DAM BREAK ANALYSIS
A CASE STUDY

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree of

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(C I V I L)

By

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≡ June, 2004 ≡

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the dissertation entitled, "**DAM BREAK ANALYSIS A CASE STUDY**" in partial fulfillment of the requirement for the award of Degree of Master of Technology in WRD (CIVIL) submitted in the Water Resources Development Training Centre, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out during the period 20 July, 2003 till the date of submission under supervision of **Dr. Ram Pal Singh**, professor, WRDTC, IIT Roorkee and **Dr. B.N. Asthana**, visiting professor, WRDTC, IIT Roorkee, India.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

Dated : 18 ,June, 2004

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(EKO WAHYUDI)

ABSTRACT

Batutegi dam is an earthen dam. The dam is located on Way Sekampung River in Lampung Province, which is the southern most province of the *Sumatra* Island, the western most large island of Indonesia Archipelago. The simulation of dam break flood wave due to a hypothetical failure of Batutegi Dam using the BOSS DAMBRK Model developed by Dr. D L Fread is presented here. The data available for the analysis are various design floods for dam, PMF hydrograph, spillway rating curve, elevation area capacity table for reservoir, cross-sectional details of various downstream reaches etc. More emphasis is given to analyse the sensitivity of dam break flood wave characteristics (stage and discharge) due to the changes in dam breach parameters (time taken for breach formation, inflow hydrograph to the reservoir, final size of breach and breach slope). The option of reservoir storage routing to compute outflow hydrograph from reservoir for different combinations of breach parameters and option of sub-critical dynamic routing for routing this flood hydrograph through the downstream channel is used.

The study of Batutegi Dam Break Analysis involves :

- Identification of the inflow hydrograph to the reservoir at the time of failure
- Flood routing of the hydrograph through the reservoir
- Development of the failure condition of the structure
- Calculating the outflow hydrograph as a result of the failure of structure
- Modeling the movement of the flood wave downstream to determine travel time, maximum water level reached and peak discharge.

Sensitivity analysis has been carried out for the breach parameters such as breach development time, breach width and breach slope to investigate the effect of change in values of each parameter on water levels, peak discharge and travel time of peak flow through the river.

For sensitivity analysis of each dam break variable on the resulting dam break flood discharge on the downstream of the dam, various runs have been made using Dambrk

Model. Analysis has been repeated by applying four design inflow hydrographs and no inflow to the reservoir condition with various values of three dam breach parameters.

The result of sensitivity analysis of breach parameters shows, that all the breach parameters are sensitive with regard to peak outflow discharge. An increase in flood inflow hydrograph, in final breach width, the breach slope have an effect to increase the peak outflow discharge. However an increase in breach development time has an effect to decrease the peak outflow discharge through breach formation

Also, from various studies, it is seen that changes in inflow hydrograph to the reservoir have insignificant/slight effect on the flood wave characteristics. It may be because of large surface area and storage capacity of the reservoir when compared to the total inflow to reservoir.

The following points can be summarize from this study :

a. Two scenarios of dam failure are applied. First is dam failure due to overtopping and second dam failure due to piping. The results of both scenarios show that dam failure due to overtopping has greater adverse effect on the down stream area than the piping failure.

b. Flood stage and peak discharge values are increasing along the river, when related to the increase in the breach parameters except the time taken in breach formation.

c. Based on results of breach parameter sensitivity analysis, selected values of breach parameters are used to simulate the PMF inflow hydrograph to the Batutegi reservoir. The result shows the maximum peak discharge just below dam site as 78,610 m^3/sec and at the end of available cross section (about 163.650 km) downstream of dam only 5,000 m^3/sec.

d. According to the flood surface elevation along the river down stream of dam site, in most of the river reaches the water levels are more than the bank flooding levels. This indicates that at many locations, the river flood wave is over the bank.

e. The time duration of flooding over the flood plain area along the downstream of dam site varies. The duration time varies from 3.2 hr to the maximum of 100.96 hr. The long duration of inundation time occurs in flat reaches of the river.

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CHAPTER I
INTRODUCTION

1.1. GENERAL

A dam is constructed for economic development, and its construction involves large investment of money, and natural and human of the region resources. A dam is an important hydraulic structure and its purpose is to store excess water in the reservoir and to release it for irrigation, whenever there is a demand for water by the crops in the surrounding area, due to uneven distribution of rainfall in space and time. It may also be constructed to moderate flood flows through a river and thus to protect the downstream areas from loss of life and property. Other purposes for which a dam is constructed include water supply, hydropower generation, navigation, etc. Usually areas downstream of a dam and adjoining areas are highly cultivated because of high fertility of the flood plains and/or availability of water through canal network, and are consequently thickly populated because of various developmental activities.

Dams are of various types. Earth dams are the most common type and constitute the vast majority of dams, but earthen dams are more susceptible to failure than any other type. The most common causes and modes of failure of earthen dams can be summarized as :

1. Overtopping caused by extreme floods
2. Structural failure due to internal erosion (piping)
3. Structural failure due to shear slide
4. Structural failure due to foundation problems
5. Failure due to natural or induced seismic

Singh presents an exhaustive list of causes for dam failure. The causes of dam failure are mainly (***Singh, 1996***) :

- 30% (±5%) flood exceeding spillway capacity

- 37% (±8%) foundation problems (seepage, piping, excess pore pressures, fault movement, settlement)
- 10% (±5%), slides (earth, rock, glacier, avalanches), and
- 23% (±12%) improper design and construction, inferior quality of material, acts of war, lack of operation and maintenance.

Foundation problems and overtopping are thus most important reasons for dam failure. Also, the probability of failure is much greater for embankment dams than for concrete and masonry dams. After 1990, almost half of the failures were due to overtopping. In 41% of those cases, the spillway was underdesigned whereas in 21% overtopping was caused by problems with spillway gate operation.

1.2. BACKGROUND

The dam safety problem is an important issue. In recent years, significant effort has been directed at determining the safety of dams in different countries. One aspect of dam safety is the potential for loss of human life and extensive property loss in downstream flood plain that would result in the event of dam failure. The organizations which are responsible for the safety of dams have to plan for preventive measures so that in the eventuality of dam failures the disaster will not lead to loss of lives of the population living in downstream areas.

One of the preventive measures in avoiding dam failure disaster is by issuing flood warning to the public downstream when there is a failure of a dam. However, it is quite difficult to conduct analysis and determine the warning time of the dam break flood at the time of disaster. Therefore, pre-determination of the warning time assuming various hypothetical dam break situations is an essential exercise in dam safety measures. Further a knowledge of the case studies of the dam failures would give an insight in evaluating and reviewing the existing conditions of the dams.

1.3. OBJECTIVES OF STUDY

The dam failure will generate flood wave propagation to the downstream and may threat human lives and properties, because many public facilities and residential areas exist in downstream area. The objective of Batutegi Dambreak Analysis is a study of hypothetical failure of Batutegi Dam in Lampung Province, Indonesia and resulting flood wave movement. The dam break flood wave characteristics are presented at different sections or areas downstream of Batutegi Dam.

1.4. SCOPE OF DISSERTATION WORK

The study will involve :

- Identification of the inflow hydrograph to the reservoir at the time of failure
- Flood routing of the hydrograph through the reservoir
- Development of the failure condition of the structure
- Calculating the outflow hydrograph as a result of the failure of structure
- Modeling the movement of the flood wave downstream to determine travel time, maximum water level reached and peak discharge.

This study uses the DAMBRK Model, which is released by BOSS Corporations. It s an expanded version of a practical operational model first presented by Dr. Fread,D.L (1977) from The U.S. National Weather Service (NWS), after taking into consideration the practical aspects of dam failure condition and subsequent flood wave movement, for the purpose of forecasting downstream flooding with reference to flood inundation information and warning times resulting from dam failure.

1.5. ORGANIZATION OF DISSERTATION

This study report is composed of following seven chapters :

Chapter I : ***INTRODUCTION***, provides the background of the study, objective of dam break study, scopes of study and organization of dissertation.

Chapter II : ***DESCRIPTION OF PROJECT AREA***, outlines the present conditions of Way Sekampung river basin including salient features of Batutegi Dam.

Chapter III : ***REVIEW OF LITERATURE***, deals with literature review of the factors considered in dam break analysis such as breach characteristics, breach parameters and flood routing considerations.

Chapter IV : ***METHODOLOGY***, this chapter deals with the methodology for the dam break analysis. Basic theory of dam break analysis is important part of this chapter.

Chapter V : ***AVAILABILITY OF DATA***, this chapter presents the source of data and availability of data for Dam Break Model operation.

Chapter VI : ***RESULT AND DISCUSSION***, presents the results of dam break model and discussion of the result. Sensitivities of breach parameters were examined, considering the possibility of breach formation.

Chapter VII : ***CONCLUSIONS***, The prominent conclusions of study are highlighted.

CHAPTER II
DESCRIPTION OF THE STUDY AREA

2.1. PROJECT DESCRIPTION

The study area is located in *Lampung* Province, which is the southern most province of the *Sumatra* Island, the western most large island of Indonesia Archipelago. The location of the study area is shown in ***Figure II-1***. The *Way Sekampung* River originates from the eastern slopes of the *Bukit Barisan* mountain range in the Province of *Lampung*. It flows eastward to the *Java* Sea by the eastern coast of *Sumatera*.

Average annual rainfall in the study area generally varies from 2,000 mm to 3,000 mm. The mean annual number of raining days varies with altitude from about 130 days in the plains area to 210 days at elevation 1,000 meters. The maximum daily rainfall observed is 205 mm. The *Batutegi* dam site is located in the *Barisan* Mountain on the *Way Sekampung* River, about 80 kilometers upstream of the existing *Argoguruh* Weir. Water is diverted to the *Way Sekampung* Irrigation Project at *Argoguruh* Weir. The basin area of *Way Sekampung* River at *Batutegi* Dam site is 424 Km^2.

The population of the *Way Sekampung* basin has increased rapidly due to the growth of agriculture and industry. The *Batutegi* Dam was planned to meet the increasing water demand of the *Way Sekampung* basin population. The increased water demand is for agricultural, municipal, and industrial uses

The *Batutegi* dam is a zoned rock-fill type with a sloping impervious core. The dam is 690 meters long, 113 meters high and 12 m wide at dam crest. The reservoir has an initial live storage of 580 Mm^3. Sedimentation will reduce this capacity to about 578 Mm^3 in fifty years. The design flood has a peak inflow of 5.300 m^3/sec and routed outflow 0f 2,300 m^3/sec. The flood was based on a PMP storm that was estimated to cause a 24 hour maximum point rainfall of 600 mm. The design flood is preceded by a 100 year flood to ensure a full reservoir, an operation spillway, and wet antecedent watershed conditions. Construction of the dam started in 1994 and was completed in 2002.

The downstream valley is narrow for the first 10 Km and becomes wider from this point. The flat plain extends over the right bank side and its width is around 5 km to 8 km from 20 km to 40 km downstream of this point. Then the terrain contracts at 50 km downstream point, just 5 km downstream of confluence with the *Way Buluh* River. This contraction portion continues up to about 55 km downstream point. Thereafter the *Way Sekampung* River flows in the relatively flat terrace, and reaches the *Argoguruh* Weir, and then starts meandering in the flood plains up to the river month. The longitudinal gradients of the riverbed of *Way Sekampung* River are shown in ***Figure II-2***. The river can be divided into six sections according to the topographical characteristic.

2.2. SALIENT FUTURE OF BATUTEGI DAM

The main features of the *Batutegi* Dam are summarized as follows:

Reservoir and Hydrology

Catchment Area	:	424	Km^2
Reservoir Area	:	17	Km^2
Average Annual runoff	:	707.5	MCM
Probable Maximum Flood	:	5,350	m^3/sec
500 year flood return period	:	2,830	m^3/sec
100 year flood return period	:	1,810	m^3/sec
50 year flood return period	:	1,660	m^3/sec
25 year flood return period	:	1,500	m^3/sec
Gross storage capacity	:	580	MCM
Effective storage	:	578	MCM (from El. 274 m to El. 208 m)
Flood Water Level	:	El. 281.50 m	
Normal High Water Level	:	El. 274.00 m	
Low Water Level	:	El. 208.00 m	

Dam

Type	:	Centre core rock-fill
Dam height	:	122 m
Crest length	:	690 m
Crest elevation	:	El. 283.00 m
Widths of crest Dam	:	12 m
Upstream face slope of Dam	:	1 V : 2.25 H
Downstream face slope of Dam	:	1 V : 1.75 H
Slope of inner core of Dam	:	1 V : 0.25 H
Embankment volume	:	9.60 MCM

Spillway

Location	:	Left abutment
Type	:	Tunnel spillway (ungated)
Design discharge	:	1,930 m^3/sec
Weir crest elevation	:	El. 274.00 m

Irrigation / Power Waterway

Location	:	Left abutment
Type of intake	:	Inclined intake structure
Maximum discharge	:	180 m^3/sec
Intake sill elevation	:	El. 201.72 m
Waterway type	:	Circular concrete-lined tunnel
Diameter of Tunnel	:	3.50 m
Waterway length	:	340 m

Power Plant

Location	:	Just downstream of the main dam
Generation capacity	:	28 MW (2 units x 14 MW)

The section and elevation of the Batutegi Dam are shown in ***Figure II-3***.

