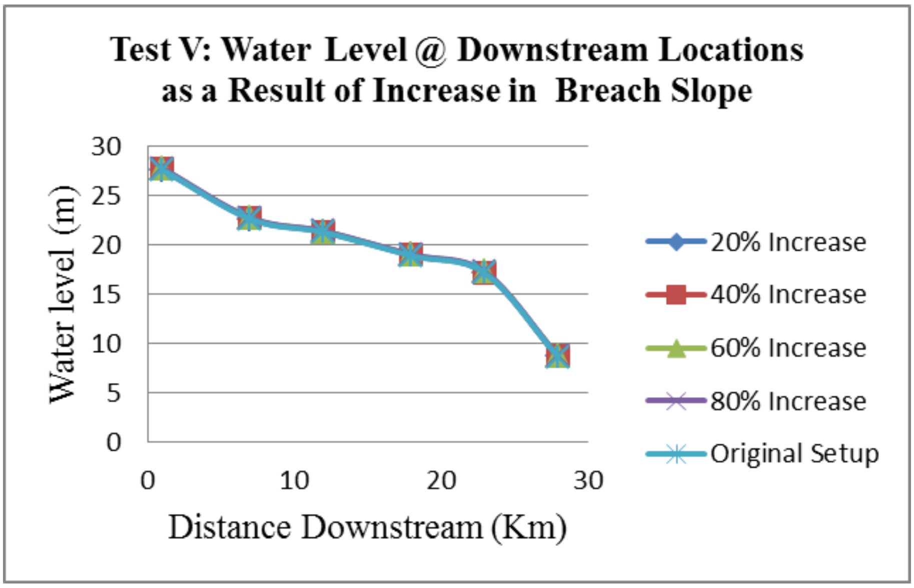


Figure 6-15 Effects of increase in breach side slope on water level.

Travel Time: The peak outflow which begins at 61.1hrs after the inflow PMF into the reservoir takes 1.1hrs to propagate to the end of the model causing 10% increment in travel time for all percentage decrease setups: 20%setup - 1.1hrs, 40%setup - 1.1hrs, 60%setup - 1.1hrs and for 80% - 1.1hrs when compared to the original setup.

6.6 Test VI: Decrease in breach side slope

In this setup, the breach side slope is decreased by 20, 40, 60 and 80 percent in the model and all other breach parameters remain constant as in original setup (Table 4.5 Froehlich, 2008). The results of Test VI. with regard to discharge, velocity, and water level and travel time of the flood wave at selected locations downstream of the dam are shown below.

Discharge: Effects of decrease in breach side slope on peak discharge in percentage setup is a decrease at various downstream locations. The Peak outflow hydrograph at: 20%setup - 39631m^3/s, 40%setup - 39380.49m^3/s, 60%setup - 39128.15m^3/s, and 80%setup - 38873.96m^3/s. Decrease in percentage setup peak outflows from original setup peak outflow of 39879.71 m^3/s ranges from 0.6 to 2.5%. Routing the flood through the downstream channel, discharge is observed to a decrease by 0.5 to 14%. See Table 6.17 and figure 6.16 for details of decrease in downstream discharge.

Table 6.17 Discharge at Downstream Locations for Test VI.

Distance Downstream	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	39158.1	38913.55	33991.32	33864.01	39401.03
7	38293.66	38061.91	34751.88	34606.63	38523.66
12	37318.89	37101.45	35159.63	35000.91	37534.67
18	36268.92	36069.13	35288.99	35117.44	36467.48
23	35435.11	35254.48	35072.36	34888.78	35614.23
28	35401.76	35221.43	35039.62	34856.33	35580.58

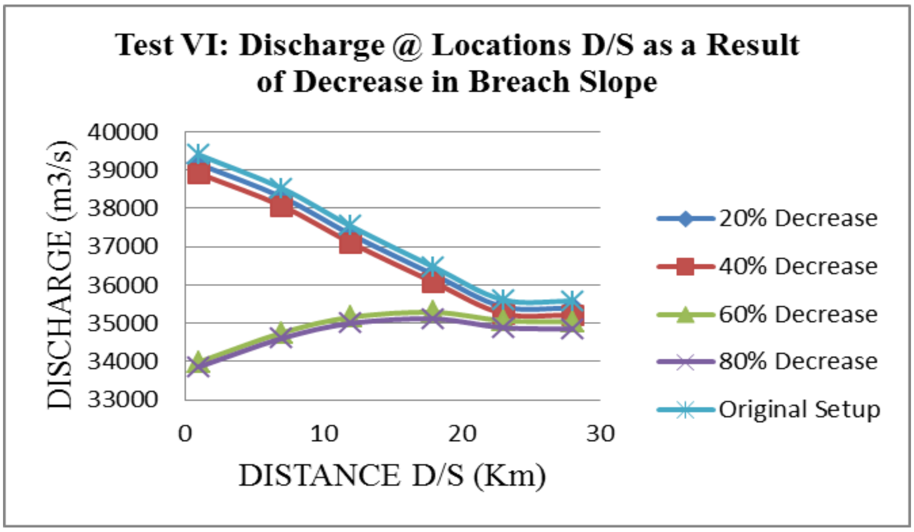


Figure 6-16 Effects of decrease in breach side slope on discharge.