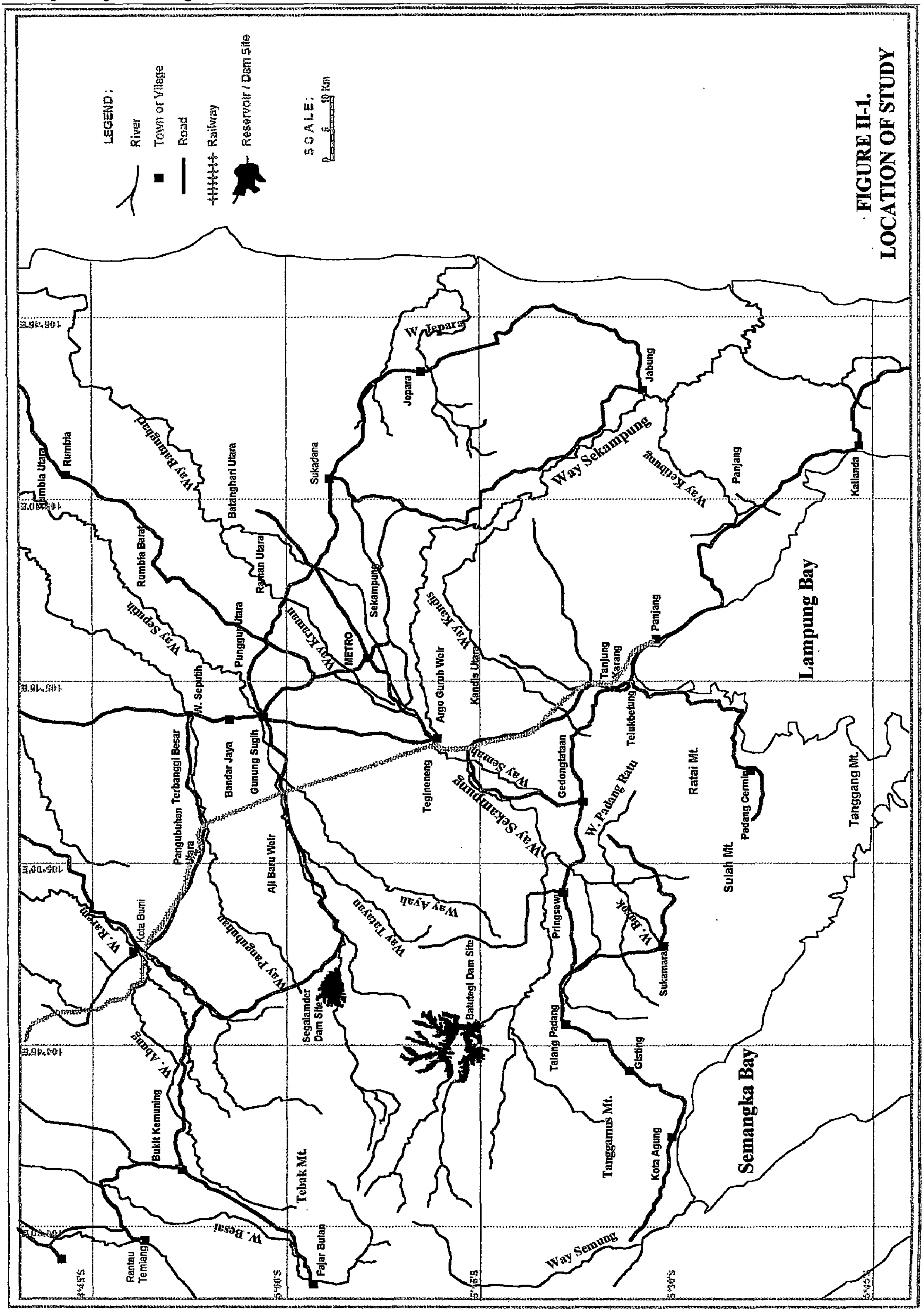

FIGURE II-1.
LOCATION OF STUDY

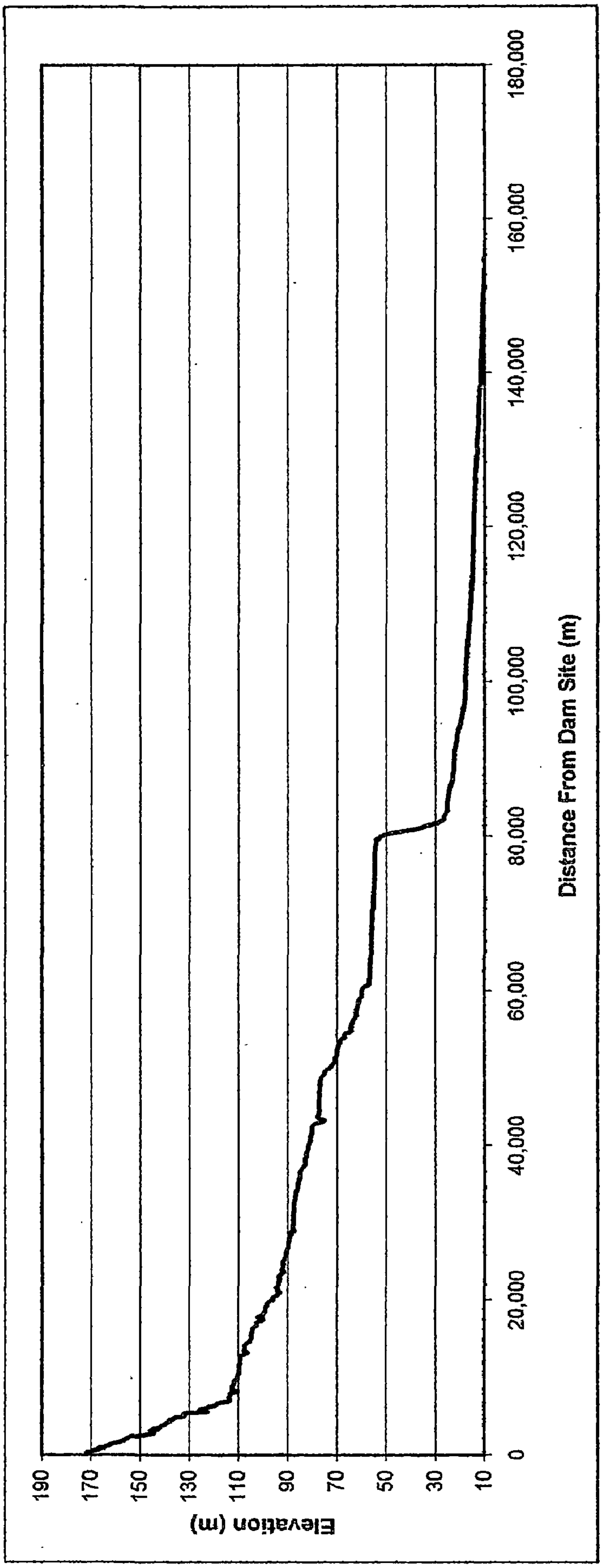

Figure II-2. Longitudinal Section of Way Sekampung River Down Stream of Dam Site

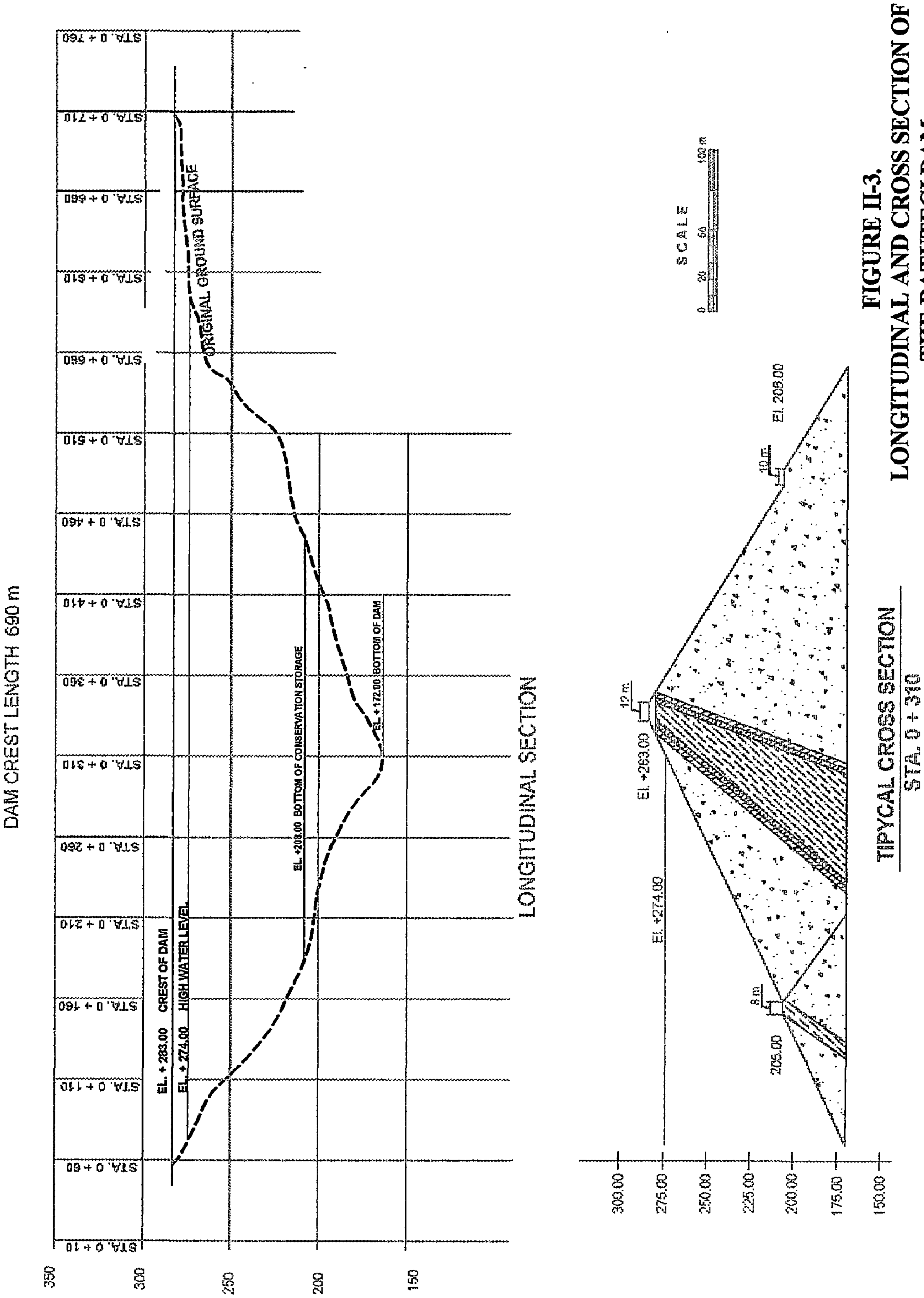

FIGURE II-3.
LONGITUDINAL AND CROSS SECTION OF THE BATUTEGI DAM

CHAPTER III
REVIEW OF LITERATURE

3.1. REVIEW OF LITERATURE

A comprehensive investigation of the hydraulics of dam failure involves flood hydrology, mechanics of erosion and sediment transport, hydraulics of flow over the crest and through the breach, geomechanics of slope instability, hydraulics of channel flow and floodplains, dam regulation and management, disaster alleviation, and damage assessment. Some of these elements are interconnected as shown in ***Figure III-1***.

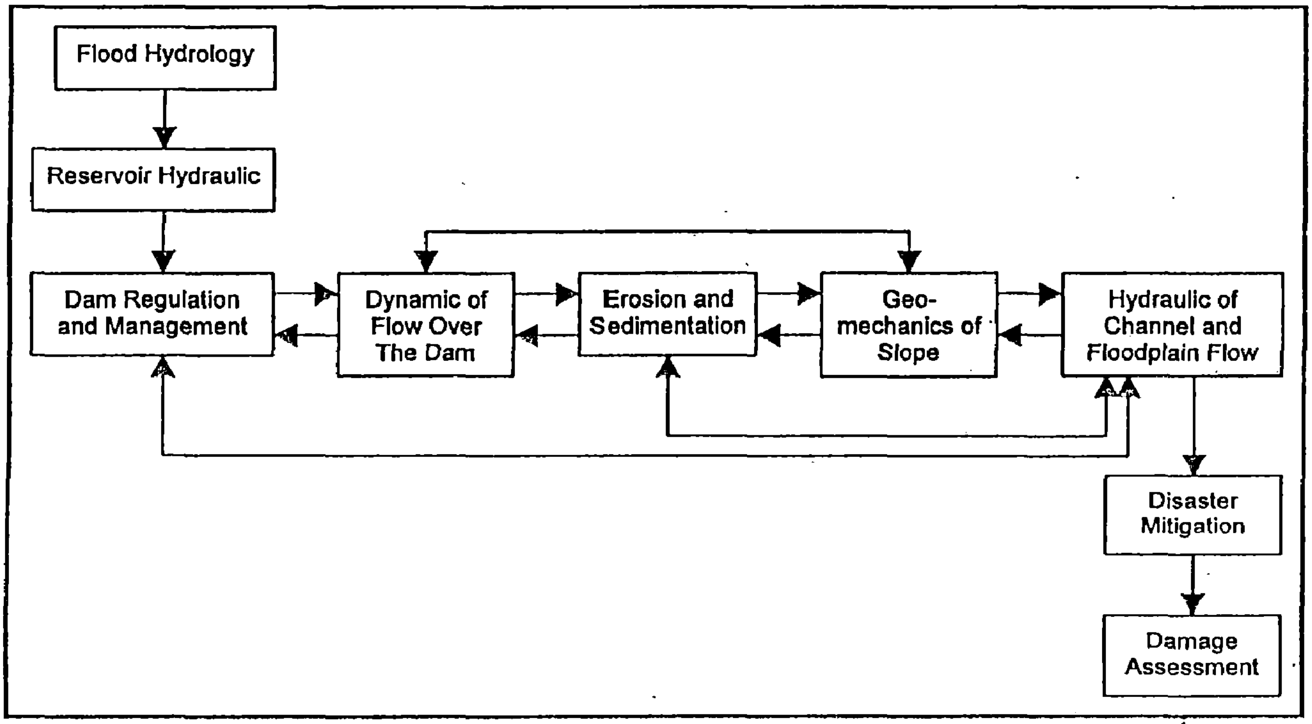

Fig. III-1. A Schematic for Comprehensive Study of The Hydraulic of Dam Breaching

A complete mathematical model incorporating all of these elements does not appear to have been reported yet in literature. Most models are based on two or more elements that hover around breach formation and dam-breach flood routing (***Singh,*** *1996*).

The first dam break study was carried out by Ritter (1892)[29], who used the method of characteristics to obtain a closed form solution for a dam of semi-infinite extent upon a horizontal bed with zero bed resistance. Both experimental and theoretical consideration, however, have shown that the neglect of bed resistance invalidates the Ritter solution in a region that starts near the leading edge of the flood wave. The U.S. Army Corps of Engineers (1960) have recognized the need to assume partial rather than complete breaches, however, they assumed the breach occurred instantaneously. The assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analysis of dam break flood waves.

Recognizing this practical aspect Cristofano (1965)[29], Harris and Wagner (1967)[3]incorporated the partial time-dependent breach formation in their dam break models. Cristofano (1965) [23] attempted to model the partial, time-dependent breach formation in earthen dam, however this procedure requires critical assumptions and specification of unknown critical parameter values. Also Harris and Wagner (1967)[5] used a sediment transport relation to determine the time for breach formation, but this procedure requires specification of breach size and shape in addition to two critical parameters for the sediment transport relation.

Today there are numerous tools available for analyzing dam failure and the resulting floods. Wurbs (1987)[29] compared state of the art models, including the National Weather Service (NWS) Dam Break Flood Forecasting Model (DAMBRK), the US Army Corps Engineers Hydrologic Engineering Center Flood Hydrograph Package (HEC-1) and the NWS Simplified Dam Break Flood Forecasting Model (SMPDBK). Other software package developed by Danish Hydraulic Institute (DHI) called MIKE 11. DAMBRK is one of the widely used dam failure analysis models. In most of the models, flood hydrology does not constitute a part of the model.

3.1.1. NATIONAL WEATHER SERVICE (NWS) DAM BREACH FLOOD FORECASTING MODEL (DAMBRK)

It is a dynamic routing model. It contains breach simulation routines in which the breach begins at the top of dam and grows uniformly downward and outward. A breach of a fixed shape is initiated when the reservoir water surface reaches a given elevation and then breach dimensions grow linearly with time. The water surface elevation at which failure begins arid the breach formation time is to be provided as input data. A trapezoidal, rectangular or triangular shaped breach is specified by inputting the breach side slopes and terminal breach bottom width and elevation. It also has an additional option for simulating a piping failure. A rectangular breach grows outward from a point linearly over the time to failure. A breach erosion model for use in conjunction with DAMBRK has also been developed.

Discharges through spillway and outlet works structures and dam breaches are computed as a function of reservoir water surface elevation using empirical weir and orifice equations. This model also has provision to consider tail water submergence effects in the outflow hydrograph computations. Reservoir routing is accomplished by either hydraulic or dynamic routing methods. Hydraulic routing is based on an assumed level water surface, reservoir geometry is described by a storage versus elevation relationship and it is applicable for wide, flat reservoir with gradual change in water surface levels. Dynamic routing methods can handle the negative waves than may be cause by sudden reservoir draw downs and positive waves produced by large reservoir. Water surface profiles through an upstream of a reservoir can be developed as well as the outflow hydrograph. In dynamic routing, the reservoir geometry is described by cross sections and Manning roughness coefficient, as is the downstream valley. Dynamic routing methods are most advantageous for long, narrow reservoirs with rapid water level changes at the breached dam, and are more accurate when the slope of the reservoir water surface is significant. In many typical situations for which the basic assumptions are valid, hydrologic reservoir routing is generally easier to use than dynamic routing and is essentially as accurate.

DAMBRK Model uses one-dimensional dynamic method of flood routing, based on four point implicit solution of St. Venant equations for conservation of mass and conservation of momentum.

3.1.2. U.S. ARMY CORPS OF ENGINEERS HYDROLOGIC ENGINEERING CENTRE (HEC) FLOOD HYDROGRAPH PACKAGE (HEC1)

This model simulates the precipitation-runoff process and routes flood hydrographs using hydrologic methods. For dam breach simulation it has a routine similar to DAMBRK model, except that the breach does not expand laterally. Discharges through spillway, outlet works and dam breaches are computed as a function of reservoir water surface elevation. The model, however, does not reflect tail water submergence effect on the outflow computations. Flood routing is accomplished by hydrologic storage routing methods (***HEC-1***, *User Manual*).

3.1.3. U.S. ARMY CORPS OF ENGINEERS HYDROLOGIC ENGINEERING CENTRE (HEC-RAS) RIVER ANALYSIS SYSTEM PACKAGE (HEC-RAS)

HEC-RAS can be used to model both overtopping as well as piping failure breaches for earthen dams. Additionally, the more instantaneous type of failures of concrete dams (generally occurring from earthquakes) can also be modeled. The resulting flood wave is routed downstream using the unsteady flow equations. Inundation mapping of the resulting flood can be done with the HEC-GeoRAS program (companion product to HEC-RAS) when GIS data (terrain data) are available.

Dams are modeled within HEC-RAS by using the in line Structure editor. The in line Structure editor allows the user to put in an embankment, define overflow spillways and weirs, and gated openings (radial and sluice gates). Gated openings can be controlled with a time series of gate openings or using the elevation control gate operation feature in HEC-RAS. Several plots and tables are available for evaluating the results of a dam break analysis within HEC-RAS. Graphics include cross section, profile, and 3 dimensional plots, all of which can be animated on a time step by time step basis in order to visualize the propagation of the flood wave. (***HEC-RAS***, *User Manual*).

3.1.4. SIMPLIFIED DAM BREACH FLOOD FORECASTING MODEL (SMPDK)

This model assumes a time dependent rectangular breach. Discharges through spillway, outlet works and dam breaches are computed on same lines as DAMBRK model. It uses generalized Dynamic routing relationship for flood routing, which involves use of a family of dimensionless curves that have been developed using a dynamic routing model. The assumption of prismatic channel of specified shape is one of the major simplifications made to develop set of dimensionless curves.

3.1.5. DANISH HYDRAULIC INSTITUTE (DHI) MIKE 11

MIKE 11 consists of a number of modules, which in principle operate independently, hydrodynamic module, which is capable of simulating unsteady flow in a network of open channel system. The results of the simulation consist of time series of water levels and discharges. Transport dispersion and sediment transport calculations may be carried from special modules, which utilize the results of a hydrodynamic computation. The Dambreak Module can model the failure of one or more dams in a river system. It can simulate the initial dam failure using one of three modes of failure, which is overtopping mode . The dam breach expansion can be modeled either as a function of time or of sediment transport rate. The Dambreak Module accounts for breach flow, overtopping flow, and spillway flow (***MIKE 11,*** *User Manual*).

3.1.6. BREACH MODEL BY DR. D.L. FREAD

The BREACH model is physically based mathematical model developed by Dr. D.L Fread for predicting the earthen dam breach outflow hydrograph. The model based on the principles of hydraulics, sediment transport, soil mechanic, the geometric and mathematical properties of the dam, and the reservoir characteristic such as storage volume, spillway characteristics, and time dependent reservoir inflow rate. The model has seven mayor components : breach formation, breach width, reservoir water level, breach channel hydraulic, sediment transport, breach enlargement by sudden collapse and computational algorithm. This model is capable of simulating failure due to overtopping or piping, but the model does not consider the downstream flood routing.

The BREACH model emphasizes breaches morphology and flow through the breach and breach channel formation. This model expresses breach width as function of the depth of flow in the breach formation. This model also analyzes forces to compute collapsing of the breach sides assuming dry soil condition (***Singh,*** *1996*)

Table III-1 shows the parameters included in various dam breach models. The Cristofano model considers only breach evolution and breach flow, whereas the BEED model is very comprehensive. The HW and BRDAM models are worthwhile extensions of the Cristofano model, with inclusion of several other parameters. The PT, Lou and Nogueira models are quite similar in their construction, and include essentially the same parameters, except that the Nogueira model also considers inflow routing and side slope collapsing explicitly.

Table III-1 shows that the most of the parameters for dam breach erosion model are applied in MIKE 11 and DAMBRK models. A simplified version of this model for obtaining quick results is the SMPDBK model; thus, it includes the same components and has essentially the same rationale. The BREACH and BEED models emphasize breach morphology and flow through the breach and breach channel; however, the BEED model, like the DAMBRK model, also considers reservoir routing and downstream routing.

The two primary tasks in the hydraulic analysis of a dam breach are the prediction of the reservoir outflow hydrograph and the routing of that hydrograph through the downstream valley. Predicting the outflow hydrograph can be further subdivided into predicting the breach characteristics (e,g, shape, depth, width, rate of breach formation), and routing the reservoir storage and the inflow through the breach.

Most of the models do not directly simulate the breach. Rather, the user of the model independently, determines the ultimate breach parameters (i.e. dimensions of the fully, developed breach), and the time required for breach formation. These parameters are provided as input to the routing model, and the model then simulates the development of the breach in a progressive fashion, usually a linear increase in breach dimensions over the span of the breach formation time. There is presently little research to support or refute the assumption of linear breach development.

3.2. REVIEW OF MODEL APPLICATION

K.S. Kumar and Dr. S.M. Seth (1992)[6] have applied MIKE 11 software to simulation of dam break flood wave formation for Manchu II Dam failure. Its results compare with the earlier study results of failure of this dam carried out at N.I.H. using N.W.S DAMBRK programme by taking common data base into consideration. The comparison of results of two models revealed that there is a significant difference in the peak flood computation. The resulting peak discharge using MIKE 11 is less than that of N.W.S. DAMBRK application.

Another application of MIKE 11 is used to simulate the proposed Karcham Dam to carry out the preparing an emergency action plan to mitigate the flood disaster in the case of a failure of a dam. From the analysis of Karcham Dam Failure, it was observed that the effect of change in breach width is more significant than the breach development time. Both maximum discharge and maximum water level increase with the increase in breach width and the peaks are attained earlier. However, with the increase in breach development time both maximum discharge and maximum water level decrease and the peak is also delayed. But the effect is not significant in comparison to the change in breach width except in case of instantaneous failure of a dam.

DAMBRK model has been applied in case study of Tvaron dam, which is located at Stornorrfors in lower reaches of the Umealven river Sweden. This paper reports, the finding of a study aiming at evaluation of effect of river valley parameters uncertainty in a hypothetical dam break event. The resulting uncertainty in flood discharge or time to peak water stage increases farther downstream along the river. The range of uncertainly in water stage is dependent upon cross sectional geometry and channel width. Larger variation can be expected at narrower sections of a river. This study recommended that, for detailed dam break modelling and contingency evacuation planning, the effect of uncertainty in cross-sectional geometry and roughness should be examined carefully (***Xiao-Liang & Chen-Shang,*** *Royal Institute of Technology, Sweden, 1994*).

N.W.S. DAMBRK model also applied to simulate the dam break flood wave formation due to a hypothetical failure of Gandhi Sagar Dam in Madya Pradesh[7]. In this study a sensitivity analysis has been performed to study the effect of various dam failure parameters on the outflow flood wave due to dam failure. The following points can be notes from this study :

1. Due to the large drainage area above the dam and a large reservoir storage, the effect of change in time taken for breach formation on the dam break flood characteristics is insignificant.

2. Since the reservoir volume (storage capacity) is almost equal to the total volume of inflow hydrograph (PMF), there is no effect to slight changes in inflow hydrograph on the out flowing flood characteristic.

3. The flood wave characteristics are affected by the changes in reservoir water level at the time of failure and the size of breach. Flood stage and discharge values are increasing with the increase in magnitudes of reservoir water level and breach size.

3.3. BREACHING CHARACTERISTIC

The breach is an opening formed in the dam as it fails. Dams can fail either gradually or instantaneously. The type of failure depends on the cause of failure and the dam type. When a dam fails instantaneously, i.e. a large portion or the entire dam is removed within a short time, a sudden release of water generates a flood wave propagating in the valley downstream. Embankment dam breaches are typically assumed to be approximately rectangular, triangular and trapezoidal.

The actual failure mechanics are not well understood for either earthen or concrete dams. In previous attempts to predict downstream flooding due to dam failures, it was usually assumed that the dam failed completely and instantaneously. Investigators of dam-break flood waves such as Ritter (1892), Schocklitsch (1917), Re (1946), Dressler (1954), Stoker (1957), Su and Barnes (1969), and Sakkas and Strelkoff (1973) assumed that the breach encompasses the entire dam and that it occurs instantaneously. These assumptions are somewhat appropriate for concrete arch-type dams, but they are not appropriate for earthen and concrete gravity-type dams (*BOSS DAMBRK, 1988*).

Singh *(1996)*[29], based on 52 historical dam failures, has summarized the main characteristics of dam breaches. The breach shape can be approximated in all cases as trapezoidal, with ratio between the top and the bottom width as 1.29

(±0.18), and extreme values ranging from 1.06 to 1.74. The angle between the breach side slope and the vertical was in majority of cases between 40° and 50°.

MacDonald and Langiredge-Monopolis (1984)[29] analyzed breaching characteristics of a number of historical dam failures. They concluded that for both earthfill and non earthfill, the breach shape could be assumed to be triangular with 2V:1H side slopes, provided the breach did just develop to the base of the embankment, and trapezoidal with 2V:1H side slopes if additional material was wasted away after the breach reached the bottom of the embankment. This, of course, should be used only if the breach size is less than the embankment size.

Houston (1985)[29] concluded using data of MacDonald and Langridge-Monopilis (1984) that for earthfill embankments properly constructed of good fill, the breach shape could be assumed to be trapezoidal with 1 vertical to 1 horizontal side slopes with a base width equal to the depth of the breach, and to extend to the bottom of the erodible foundation under the dam. For old dam, unengineered dams or dams constructed of poor materials, longer breach size should be taken.

3.3.1. OVERTOPPING BREACH

Overtopping failures result from the erosive action of water on the embankment. Erosion is due to uncontrolled flow of water over, around, and adjacent to the dam. The earth embankments are not designed to be overtopped and therefore are particularly susceptible to erosion. Once erosion has begun during overtopping, it is almost impossible to stop. A well-vegetated earth embankment may withstand limited overtopping if its top is level and water flows over the top and down the face as an evenly distributed sheet without becoming concentrated.

When the breach stems from overtopping, excessive stress on the surface, induced by water flow, initiates the erosion process. Erosion will begin when local shear exceeds a critical value, after which the soil particles are set in motion. Any initial small breach represents a weak point where shear stress is higher than adjacent points and may quickly develop into the larger breach. The extent of breaching depends upon the duration of overtopping and properties and design of the dam itself. If this time is short, minimal erosion may occur. The initial breach is predominantly "V" shape, and between three to four time as wide as it is deep.

As the washout progresses, the softer material of the dam body is washed away and the longitudinal slope of breach is gradually flattened as the breach section grows. The extent of this lateral erosion depends upon the size of reservoir and the volume of stored water. The erosion will continue until a non-erodible layer is encountered, which may be the dam foundation, or until the erosive capacity of the flowing water is less than the dam's resistance to erosion (***Singh, 1996***).

An overtopping failure is one in which the breach is simulated as a rectangular, triangular, or trapezoidal shaped opening that grows progressively downward from the dam crest with time (see ***Figure III-2***). Flow through the breach at any instant is calculated using a broad crested weir equation.

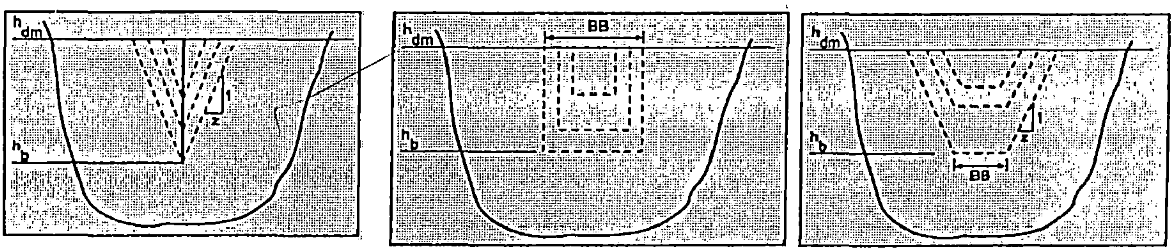

Figure III-2. Front view of dam showing three type of overtopping breach formation

3.3.2. PIPING BREACH

Every embankment dam has water passing through or under the embankment because all earth materials are porous. The passage of water through or under the embankment is known as seepage. Seepage quantities and rates increase as the depth of the water in the reservoir increases due to the greater pressure upstream of the embankment. Seepage must be controlled in both velocity and quantity. If uncontrolled, it can progressively erode soil from the embankment or its foundation, resulting in rapid failure of the dam.

Internal erosion and piping has historically resulted in about 0.5% (1 in 200) embankment dams failing, and 1.5% (1 in 60) experiencing a piping incident. Of these failures and accidents, about half are in the embankment, 40% in the foundations, and 10% from the embankment to foundation. Fewer incidents of piping in the foundation, and particularly from embankment to foundation, progress to failure, are repeated than for piping in the embankment. About two thirds of the failures occur on first filling or in the first 5 years of operation (***Robin Fell,*** *Journal of Geo-technical and Geo-environmental Engineering, ASCE, APRIL 2003*)

Erosion of the soil begins at the downstream side of the embankment, either in the dam proper or the foundation, progressively works toward the reservoir, and eventually develops a "pipe" or direct conduit to the reservoir. This phenomenon is known as "piping." Piping action can be recognized by an increased seepage flow rate, the discharge of muddy or discolored water, sinkholes on or near the embankment, and a whirlpool in the reservoir. Once a whirlpool (eddy) is observed on the reservoir surface, complete failure of the dam will probably follow in a matter of minutes (*Indiana Department of Natural Resources Division of Water, Dam Safety: Earth Dam Failures, Indianapolis, Indiana*). As with overtopping, fully developed piping is virtually impossible to control and will most likely cause failure.