Velocity: Decrease in breach side slope in the model causes a decrease in most percentage setups except 40% setup at 23Km where the velocity is increased by 19%. The decrease of velocity at other downstream locations ranges from 0.2 to 1.1% when compared the original setup velocity. Details of velocity can be seen in Table 6.18 and figure 6.17.

Table 6.18 Velocity at Downstream Locations for Test VI.

Distance Downstream	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	4.93	4.92	4.94	4.9	4.94
7	3.58	3.57	3.56	3.55	3.59
12	4.37	4.35	4.34	4.33	4.38
18	3.08	3.08	3.07	3.06	3.09
23	3.44	4.18	3.48	3.47	3.5
28	3.79	3.78	3.79	3.78	3.81

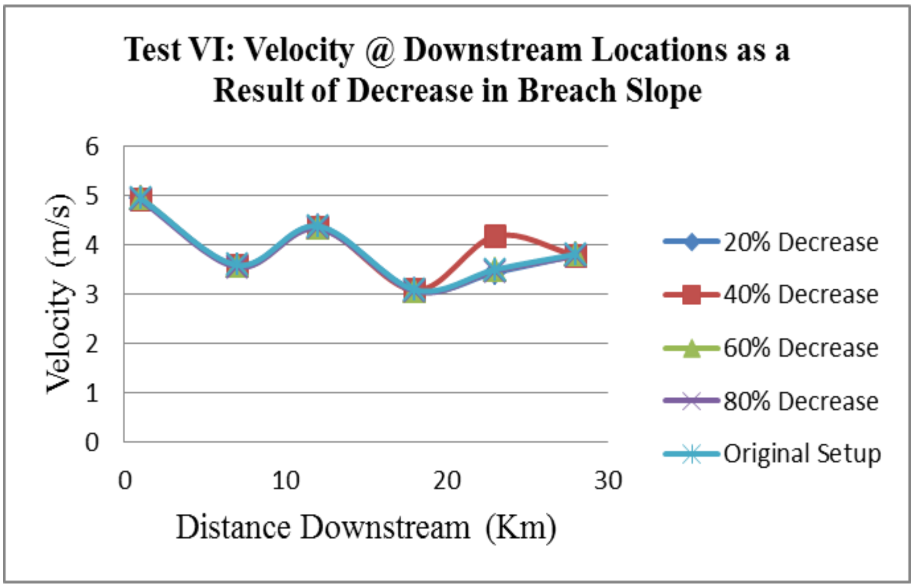


Figure 6-17 Effects of decrease in breach side slope on Velocity.

Water Level: As a result of the decrease in breach side slope in the model, outflow through the breach decreased as well as water level when routed along the channel. The percentage of a decrease observed when compared to the original setup, ranged from 0.1 to 0.9%. Details of the decrease in percentages are shown in Table 6.19 and figure 6.18.

Table 6.19 Water Level at Downstream Locations for Test VI.

Distance Downstream	20% Setup	40% Setup	60% Setup	80% Setup	Original Setup
1	27.54	27.5	27.46	27.42	27.58
7	22.57	22.53	22.5	22.46	22.6
12	21.23	21.2	21.17	21.14	21.26
18	18.86	18.83	18.8	18.77	18.89
23	17.1	17.07	17.03	17	17.13
28	8.63	8.6	8.58	8.56	8.64

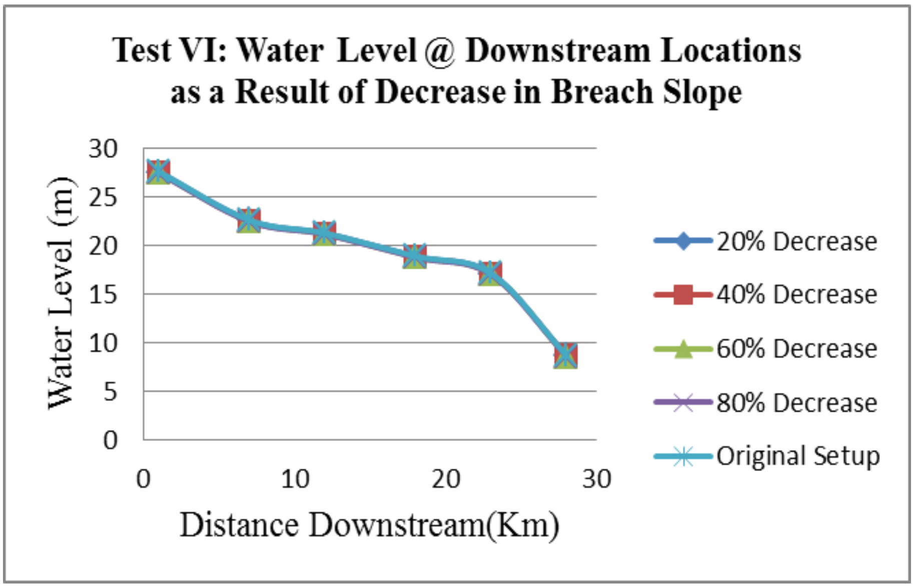


Figure 6-18 Effects of decrease in breach side slope on Velocity.