Seepage can cause slope failure by creating high pressures in the soil pores or by saturating the slope. The pressure of seepage within an embankment is difficult to determine without proper instrumentation. A slope, which becomes saturated and develops slides, may be showing signs of excessive seepage pressure.

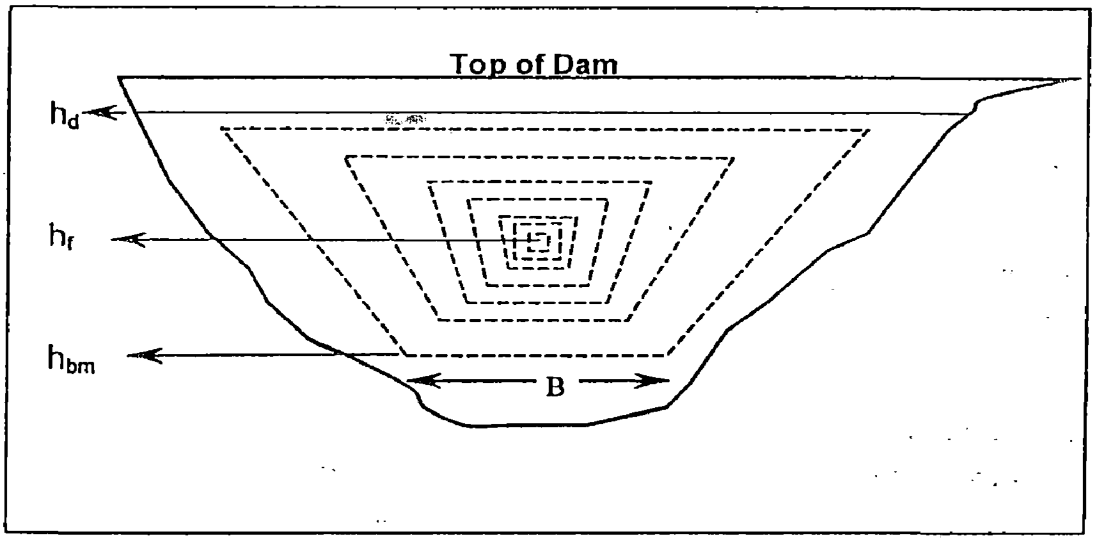

Figure III-3. Front view of dam showing formation of orifice breach

Figure III-3 presents the dam break due to piping process. Dam break due to piping can be simulated by determining axis elevation of the piping. This is simulated as orifice breach of rectangular shape. The water level (h_d) in the reservoir must be greater than the assumed centre line elevation (h_f) of the initial breach before the pipe starts to increase due to erosion.

3.4. BREACHING PARAMETER

To determine the dam failure hydrograph, four parameters are needed to define the breach formation through the dam. These parameters include the breach bottom width, side slopes, time of failure, and the selected failure and breach bottom elevations. **Table III-2** shows some suggestion of breaching parameter for earth dams, ***(Bruce W. Harrington,** 2003)*.

Table III-2. Suggested Breach Parameters For Earth Dams

Source	Average Breach Width (m)	Breach Side Slope	Breach Failure Time (hrs)
NWS (*1988*)	1H to 5H	Z = 0 to 1	0.1 to 2.0
COE (*1980*)	0.5H to 4H	Z = 0 to 1	0.5 to 4.0
FERC (*1991*)	1H to 5H	Z = 0 to 1	0.1 to 1.0
USBR (*1982*)	3 H	N / A	0.00333 b
NWS (*1988*)	1H to 4H	Z = 0 to 1	0.5 to 4.0
Harrington (*1999*)	1H to 8H	Z = 0 to 1	H/180 to H/120

Source : Bruce W.Harrington, 2003

Notes : b = final breach width
H = high of dam

3.4.1. TIME OF BREACH

There is a need to better understand and quantify the times associated with breach initiation and breach formation for earth embankment dams. Definitions for these times discussed by *The International Dam Breach Processes Workshop, Stillwater, Oklahoma* are as follow :

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

The second part of this definition describes breach initiation phase and the breach formation phase. For purpose of estimating available warning and evacuation time in the event of a dam failure, the breach initiation time is defined as follows :

Breach initiation time - It is the duration of time, beginning with the first observable flow over or through a dam that might initiate warning, evacuation, or heightened awareness, and ending with the start of the breach formation phase.

Variation of breach parameters can affect peak discharge and inundation levels, as well as warning and evacuation time. For small reservoirs (those which experience significant draw-down before the breach is fully developed), changes in breach formation time can dramatically affect peak outflow. For locations well downstream of a dam, timing of the flood wave peak can change significantly with changes in breach formation time, but peak discharge and inundation levels are insensitive to changes in breach parameters (***Tony L. Wahl**, 1997*). Warning time is defined as the sum of the breach initiation time, breach formation time, and flood wave travel time from the dam to a population center.

Gradual dam failures occur over a period of time. Documented earth dam disasters indicate gradual and progressive modes of failure. Singh (1996) and Data Base of Dam Failure release by American Society of Civil Engineers (***ASCE**, 2002*) have noted that duration of earth dam breaches can vary from 15 minutes to more then 5 hours. In most of dams above 30 meter height, duration time to failure vary from 3 to 14 hours. This clearly shows that most of earth dams do not fail instantaneously.

3.4.2. BREACH WIDTH

Variations of breach width can also produce large changes in peak outflow, especially for large reservoirs. Variations of breach height have a relatively small effect on peak outflow, (***K. Singh and Snorrason,*** *1984*).

Petrascheck and Sydler (1984)[5] demonstrated the sensitivity of peak flow, inundation levels, and flood arrival time to changes in breach width and breach formation time. For locations near a dam, breach width can have a significant influence.

3.4.3. BREACH SLOPES

The slope of breach is very important in computation of the out flow hydrograph. One of the factors affecting the breach slope is the way the material is initially eroded. Another factor affecting the slope is the type of the material in the fill. Different types of materials, mainly granular or cohesive, may develop different slopes. Thus, it is clear that one slope cannot be used to determine the nature of failure. It was assumed that the breach slope would be created by initial slippage of a section of fill in accordance with the friction angle of the material.

During breaching, whenever gravity and hydrodynamic forces due to seepage or water through the body of dam become greater than the soil friction and cohesion, the breach slopes become unstable and breach sides collapse. As a result, the breach section suddenly becomes enlarged, allowing, in turn, greater breach flow and greater breach erosion. This cycle may continue until appropriate stabilized conditions are met, or the reservoir is emptied (***Singh,*** *1996*).

The breach slope can vary during lateral and vertical erosion of the breach-channel. During the early stage of the *overtopping* of the embankment, the initial channel may have a very steep slope. The first surge of water over the dam would flow down the dam face at the fill angle. With the progression of erosion the angle may significantly decrease, remain about the same, or even increase. If *piping* is the cause of slippage, the initial channel slope may be large or small and may subsequently increase, decrease or remain about the same.

A dam may be built of a wide range of soils. One type of the dam may have granular materials on the faces and an impervious core. When erosion progresses vertically, it begins in the granular materials and increasingly intersects the core. This shows that different types of materials, more than one slope of the breach channel and that the slope of the upstream granular materials will be different from the slope through the impervious material. Because the tractive force required to pick up a clay particle is smaller than that to pick up a granular particle (of larger dimensions), a gouging of the core material may develop. This would imply that the final slope through the downstream face of fill would become less because of the inability of the water to pick up additional materials.

3.5. FLOOD ROUTING

Flood routing is defined, as the procedure whereby the time and magnitude of a flood wave at a point on a stream is determined from the known or assumed data at one or more points upstream. Flood routing is considered under two broad but somewhat related types, namely *reservoir routing* and *open channel routing* (**Chow,** *1964*). The movement of a flood wave down a river and through a reservoir and the associate change in timing or attenuation of the wave constitutes an important topic in dam break study. It is essential to predict the temporal and spatial variation of a flood wave through a river reach or reservoir.

Reservoir routing is part of dam break analysis. In case of reservoir routing one most common method is storage routing. The storage routing through reservoir will generally attenuate the peak outflow and lag time to peak for the outflow hydrograph. The rate of change of storage can be written as the continuity equation. One important aspect in reservoir routing is that the peak of the outflow from reservoir should intersect the inflow hydrograph to the reservoir.

Open channel routings are used to determine the time and magnitude of flood waves in rivers. River routing differ from reservoir routing in that the storage in a river reach depends on more than just outflow. The peak flow from a reach is usually attenuated and delayed compared to that of the inflow hydrograph. Because storage in a river reach is function of whether stage is rising or falling, it is a function of both outflow and inflow for the routing reach. Also, as river stages rise high enough to inundate a floodplain beyond the bank of river, significant velocity reductions are observed in the floodplain compared with the main river. ***Figure III-4*** illustrates some differences between river and reservoir routing.

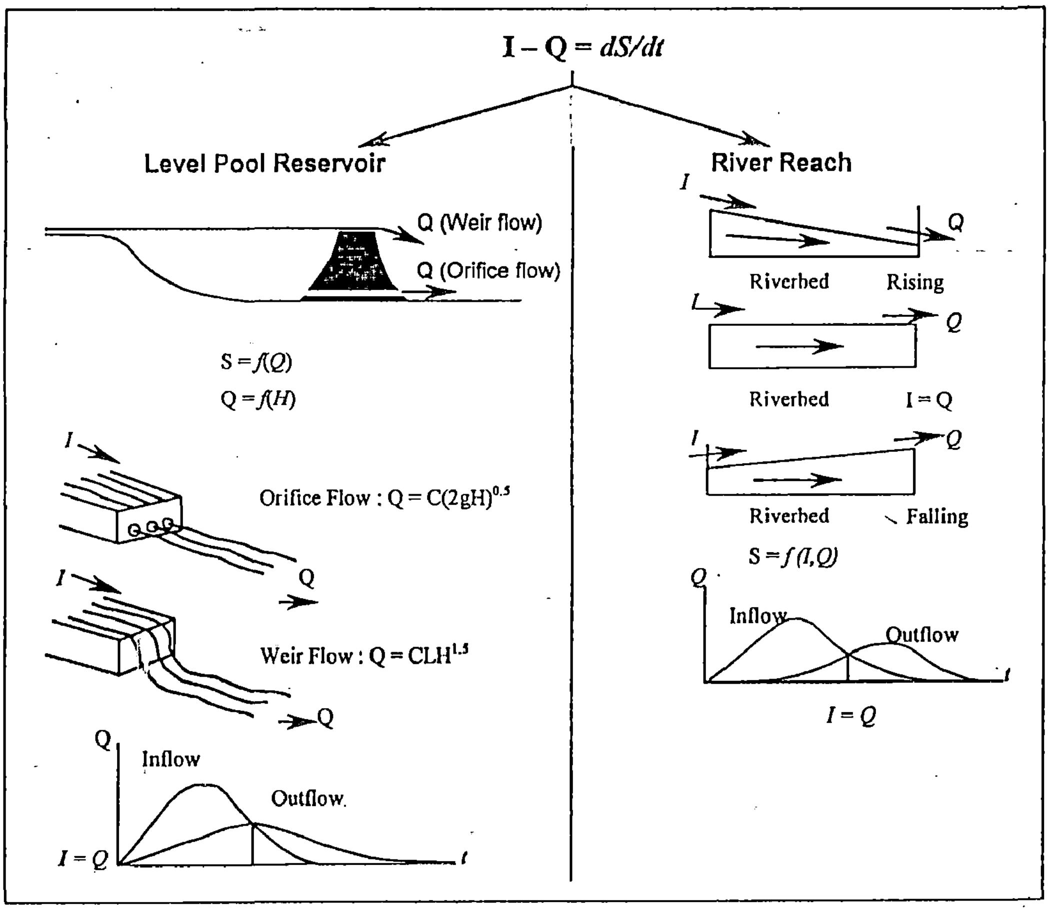

Figure III-4. Some differences between reservoir and river routing

The gush of water in a channel due to the sudden opening of a gate or the failure of a dam can be analyzed using some routing techniques. Routing techniques may be classified in two mayor categories : Simple hydrologic routing and more complex hydraulic routing (***Philip Bedient,*** *2002*).

3.5.1. HYDROLOGIC ROUTING METHODS

Hydrologic routing methods involve the balancing of inflow, outflow and volume storage through use of the continuity equation. A Second relationship, the storage-discharge relation, is also required between outflow rate and storage in the system.

The Applications of hydrologic routing in dam break study is reservoir flood routing due to dam failure.

3.5.2. HYDRAULIC ROUTING METHODS

Hydraulic routing is more complex and accurate than hydrologic routing and is based on solution of continuity equation and the momentum equation for unsteady flow in open channels. These differential equations known as the Saint Venant Equations, the first derived in 1871 are usually solved by explicit or implicit numerical methods. Explicit methods calculate values of velocity and depth over a grid system based on previously known data for river reach. Implicit methods set up a series of simultaneous numerical equation over a grid system for the entire river, and equation is solved at each time step.

3.6. HAZARD & RISK ASSESMENT

Safety is a basic need. But its expression, however, is vague and is not always made in consistent terms. Safety is related to hazard and risk to damage. Hazards are natural as well as man-made. Earthquakes, volcanic eruptions, landslides, floods, hurricanes, etc. represent natural hazards. Man-made hazards include damage structures like dams and reservoirs, tunnels, bridges, buildings, etc. When a dam fails, blame goes to somebody-either the designer, the builder, the operator, or the maintenance man.

A dam represents a potential threat to those living downstream. The scale of this threat can be considered in terms of (a) hazard and (b) risk. Hazard is represented by the consequences of failure through possible loss of life and damage. Risk is represented by the probability that failure may occur at some time from whatever cause during the life span of the dam. The risk is in fact the product of potential damage times the probability of failure. Both these factors are, however, complex and are difficult to precisely define. The concept of safety is expressed in terms of a "factor of safety", or probability of failure. The hazard classification of the dam is usually based on the increased flood depths and velocities against or over downstream structures. These structures include houses, buildings, important utilities, roads and railroads.

The problem of dam safety is characterized by a high degree of complexity. Consequently, the related failure models necessarily contain high level of uncertainty and incompleteness due to :

(a) The difficulty of determining probabilities of extreme events,

(b) Lack of an exhaustive set of scenarios,

(c) The socio -technical nature of some components of the dam project,

(d) The impossibility of testing models, and

(e) Peculiarity of each dam.

Dam safety should be seen as a process of continuing quality management from design through construction to operation. Computed safety factors or failure models should be used in this quality management. The quality-testing process should be continued throughout the life of the dam. A process such as these results in an increased set of data derived from the structure itself. These together with the results of analyses of modeled scenarios should be used to assess dam safety. Dependability indices can be defined to express the results of the quality testing process.

Classification of reservoir hazard in term of the scale of human and economic loss to be anticipated in event of catastrophic dam failure is increasingly common. At its simplest, hazard level is assessed and rates on descriptive categorization is as below, (***Bruce W. Harrington,*** *2003*)

- *Low scale,* unlikely loss of life; minor increases to existing flood levels at roads and buildings
- *Significant scale*, possible loss of life, significantly increased flood risks to roads and buildings with no more than 2 houses or 6 lives in jeopardy
- *High scale*, probable loss of life; major increase to existing flood levels at houses, buildings, major interstates and state roads with more than 6 lives in jeopardy

An element of discretion may then be applied in defining the surveillance regime to be applied to specific dam. The limitations implicit in such subjective appraisal and classification are evident.

3.7. SUMMARY OF REVIEW OF LITERATURE

The predominant mechanism of breaching of earthfill dam is erosion of the embankment material by the flow water either over or through the dam. Based on review of literatures, the prominent breach parameters which affect the magnitude and duration of outflow discharge are summarized as follow :

- Time to failure studies show, that most of earth dam failures are not instantaneous. Historical data show that recorded time to failure is half an hour to twelve hour. Particularly for dams above 30 meters height the failure time is between 4 to 12 hours.

- Shape of breach formation. Most of dam failures have shape of breach formations approximately trapezoidal. A variable relevant with trapezoidal formation is the angle between breach slide slope and the vertical. The breach slide slope (Z) varies from 1 to 4.

- Final breach formation. There are two parameters of final breach formation. First is final elevation of breach formation and secondly final breach width. It is difficult to predict the exact location of the initial breach. There is not adequate information about the final breach width, but the historical data show that in most of the cases the final breach elevation is nearly bottom of dam.

- Another important parameter which affect the magnitude and duration of outflow discharge through the breach formation is flood inflow hydrograph. More high the return period of Flood inflow hydrograp of high return period would generate high outflow discharge.

Table 3.1. Components of Various Mathematical Models for Dam Breach Erosion

Model	Author (Year)	Reservoir Hydraulics		Hydraulic of Flow Over The Dam			Breach Morphology			Flow Routing	
		Inflow Routing	Reservoir Water Elv.	Flow Over The Crest	Flow Through The Breach	Flow Through The Breach On The D/S Face	Sediment Transport	Breach Shape	Side Slopes Collapsing	Channel Routing	Floodplain Routing
Cristofano Model	Cristofano 1965	No	No	No	Yes	No	Yes	Yes	No	No	No
HW Model	Harris and Wagner (1977)	No	Yes	No	Yes	No	Yes	Yes	No	No	No
BRDAM Model	Brown and Rogers (1977)	Yes	Yes	No	Yes	No	Yes	Yes	No	Yes	No
PT Model	Ponce and Tsivoglu (1981)	No	Yes	No	Yes	Yes	Yes	Yes	No	No	No
Lou Model	Lou (1981)	No	Yes	No	Yes	Yes	Yes	Yes	No	No	No
Nouguera Model	Nouguera (1984)	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	No	No
DAMBRK Model	Fread (1984)	Yes	Yes	Yes	Yes	No	No	Yes	No	Yes	Yes
SMPDBK Model	Welmore and Fread (1984)	No	No	Yes	Yes	No	No	Yes	No	Yes	No
Breach Model	Fread (1984)	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No
BEED Model	Singh and Scarlatos (1986)	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
MIKE 11	DHI (1986)	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Source : Singh, 1996

CHAPTER IV
METHODOLOGY

4.1. GENERAL

Catastrophic flash flooding occurs when a dam is breached and the impounded water escapes through the breach into the downstream valley. Usually the response time available for warning is much shorter than for precipitation-runoff floods. Overtopping of the dam due to inadequate spillway capacity during large inflows to the reservoir from heavy precipitation runoff often causes Dam failures. Dam failures may also be caused by seepage or piping through the dam or along internal conduits, slope embankment slides, earthquake damage and liquefaction of earthen dams from earthquakes, and landslide-generated waves within the reservoir.

The potential for catastrophic flooding due to dam failures could result in loss of human life and appreciable property damage. The Government has the responsibility to advise the public of downstream flooding when there is a failure of a dam. Although this type of flood has many similarities to floods produced by precipitation runoff, the dam-break flood has some very important differences, which make it difficult to analyze with the common techniques, which have worked so well for the precipitation-runoff floods.

In this study DAMBRK simulates the failure of a dam, computes the resultant outflow hydrograph and simulates movement of the dam-break flood wave through the downstream river valley. The results of these computations can be used to develop potential inundation maps for either hypothetical and historical failures, establish times of travel of various portions of the flood wave to downstream locations, and evaluate the effects of uncertainties in the dam failure parameters on these quantities.

The reservoir inflow hydrograph in this study is adopted from *Way Sekampung Irrigation Project* (Final Report of Hydrology Study). The hydrograph inflow to the reservoir has been carried out based on Probable Maximum Precipitation (PMP) and accordingly PMF, hydrograph is produced, it can be evaluated whether the flood is due to *overtopping* on the dam crest or otherwise. In addition, valley cross section on the down stream of dam site and the structures along river stream shall be taken into account and to be prepared beforehand. After the preparation of above mentioned, dam break analysis work can be executed.

Study of Batutegi Dambreak Analysis has been made using Dambrk Model release by BOSS DAMBRK, which is prepared based on has been made issued by Dr. Fread from U.S. National Weather Service, the United States, whose result can present 30 graphics for presentation, including *"conditions plot"*, preparation of flood map area plot, 3 dimensions of river valley, flood hydrograph on the respective cross section, flood wave arrival to reach the respective cross section.

4.2. PREVIOUS OF DAMBRK MODEL

DAMBRK is an expanded version of a practical operational model first presented by Fread (1977). That model was based on previous work by the same author on modeling breached dams (Fread and Harbaugh, 1973) and routing of flood waves (Fread, 1974, 1976) (***BOSS DAMBRK***, User Manual).

The model consists of three functional parts, namely:

1. Description of the dam failure mode, i.e, the temporal and geometrical description of the breach.
2. Computation of the time history (hydrograph) of the outflow through the breach as affected by the breach description, reservoir inflow, reservoir storage characteristics, spillway outflows, and downstream water elevations.
3. Routing of the outflow hydrograph through the downstream valley in order to determine the changes in the hydrograph due to valley storage, frictional resistance, downstream bridges or dams, and to determine the resulting water surface elevations (stages) and flood-wave travel times.

To illustrate an example for reservoir routing, the inflow hydrograph is routed through a reservoir using either storage or dynamic (complete unsteady flow equations) routing. Outflow at the dam at any instant is computed by summing the discharge over the spillway, over the top of the dam, through the breach, through a gated outlet, and through turbines. Each individual discharge is calculated from supplied discharge coefficients and the instantaneous head on each outflow point. A spillway or gated outlet rating curve can be supplied, if desired. Also, a time-dependent gate coefficient curve can be specified. An overtopping or piping failure can also be simulated if desired. The hydrograph is then routed through the downstream river valley using the complete unsteady flow equations.

4.2.1. PROGRAM CAPACITY

This DAMBRK program can simulate dam failures, compute outflow hydrograph, and simulate flood wave movement due to dam break (dam break flood) through valley on the dam downstream. Computation results using this DAMBRK program can be used for potential flood map, determination of travel time of a member part of flood wave to downstream location and evaluate effects of uncertainties of dam break parameter.

Other DAMBRK program capabilities are :

- *Capable of handling a series of broken dams on a single river on one computer process.*
- Capable of simulating bridge effect and its embankment.
- Capable of simulating river channel effect, meandering on wide flood plain.
- Capable of simulating sub-critical and supercritical flow on the same routing.
- Capable of routing certain hydrograph using dynamic routing.
- Capable of simulating the backwater effect of dam break that spreads throughout its tributaries and the main river.

DAMBRK model attempts to present methods in understanding dam break and the use of hydrodynamic theory to forecast dam break wave formation and its movement to downstream direction.

4.2.2. PROGRAM LIMITATIONS

The river routing procedure used in DAMBRK is based on solving the one dimensional equations of motion for unsteady flow in an open channel. This implies that cross sections are oriented perpendicular to the flow so that the water surface is horizontal across the cross section. Situations that involve large off-channel storage areas that take a long time to fill and empty may not be accurately simulated.

The assumption is made in DAMBRK that the channel boundaries are rigid, i.e., cross sections do not change shape due to scour or deposition. Therefore, situations involving large changes in channel configuration or location as a result of the dam break flood can not be accommodated. Furthermore, changes in channel roughness caused by passage of the flood wave can not be simulated, because Manning's n cannot be made a function of time.

The reservoir pool elevation at which breaching begins, rate of development, shape, and final size of the breach must be specified. As only gross estimates of these parameters can be made, sensitivity of the computed results to these parameters should be evaluated. Some suggested typical ranges for breach parameters are given in Table IV-1

Table IV-1. Suggested Breach Parameters

Parameters	Fill Dam	Concrete Dam	Curved Dam
Breach Width	1/2 to 4 x dam height	Several time monolith width	Dam total width
Breach Side Slope	0 to 1	0,1 to 0,5	Valley wall slope
Time to Failure (hrs)	0,5 to 4	0,1 to 0,5	Closing to sudden (0,1 hr)
Reservoir water level during the break	1 to 5 ft on the dam crest	10 to 50 ft on the dam crest	10 to 50 ft on the dam crest

Source : BOSS DAMBRK User Manual

Other Dambrk limitations are :

- Dam break on a dendritic river system (where dams are not arranged in a series but in a tree system under river system), cannot be simulated.

- In general, river channel on dam downstream cannot be dried during initial simulation, in the sense that there shall be basic flow (whatever small).
- Change from sub-critical to supercritical flow on time or distance basis cannot be calculated. This condition can cause non-convergence on its solution. Correction measures shall be taken in such condition.

4.3. BASIC THEORY OF DAM BREAK ANALYSIS

DAMBRK is used to prepare dam outflow hydrograph and routing of flood that occurs hydraulically along the downstream valley. The formula used in the said model is St. Venant formula of complete and single dimension for unsteady flow is correlated with internal boundary equations as rapidly varied flow through any structure such as dam and bridge/embankment that can develop into breach, which is time dependent. The External boundary equation is also used on upstream and downstream end of routing reach. The flow can be in the form of sub-critical or supercritical or combination of both. The liquid characteristic of the flow can follow Newtonian or non-Newtonian flow.

Hydrograph is determined as inputs in the form of time series. The possibility of a dam on the downstream that can be breached by flood, flow narrowness due to bridge/embankment, inflow from tributaries, river bents, dikes along the downstream river and the effect of tides on the river, is soundly studied during the travel of flood wave to downstream.

4.3.1. BREACHING CHARACTERISTIC

The breach is the opening formed in the dam as it fails. The actual failure mechanics are not well understood for either earthen or concrete dams. In previous attempts to predict downstream flooding due to dam failures, it was usually assumed that the dam failed completely and instantaneously.

Investigators of dam-break flood waves such as Ritter (1892), Schocklitsch (1917), Re (1946), Dressler (1954), Stoker (1957), Su and Barnes (1969), and Sakkas and Strelkoff (1973) assumed that the breach encompasses the entire dam and that it occurs instantaneously. Others, such as Schocklitsch (1917) and the Army Corps of Engineers (1960), have recognized the need to assume partial rather than complete breaches; however, they assumed that the breach occurred instantaneously. The assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam-break flood waves. These assumptions are somewhat appropriate for concrete arch-type dams, but they are not appropriate for earthen dams and concrete gravity-type dams.

For reasons of simplicity, generality, wide applicability, and the uncertainty in the actual failure mechanism, the DAMBRK model allows the forecaster to input the failure time interval (t) and the terminal size and shape of the breach (Fread and Harbaugh, 1973)[9].

There are two types of breaches, called :

- Breach due to overtopping
- Breach due to piping

a. Breach Due to Overtopping

Breach due to overtopping is simulated in the form of rectangular, triangular and trapezium breach. The breach grows bigger and bigger progressively with time, from dam crest downward to reach dam foundation. The flow passing the breach is computed as flow passing broad crested weir.

Form of terminal breach is determined by parameter (Z) that identifies side slope of breach, namely vertical slope: Z horizontal, parameter (b), which is termed as terminal width from breach bed. The range of parameter value of side slope Z is : $0 < Z < 2$ (suggested). This value is dependent on natural slope of material that is compacted and wetted. . Rectangular, triangular or trapezoidal are determined by the use of value combination Z and b.

The final breach width (b) is related to the average width of the breach (b_{bar}), the breach depth (h_d), and breach side slope (Z) such that:

$$b = b_{bar} - 0{,}5\, Z h_d \quad \text{.......................................} (1)$$

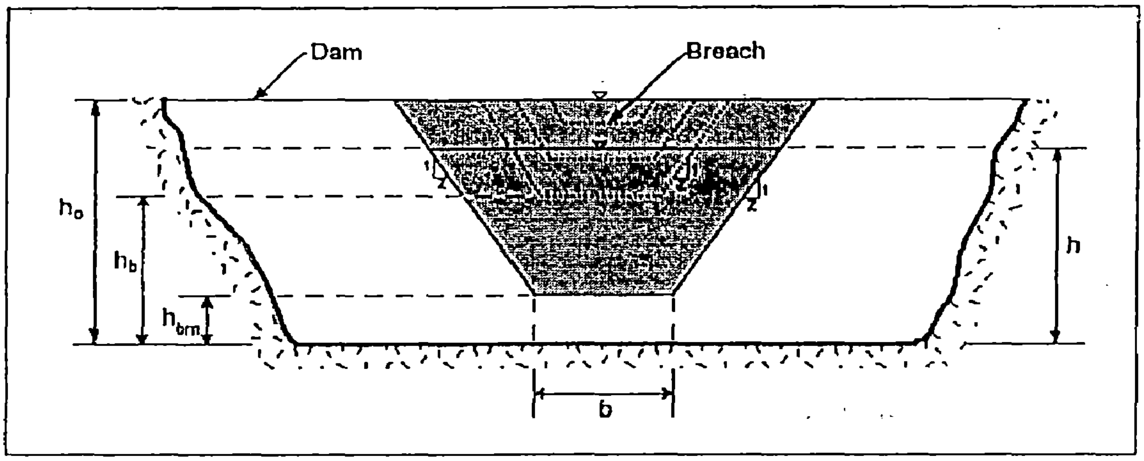

Figure IV-1. *Front View of Dam Showing Formation of Overtopping Breach*

As shown in Figure IV.1, the model assumes the breach bottom width starts at a point and enlarges at a linear or nonlinear rate over the failure time interval (t) until the terminal width (b) is attained and the breach bottom has eroded to the elevation h_{bm} which is usually, but not necessarily, the final bottom elevation of the reservoir or outlet channel. If t is less than 1 minute, the width of the breach bottom starts at a value of b rather than at zero. This represents more of a collapse failure than an erosion failure.