Travel Time: The peak outflow which begins at 61.1hrs after the inflow PMF into the reservoir takes 1.1hr to propagate to the end of the model causing 10% increment in travel time for all percentage decrease setups: 20%setup-1.1hrs, 40%setup-1.1hr, 60%setup-1.1hr and for 80%-1.1hr when compared to the original setup.

6.7 Discussion

The sensitivity analysis is done with the comparison of the original setup adopted by the study which is the Froehlich, 2008 method of breach prediction that reflected the worst-case scenario of overtopping dam breach at the Mount Coffee Dam. Based on this, six (6) different tests comprising of four percentage setups each with interchanged parameters of by both increasing and decreasing in percentage were formulated. It is proved that the most sensitive effects were on travel time. Out of the total of 24 percentage setups, the following are recorded: 17 increment effect from 3-20%, 4 decrease effects from 4-18% and 3 equilibrium effects to the original setup travel time for interchange in all parameter tests. Percentage increase in breach width is most sensitive on peak outflow, routing peak discharge downstream, velocity and water level. At the increase in breach width, the percentage increment in peak outflow ranges from 0.6-22%, in peak discharge at downstream locations, increment ranges from 0.4 to 22.2%, in flow velocity an increment of 1-10% and in peak water level, an increment of 0.2-8.3%. These effects recorded for the increase in breach width are the most sensitive when compared to the increase in breach slope, and the decrease in formation time that does have incremental effects on peak outflow, peak discharge downstream, velocity and peak water level. Decrease in formation time also causes an increase in peak outflow, peak discharge downstream, velocity and water level, however; these outcomes are fluctuating at the increase and decrease along the channel. As the increase in breach width produced the highest incremental values for the outcome parameters, a decrease in breach width produces the highest value of decrease in peak outflow, peak discharge downstream and water level.

Chapter 7

Conclusion

The study was set to explore dam break worse-case scenario, the importance, the methods and extent of the flood impact on the inundation zone. The study also evaluated the adequacy of the present spillway design flood using the incremental increase method of routing flood flow downstream of the dam. The Hatch investigation framework of and results were based on a familiar HEC-RAS computer software (Hatch, 2008). However, from this study using the 1-dimensional MIKE 11 hydrodynamic software, it is seen that a more hazardous effect of dam break at Mount Coffee is expected at any inflow greater than 0.3 PMF into the reservoir at maximum water level. This study coincides with Hatch's inflow design flood method established by FERC but a contrast in breach parameter results of worst-case scenario. Regression formulas developed by FERC and Froehlich, 2008 for breach prediction parameters were simulated into MIKE 11, outflow flood, and routing of the flood analyzed. Results proved that Froehlich, 2008 breach prediction parameter outflow as well as routing of flood wave downstream hazardous propensity is higher than that of FERCs' breach prediction parameter. Base on this, it was concluded that further investigation is done using the Froehlich, 2008 simulation results. Therefore, the flood inundation map, sensitivity analysis, and inflow design flood investigations were formulated based on Froehlich, 2008 effects. From the flood map, it is seen that lots of inhabitants further downstream will be affected due to the dam-break. Submerged areas include portion of New Kru Town, Caldwell, Clay Ashland, Yawah Town and Baker's Community. Findings from the sensitivity analysis made it known that the most sensitive breach parameter in this study is the breach width. The increase in breach width caused high increased in the outflow, velocity, water level and decrease or equilibrium in travel time. For output parameters, the most sensitive are the travel time. At almost every interchange of breach parameter, the travel time either increase or decrease. Evaluation of the recent Mount coffee spillway capacity that is 9910m^3/s is inadequate for smaller fractions of the probable maximum flood (PMF) except 0.3PMF- 8155.3 m^3/s. Other smaller fractions include 0.4PMF-10,873.7 m^3/s, 0.7PMF - 19,028.9 m^3/s , and 0.8PMF- 21,747.4 m^3/s however, any amount of water more than the 0.3PMF discharge to the downstream from the spillway will pose a threat to the inundation area.

7.1 Recommendations

The results acquired from this study have prompted the following recommendations:

i. As an aspect of this study, I recommend that it is worthwhile to consider a risk management and environment impact analysis.

ii. To reduce the level of risk posed to the downstream of the dam, a weir constructed further upstream of the dam can serve as a mitigation measure.

iii. The flood map is used as a guide in planning future development in the inundation zone.

Chapter 8

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