The bottom elevation of the breach is simulated as a function of time (t) according to the following relationship:

$$h_b = h_d - (h_d - h_{bm})(t_b / \tau)^\rho \quad \text{for } 0 < t_b \leq \tau \quad (2)$$

where :

h_b = bottom elevation of breach bed

h_d = water level at reservoir

h_{bm} = final elevation of breach bed

t_b = time interval since breach started forming

ρ = degree that indicates non linearity of rate between 1 to 4

= 1 for linear breach velocity

= 2 for quadratic non linear breach velocity

commonly used value is 1

Instantaneous basic width (b) of breach is presented as below :

$$b_i = b\,(t_b / \tau)^{\rho} \quad \text{for } 0 < t_b \leq \tau \qquad (3)$$

where :

b_i = instantaneous breach bottom width

During the simulation of dam break, the real breach formation is initiated if water level (h) is exceeding value h. This condition makes it possible for overtopping simulation of dam where the breach is not formed until considerable flow passes over the dam crest.

b. Breach Due to Piping

Piping failures occur when initial breach formation takes place at some point below the top of the dam due to erosion of an internal channel through the dam by escaping water. Times of failure are usually considerably longer for piping failures than for overtopping failures. This is because the upstream face is slowly being eroded in the very early phase of the piping development. As the erosion proceeds, a larger and larger opening is formed; this is eventually hastened by caving-in of the top portion of the dam. Poorly constructed earthen dams and coal-waste slag piles (dams) which impound water tend to fail within a few minutes, and have average breach widths in the upper range of the earthen dams mentioned above.

Dam break due to piping can be simulated by determining axis elevation of the piping. This is simulated as orifice breach of rectangular shape. The water level (h_d) in the reservoir must be greater than the assumed centre line elevation (h_f) of the initial breach before the pipe starts to increase due to erosion.

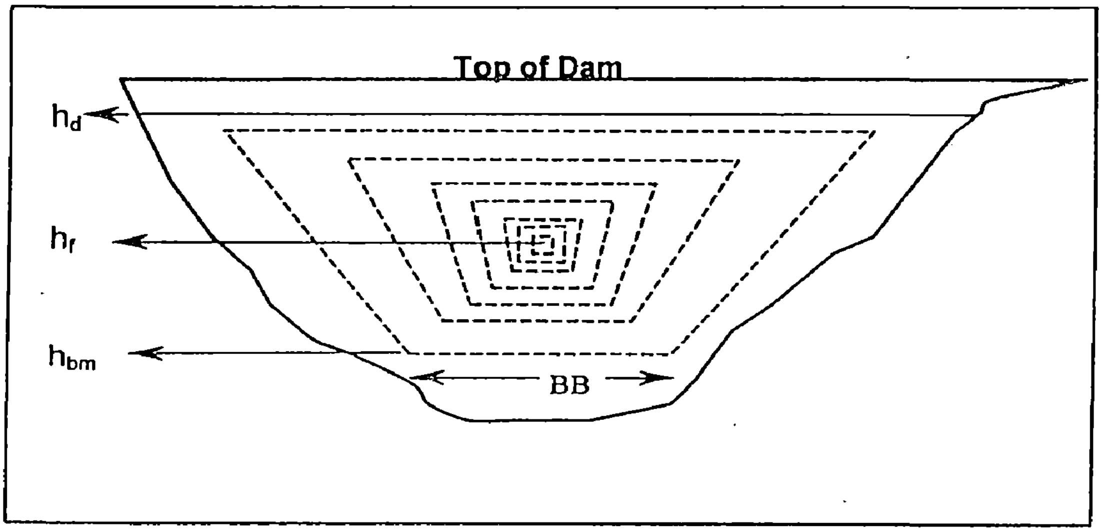

Figure IV.2. *Front View of Dam Showing Formation of Orifice/Piping Breach*

Breach due to piping is simulated as rectangular breach orifice developing upward and downward on dam body. The flow discharge passing through the breach is calculated using orifice formula or as wide crest, depending on reservoir water level and orifice top.

The outflow discharge from both breach type is a flood hydrograph that occurs on cross section 0 (beginning/dam site), that shall be routed to downstream direction along river valley by unsteady flow method.

4.3.2. EXAMINING OF BREACH PARAMETER

The other method to examine the accurate breach parameter (b_{bar} and τ) is using the following formula :

$$Q_p^* = 370\ (V_r h_d)^{0,5} \quad \text{.......... (4)}$$

$$Q_p = 3,1\ b_{bar} [C / \{\tau + C/(_{hd})^{0,5}\}]^3 \quad \text{.......... (5)}$$

$$C = 23,4\ As / b_{bar} \quad \text{.......... (6)}$$

Where :

Q_p^* = peak discharge to be expected passing the breach (*Formula developed by Hagen, 1962*)

Q_p = peak discharge to be expected passing the breach (*Formula developed by Fread, 1981*)

Vr = reservoir volume

As = reservoir surface area upon dam crest

Equation (4) is developed by Hagen (1962) from historical data of 14 dam break and produced maximum environment of the overall 14 discharge to be studied. Equations (5) and (6) are prepared by Fread (1981) and applied by National Weather Service on Simplified Dam Break Model, SMP DBK (Wetmore dan Fread, 1984).

After selecting b_{bar} and t, equation 5 can be used to compute Q_p which then can be compared with Q_p^* from equation 4. Thus, if $Q_p >> Q_p^*$, then either b_{bar} is too large and/or t is too small. However, if $Q_p << Q_p^*$ then either b_{bar} is too small and/or t is too large.

Fread has found that equation (4) over-estimated the peak discharges for each of 21 dam failures (including the previously mentioned 14 failures) by an average of 130 percent. Equation (5), although not used in DAMBRK, has been found to yield peak discharges within 5 to 10 percent of those produced in DAMBRK when equivalent values of b_{bar} and t are utilized in Equation (5) and in Equations (2) and (3) within DAMBRK.

4.3.3. RESERVOIR OUTFLOW HYDROGRAPH

The reservoir outflow consists of breach flow (Q_b) and flow through spillways (Q_s) if any. The breach flow is computed using a broad-crested weir flow relation, taking into account the effect of velocity of approach, and the effect of submergence at the downstream face on weir outflow.

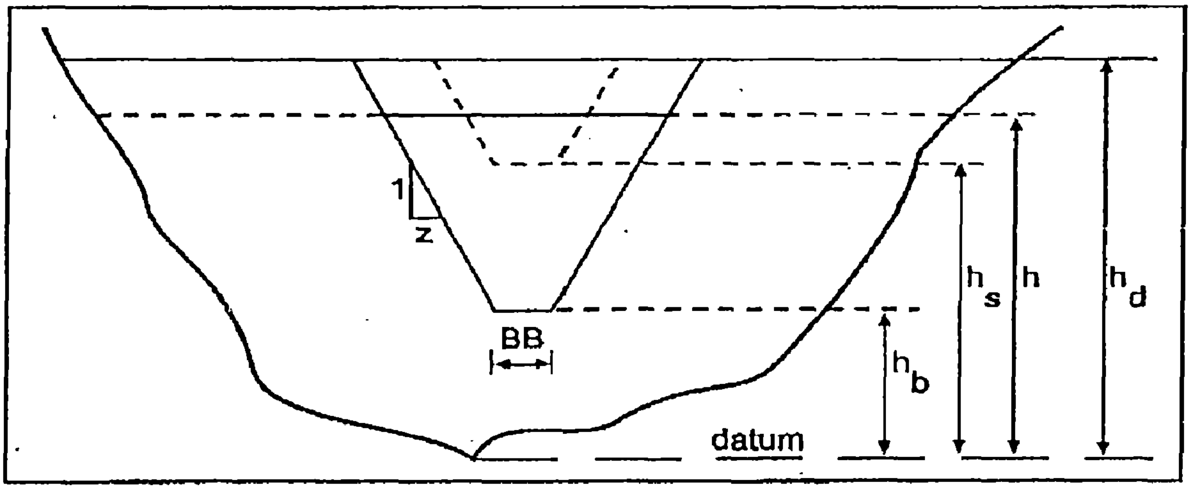

Figure IV-3. Location of equation variables for breach outflow.

$$Q = Q_b + Q_s \quad (7)$$

$$Q_b = K_s C_v \left[3.1 BB \frac{t_b}{\tau} (h - h_b)^{1.5} + 2.45 Z (h - h_b)^{2.5} \right] \quad (8)$$

For spillways, outflows are computed separately for gated spillway, ungated spillways and the water flowing from other structure.

$$Q_s = K_{ss} C_s L_s (h - h_s)^{1.5} + C_g A_g \sqrt{(h - h_b)} + C_d L_d (h - h_d)^{1.5} + Q_t \quad (9)$$

where:

K_s = 1.0 If $r \le 0.67$

K_s = $1.0 - 27.8\ (r_s - 0.67)^3$ If $r > 0.67$

r = $(h-h_t)/(h-h_b)$

K_{ss} = 1.0 If $r_s \le 0.67$

K_{ss} = $1.0 - 27.8(rs - 0.67)^3$ If $r_s > 0.67$

r_s = $(h-h_t)/(h-h_s)$

C_v = $1.0 + 0.023(V_2/(h-h_b)$

t_b = Time since breach began forming

Kss = a submergence correction for tailwater effects

Cs = the uncontrolled spillway discharge coefficient

hs = the uncontrolled spillway crest elevation

Cg = the fixed-gated spillway discharge coefficient

h_b = the center-line elevation of the gated spillway, or it is the tailwater elevation if the later is greater

k_d = a submergence correction for tailwater effects

C_d = the discharge coefficient for flow over the crest of the dam

L_s = the spillway length

A_g = the gate flow area

L_d = the length of the dam crest less L_s and the length of the gates located along the dam crest

(L$_d$ may also vary with h according to a specified table of L_d versus h; this allows for crest of dam, which are not level)

Q_t = a constant (or variable with time) outflow term which is head independent.

The reservoir outflow depends on the reservoir storage and water level. The inflow coming to the reservoir increases the storage. In order to simultaneously account for the effect of inflow and outflow, reservoir routing is performed using volume balance.

$$I - Q = \frac{dS}{dt} \quad \text{.......... (10)}$$

Where:

I = the reservoir inflow

Q = the total reservoir outflow

dS/dt = the time rate of change of reservoir storage volume.

Equation 10 may be expressed in finite difference form as follows:

$$\frac{I+I'}{2} - \frac{Q+Q'}{2} = \frac{\Delta S}{\Delta t} \quad \text{.......... (11)}$$

in which the prime (') superscript denotes values at the time t-Δt and the Δ approximates the differential. The term ΔS may be expressed as follows:

$$\Delta S = \frac{A_s + A_s'}{2} \times (h - h') \quad \text{.......... (12a)}$$

$$(A_s + A_s')\frac{(h-h')}{\Delta t} + Q + Q' - I - I' = 0 \quad \text{.......... (12b)}$$

Where:

A_s	=	Surface area
A_s'	=	Surface area at t-Δt
h	=	Water surface elevation
h'	=	Water surface elevation at t-Δt
Δt	=	Time step
Q	=	Total instantaneous outflow
Q'	=	Outflow at t-Δt
I	=	Inflow
I'	=	Inflow at t-Δt

Equations (7), (8), (9) and (10) are employed to rout the flow. It is assumed in this method of routing that the water surface elevation within the reservoir is horizontal, which is satisfactory for gradually occurring breaches with no substantial reservoir inflow.

4.3.4. HYDRAULIC COMPUTATION ALGORITHM

For flood wave routing due to dam break, dynamical wave method is applied. This method is based on single dimension non-permanent flow that is used to route flood hydrograph due to dambreak. The equations of St. Venant, expressed in conservation form (Fread, 1974b), with additional terms for the effect of expansion/contractions (Fread, 1976), channel sinuosity (DeLong, 1986) and non-Newtonian flow (Fread, 1987b) consist of conservation of mass equation, i.e.[5],

$$\frac{\partial Q}{\partial x} + \frac{\partial s}{\partial t}(A + A_0) - q = 0 \quad \text{.......} \quad (13)$$

and a conservation of momentum equation, i.e.,

$$\frac{\partial}{\partial t}(sQ) + \frac{\partial}{\partial x}(\beta Q^2) + gA\left(\frac{\partial h}{\partial x} + S_f + S_e + S_i\right) + L' = 0 \quad \text{.......} \quad (14)$$

Where :

h = water level

A = active on canal cross section

Ao = non- active off canal cross section

s = various sinuosity factor with h

x = longitudinal distance following the valley

t = time

q = inflow or lateral outflow per longitudinal distance following the valley (inflow, positive and outflow, negative)

β = moment coefficient for velocity distribution

g = gravitation acceleration

S_f = boundary friction slope

S_e = widening/narrowing slope

In equation (14) L' is moment effect of lateral flow, inflow or outflow discharge perpendicularly against main flow direction :

1. Lateral inflow :

   L' = 0

2. Outflow of lateral seepage :

   L' = –0,5 q Q/A

3. Bulk lateral outflow

   L' = –q Q/A

Boundary friction slope (S_f) in equation (14) is computed from Manning equation for uniform flow and by formula :

$$S_f = (n^2 |Q| Q) / (2.21A^2R^4/3) = |Q| Q/K^2 \quad \text{.. (15)}$$

Where :

n = manning coefficient for friction phase

R = hydraulic radius

K = conveyance factor

Conveyance factor K is calculated as follows :

$$K_L = (1.49 / n_l) A_l R_l\text{^}(2/3) \quad (16)$$
$$K_C = (1{,}49\, A_c R_c^{\,2/3})\,(n_c^{(2/3)} \quad (17)$$
$$K_R = (1{,}49 / n_r)\, A_r R_r^{\,2/3} \quad (18)$$
$$K = K_l + K_c + K_r \quad (19)$$

Where index K_L, K_C and K_R indicates the conveyance factor for flood plain of channel left, flood plain of channel right and flood plain of centre. Sinuosity factor s in equation (14) is weighted ratio of distance along the flood plain, which varies according to channel depth as correlated :

$$\beta = \frac{1.06\,(K_l^2/A_l + K_c^2/A_c + K_r^2/A_r)}{(K_l+K_c+K_r)^2 \,/\, (A_l+A_c+A_r)} \quad (20)$$

Where :

β = 1.06 if flood plain characteristic is not specified and its total profile is treated as composite profile.

Fraction S_e in equation (14) is determined as follows :

$$S_e = (k_{ce}\, \Delta\, (Q/A)^2 / (2g\ \Delta\, x)) \quad (21)$$

Where :

k_{ce} = coefficient of shrinkage development that moves from 0 ± 1 (+ if shrinkage and - if develops)

$\Delta\,(Q/A)^2$ = fraction ratio $(Q/A)^2$ on two adjacent with distance x.

4.3.5. TECHNIQUE OF ST. VENANT FORMULA SOLUTION

The expanded St. Venant Equations (13) and (14) constitute a system of partial differential equations with two independent variables, x and t, and two dependent variables, h and Q. The remaining terms in the equations are either functions of x, t, h, and/or Q, or these are constants. These equations are not amenable to analytical solutions except in cases where the channel geometry and boundary conditions are uncomplicated and the nonlinear properties of the equations are either neglected or made linear.

Equations (13) and (14) may be solved numerically by performing two basic steps. First, the partial differential equations are represented by a corresponding set of finite-difference algebraic equations. Second, the system of algebraic equations is solved in conformance with prescribed initial and boundary conditions.

Equations (13) and (14) can be solved by either explicit or implicit finite-difference techniques (Liggett and Cunge, 1975)[5]. Explicit methods, although simpler in application, are restricted by mathematical stability considerations to very small computational time steps (the order of a few seconds for most dambreak waves). Such small time steps cause the explicit methods to be very inefficient in the use of computer time.

Implicit finite-difference techniques (Preissmann, 1961; Amein and Fang, 1970; Strelkoff, 1970), however, have no restrictions on the size of the time step due to mathematical stability. However, convergence considerations may require its size to be limited (Fread, 1974a). Of the various implicit schemes that have been developed, the 'weighted four-point' scheme first used by Preissmann (1961), and more recently by Chaudhry (1973) and Fread (1974b, 1978) appears most advantageous since it can readily be used with unequal distance steps and its stability-convergence properties can be conveniently controlled[9].

In the weighted, four-point implicit finite-difference scheme, the continuous x - t region in which solutions of h and Q are sought, is represented by a rectangular net of discrete points. The net points are determined by the intersection of lines drawn parallel to the x and t axes. Those parallel to the t-axis represent locations of cross sections; they have a spacing of Δx, which need not be constant. Those parallel to the x-axis represent time lines; they have a spacing of Δt, which also need not be constant. Each point in the rectangular network can be identified by a subscript (i), which designates the x position and a superscript (j) which designates the particular time line.

The time derivatives are approximated by a forward difference quotient centered between the ith and i+ 1th points along the x-axis, i.e.,

$$\frac{\partial K}{\partial t} = \frac{K_i^{j+1} + K_{i+1}^{j+1} - K_i^j - K_{i+1}^j}{2\,\Delta t_j} \qquad (22)$$

where is K represents any variable (Q, h, A, A_o, s).

The spatial derivatives are approximated by a forward difference quotient positioned between two adjacent time lines according to weighting factors of θ and 1-θ, i.e.,

Variables other than derivatives are approximated at the time level where the spatial derivatives are evaluated by using the same weighting factors, i.e.,

$$\frac{\partial K}{\partial x} = \theta\,\frac{K_{i+1}^{j+1} - K_i^{j+1}}{x_i} + (1-\theta)\frac{K_{i+1}^{j} + K_i^{j}}{x_i} \qquad (23)$$

Variables not in the form of differential are approached based on linear time where space rate is expressed by weight factor as follows :

$$K = \theta\,\frac{K_i^{j+1} + K_{i+1}^{j+1}}{2} + (1-\theta)\frac{K_i^j + K_{i+1}^j}{2} \qquad (24)$$

A weighting factor θ of 1.0 yields the fully implicit or backward difference scheme used by Baltzer and Lai (1968). A weighting factor of 0.5 yields the box scheme used by Amein and Fang (1970). The influence of the q weighting factor on the accuracy of the computations was examined by Fread (1974a), who concluded that the accuracy tends to somewhat decrease as q departs from 0.5 and approaches 1.0. This effect becomes more pronounced as the magnitude of the computational time step increases[5].

Usually, a weighting factor of 0.60 is used so as to minimize the loss of accuracy associated with greater values while avoiding the possibility of a weak or pseudo instability noticed by Baltzer and Lai (1968), and Chaudhry (1973) for q values of 0.5. However, q may be specified other than 0.60 in the data input to the DAMBRK model via the parameter F_1.

When the finite-difference operators defined by Equations (22) through (24) are used to replace the derivatives and other variables in Equations (13) and (14), the following weighted, four-point implicit, finite-difference equations are obtained:

$$\theta \frac{Q_{i+1}^{j+1} + Q_i^{j+1}}{\Delta x_i} - \theta q_i^{j+1} + (1-\theta)\frac{Q_{i+1}^{j} - Q_i^{j}}{\Delta x_i} - (1-\theta) q_i^{j} +$$

$$\frac{s_i^{j+1}(A+A_0)_i^{j+1} + s_{i+1}^{j+1}(A+A_0)_{i+1}^{j+1} - s_i^{j}(A-A_0)_i^{j} - s_{i+1}^{j}(A+A_0)_{i+1}^{j}}{2\Delta t_j} = 0 \quad \text{.................} \quad (25)$$

$$\left(\frac{(sQ)_i^{j+1} + (sQ)_{i+1}^{j+1} - (sQ)_i^{j} - (sQ)_{i+1}^{j}}{2\,\Delta t_j}\right) + \theta\left(\frac{(sQ^2/A)_{i+1}^{j+1} - (sQ^2/A)_i^{j+1}}{\Delta x_i} +\right.$$

$$\left.\left. g\bar{A}^{j+1}\left(\frac{h_{i+1}^{j+1} - h_i^{j+1}}{\Delta x_i} + \bar{S}_f^{j+1} + S_e^{j+1} + \bar{S}_i^{j+1}\right)\right) + \right.$$

$$(1-\theta)\left(\frac{(sQ^2/A)_{i+1}^{j} - (sQ^2/A)_i^{j}}{\Delta x_i} +\right.$$

$$\left. g\bar{A}^{j}\left(\frac{h_{i+1}^{j} - h_i^{j}}{\Delta x_i} + \bar{S}_f^{j} + S_e^{j} + \bar{S}_i^{j}\right) = 0\right) \quad \text{.......................................} \quad (26)$$

where :

$A_{bar} = (A_i + A_{i+1}) / 2$

$S_t = n^2 Q \, |Q| \, / (2{,}2 \, A_{bar}^2 \, R_{bar}^{4/3}) = Q_{bar} \, |Q_{bar}| \, / \, K_{bar}^2$

$R_{bar} = A_{bar} / B_{bar}$

$B_{bar} = (B_i + B_{i+1}) / 2$

$K_{bar} = (K_i + K_{i+1}) / 2$

The terms associated with the j^{th} time line are known from either the initial conditions or previous computations. The initial conditions refer to values of h and Q at each node along the x-axis for the first time line (j=1). The initial conditions are further described in the section titled Initial Conditions, found later in this chapter.

Equations (25) and (26) cannot be solved in an explicit or direct manner for the unknowns since there are four unknowns and only two equations. However, if Equation (25) and (26) are applied to each of the (N-1) rectangular grids between the upstream and downstream boundaries, a total of (2N-2) equations with 2N unknowns can be formulated. (N denotes the total number of nodes or cross sections). Then, prescribed boundary conditions for sub-critical flows, one at the upstream boundary and one at the downstream boundary, provide the necessary two additional equations required for the system to be determinate. The boundary conditions are further described in the sections titled Upstream Boundary and Downstream Boundary, found later in this chapter. The resulting system of 2N nonlinear equations with 2N unknowns is solved by a functional iterative procedure, the Newton-Raphson method (***Amein and Fang,*** *1970*).

Computations for the iterative solution of the nonlinear system are begun by assigning trial values to the 2N unknowns. Substitution of the trial values into the system of nonlinear equations yields a set of 2N residuals. The Newton-Raphson method provides a means for correcting the trial values until the residuals are reduced to a suitable tolerance level. This is usually accomplished in one or two iterations through use of linear extrapolation for the first trial values. If the Newton-Raphson corrections are applied only once (i.e., there is no iteration), the nonlinear system of difference equations degenerates to the equivalent of a quasilinear, finite difference formulation of the St. Venant equations which may require smaller time steps than the nonlinear formulation for the same degree of numerical accuracy.

A system of 2N x 2N linear equations relates the corrections to the residuals and to a Jacobian coefficient matrix composed of partial derivatives of each equation with respect to each unknown variable in that equation. The Jacobian (coefficient) matrix of the linear system has a banded structure which allows the system to be solved by a compact, quad-diagonal, Gaussian elimination algorithm (Fread, 1971, 1985b)[5], which is very efficient with respect to computing time and storage. The required storage is 2N x 4 and the required number of computational steps is approximately 38N. A more detailed treatment of the solution technique is given elsewhere by Fread (1976, 1985b).

When flow is supercritical, the solution technique previously described can be somewhat simplified. Instead of a solution involving 2N x 2N equations, supercritical flow can be solved via a system of only 2 x 2 equations. The unknown h and Q at the upstream section are determined from the two boundary equations. Then, progressing from upstream to downstream in a cascade manner, Equations (25) and (26) are used to obtain h_{i+1} and Q_{i+1} at each section. Since equations (25) and (26) are nonlinear with respect to h_{i+1} and Q_{i+1}, they are solved by the Newton-Raphson iterative technique applied to a system of two equations with two unknowns. For supercritical flow, this technique has been found to provide a somewhat more stable solution than one involving 2N x 2N equations (Traver 1988).

4.1.6. UPSTREAM BOUNDARY CONDITIONS

The upstream boundary is required to obtain a solution of the St. Venant equations. In most applications of the model, this is simply a specified discharge hydrograph, i.e.,

$$Q_1 = QI(t) \qquad (27)$$

in which:

Q_1 = the flow at section 1 (the most upstream cross section)

$QI(t)$ = represents the specified flow at time (t)

The hydrograph values, QI(t), are specified at either constant or variable time intervals. Discharges are linearly interpolated from the table of discharge versus time. If the upstream flow is steady, i.e., it is constant for all time, the specified discharge table has the same discharge specified for all times. Generally, the upstream flow should not be zero. Also, the upstream hydrograph should be specified for the total duration of time that the St. Venant equations are to be solved.

If the water surface of the most upstream reservoir is assumed to remain level as it varies with time due to the inflows and spillway/breach outflows, then the following boundary equation is used:

$$Q_1 = QI(t) - 0.5 S_{bara} 43560.0 \frac{\Delta h}{\Delta t} \qquad (28)$$

Where :

Q_1 = the discharge at the upstream most section (the upstream face of the dam)

$QI(t)$ = the specified inflow to the reservoir

S_{bara} = the average surface area (acre) of the reservoir during the Δt time interval

Δt = the time step

Δh = the change in reservoir elevation during the time step

Equation (28) represents a level-pool routing algorithm in the form of an upstream boundary condition. The use of equation (28) requires that a table of reservoir surface area versus elevation be specified.

If the flow is supercritical at the upstream end of the routing reach, i.e.,

$$Fr_1 = \frac{V}{\sqrt{(gA_1B_1)}} \geq 1 \qquad (29)$$

Where :

Fr = the Froude number

A = Area of river cross section

B = Wide of river cross section

V = Velocity

g = gravitation acceleration

Two boundary equations are used at the upstream section. The first is Equation (25) and the second is the following stage-discharge relation:

$$Q_i = \left(\frac{1.49}{n_i}\right) A_i R^{2/3} \sqrt{S} = K_i \sqrt{S} \quad \text{.......... (30)}$$

in which :

$$S = \frac{(h_i - h_{i+1})}{\Delta x_i} \quad \text{.......... (31)}$$

and i = 1, the most upstream cross section.

4.3.7. DOWNSTREAM BOUNDARY CONDITIONS

When the flow near the downstream extremity of the routing reach is sub-critical, which is indicated the Froude number (Fr) values less than 1.

$$Fr_N = \frac{V_N}{\sqrt{\frac{gA_N}{B_N}}} < 1 \quad \text{.......... (32)}$$

where:

N = designates the number of the most downstream cross section, a known relationship between flow and depth or depth and time must be specified

Depending on the physical characteristics of the downstream section, the model can use one of the following downstream boundary equations:

a. *Single Rating Curve*

$Q_i = Q(h)$

b. Generate dynamic loop rating

$$Q_i = \left(\frac{1.49}{n_i}\right) A_i R^{2/3} \sqrt{S} = K_i \sqrt{S}$$

$$S = \frac{(h_{i-1} - h_i)}{\Delta x_{i-1}} + \frac{(Q'_i - Q_i)}{(0.5g(A_i + A_{i-1})\Delta t)} + \frac{\left[\frac{Q_{i-1}^2}{A_{i-1}} - \frac{Q_i^2}{A_i}\right]}{(0.5g(A_i + A_{i-1})\Delta x_{i-1})}$$

in which :

Q_i = the discharge at time (t - Δt)

R = the hydraulic radius

S = the slope

c. Water level time series

$$H_l = H(t)$$

In which :

H(t) = a specified time series of water elevation versus time (t)

CHAPTER V
AVAILABILITY OF DATA

5.1. SOURCES OF DATA

Almost all of data for Batutegi Dam such as reservoir and downstream channel is available. The input data required for the BOSS DAMBRK model can be categorized into two groups. The first data group pertains to the dam and inflow hydrograph into the reservoir, and the second group pertains to the routing of the outflow hydrograph through the downstream valley.

With reference to the data group pertaining to the dam, the information on reservoir elevation-volume relationship, spillway details elevation of bottom and top of dam, elevation of water surface in the reservoir at the beginning of analysis and at the time of failure, breach description data are required. Most of this information has been taken from the reports of *Way Seputih-Way Sekampung Irrigation Project by the Government of Ministry of Public Works, Directorate General of Water Resources Development Republic of Indonesia,* March 1982.

In the hypothetical dam break study using 'DAMBRK' model, the user has to supply the expected breach parameters. The typical ranges of breach parameters for different types of structures have been taken from some reference literatures.

5.2. AVAILABILITY OF DATA

5.2.1. FLOOD HYDROGRAPH INFLOW TO THE RESERVOIR

The design floods at the Batutegi Dam site had already been estimated for various return periods. The Peak flood design for various return periods are shown in ***Table V-1*** and ***Figure V-1***. The Probable Maximum Flood (PMF) is the synthetic unit hydrograph derived from the Probable Maximum Precipitation (PMP), which is adopted for the subsequent computation of the dam break analysis. The PMF at Batutegi Dam site is taken as in Final Report Hydrology of *Way Seputih-Way Sekampung Irrigation Project* is shown in ***Figure V-2***.

Table V-1. Design Flood For Way Sekampung River at Batutegi Dam

Return Period (Yr)	Peak Flood (m^3/sec)
10	1280
25	1500
50	1660
100	1810
250	2450
500	2830

5.2.2. RESERVOIR ELEVATION-VOLUME RELATIONSHIP

The reservoir elevation-volume relationship of Batutegi Dam has been taken from Final Report Hydrology of *Way Seputih-Way Sekampung Irrigation Project* is shown in ***Figure V-3***.

5.2.3. DAM STRUCTURES INFORMATIONS

The data of dam structures dimension are required as input data for Dam Break model such as length of dam crest, elevation of dam crest, spillway crest elevation (gate or ungated spillway), bottom of dam, cross section of dam site and spillway discharge rating curve. All of dam structures dimension have been taken from Salient Future of Batutegi Dam.

5.2.4. RIVER CROSS SECTION

The river cross sections pertaining to the routing of the outflow hydrograph through the downstream valley consist of a description of cross sections, hydraulic resistance coefficients and construction-expansion coefficient of the reach, steady state flow in the river at beginning of the simulation and the downstream boundary conditions. The cross sections are specific in kilometer, and tables of top width and corresponding elevation. The river cross section data of Way Sekampung River ware available from the Batutegi Dam site to around 20 Km up stream from river mouth.

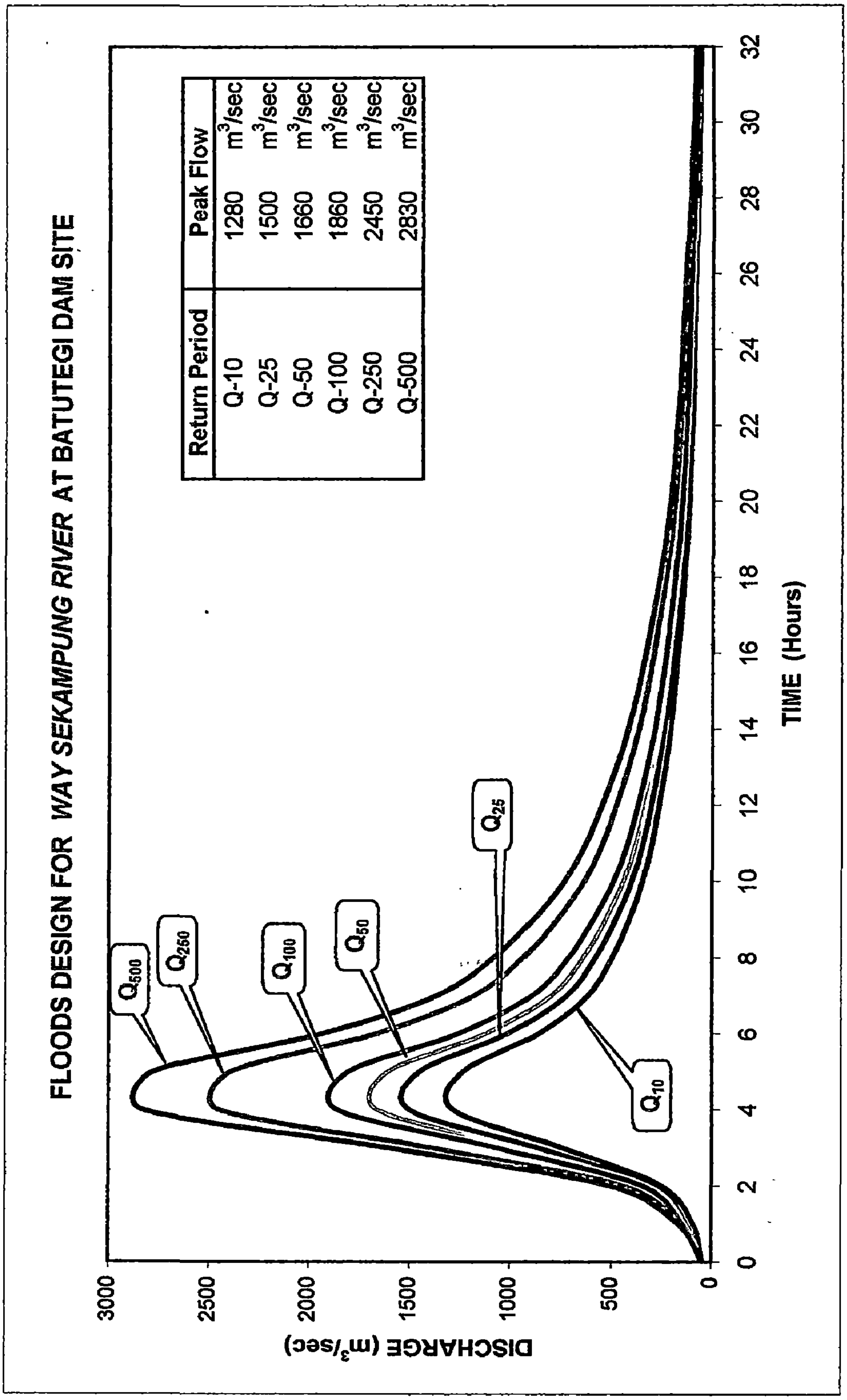

FIGURE V-1. VARIOUS RETURN PERIOD FLOOD DESIGN FOR BATUTEGI DAM

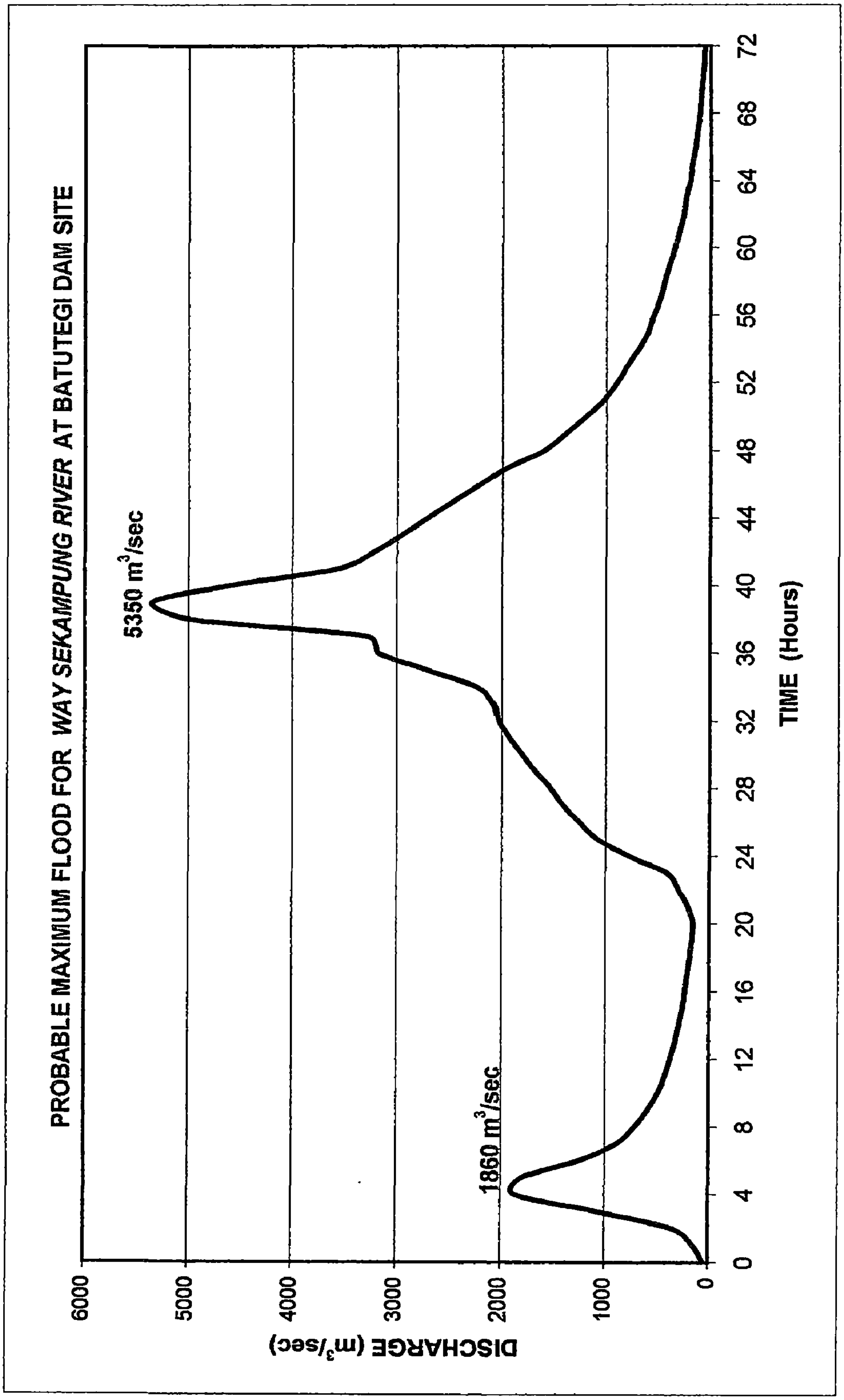

FIGURE V-2. PROBABLE MAXIMUM FLOOD DESIGN FOR BATUTEGI DAM

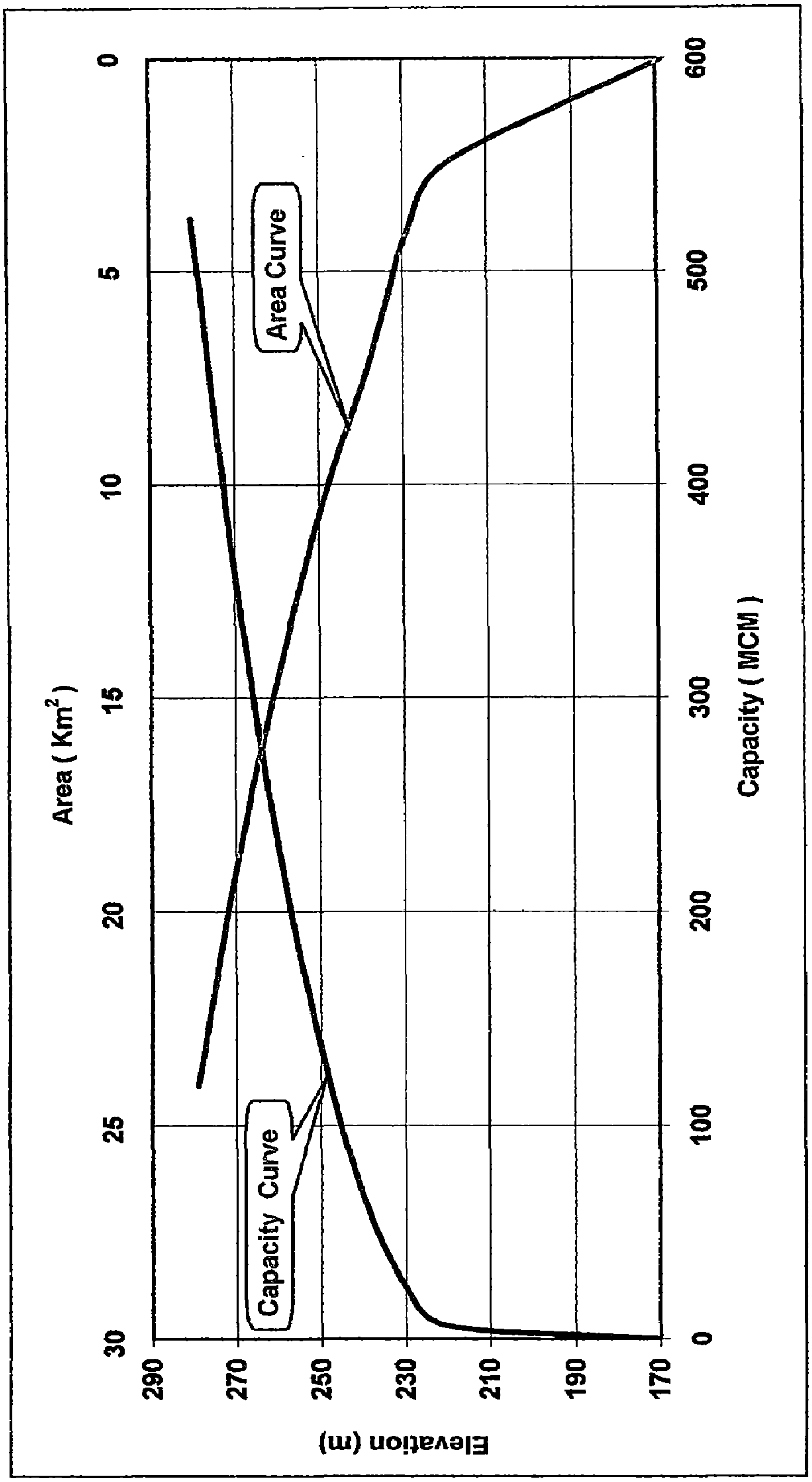

FIGURE V-3. BATUTEGI RESERVOIR CAPACITY - AREA CURVE

CHAPTER VI
RESULTS AND DISCUSSION

6.1. GENERAL

In this chapter, the results of dam break simulation carried out by the application of DAMBRK Model to Batutegi Dam failure have been discussed. It also includes the results of sensitivity analysis to investigate the effect of dam breach parameters on water level, peak discharge and time to reach peak discharge.

The computation of flood wave resulting from a dam breach basically involves two problems which can be considered jointly or separately: (1) the outflow hydrograph from the reservoir and (2) the routing of the flood wave downstream from the breached dam along the river channel and the flood plain. If breach outflow is independent of downstream conditions, or if their effect can be neglected, the reservoir outflow hydrograph is referred to as the free outflow hydrograph.

As explained in previous Chapter the study of Batutegi Dam Break used DAMBRK Model developed by Dr. D.L. Fread at US National Weather Service (NWS) and improved by BOSS Corporation. In this program, a parametric approach is used to describe the breaching process of a dam. For overtopping failure, the breach starts at a point on the dam crest, then enlarges with a given constant side slope until the prescribed terminal bottom elevation and width are achieved at the end of the prescribed failure time. The initial water-surface elevation and elevation at the start of the breach in the reservoir are specified. For piping failure, piping centerline elevation is specified.

The complete Saint-Venant equation used for one-dimensional unsteady open - channel flow, are written in conservation form. The numerical algorithm is the weighted, four-point implicit scheme. The resultant non-linear equations at each time step are solved by the Newton-Raphson iterative procedure.

6.2. INPUT DATA REQUIREMENT

The input data requirements for the 'DAMBRK' program are classified into two groups:

1. **Data Pertaining To The Dam**

   (I) Reservoir data

   These are inflow hydrograph, initial elevation of water in reservoir, elevation of water in reservoir when breach occurs, elevation of top of dam, bottom elevation of dam, and reservoir volume and surface area and their corresponding elevations.

   Inflow hydrograph to the reservoir adopted from Design Flood Hydrogaph for Batutegi Dam. There are flood hydrographs of five return periods of 10 year, 100 year, 500 year and PMF.

   Other reservoir data is taken from salient futures of Batutegi Dam. Based on reservoir conditions some scenarios have been analysed in this study. Following reservoir data input are used in this study :

   - *Reservoir initial elevation of water* is + 274.00, which is the full water level at reservoir equal to crest of spillway elevation.
   - *Water surface elevation in the reservoir when breach occurs.* It is dependent on flood inflow hydrograph and scenario of dam failure. For no inflow to the reservoir and 10, 100, 500 year return period scenario the elevation of water in reservoir when breach occurs was applied at elevation +274.00. For PMF flood hydrograph inflow to the reservoir scenario, cases are analised when breach occurs 1, 2, 3 hour before peak flow of PMF occurs and 1 hour after peak flow of PMF. The elevation of water in the reservoir at 1, 2, 3 hour before peak flow occurs are +279.305, +279.816, +280.274 respectively, and for 1 hour after peak flow case the water elevation is at +280.68.
   - *Top of dam crest elevation* is +283.00, which is the maximum elevation of dam.
   - *Bottom elevation* of Batutegi Dam is +172.00. It is same as the elevation of river bed

- *Reservoir Capacity-Surface Area Curves.* It is curve between elevation and reservoir volume and surface area. This curve is after 50 years lifetime of the reservoir. **Figure V-3** shows the relationship between Elevation with capacity and area of reservoir.

(II) Breach Parameter data

Some parameters of breach form are final bottom width, side slope, elevation of breach bottom and time taken for the full breach formation. Those parameters are assumed and are described below :

The final bottom width of breach. It is means final width of the breach base after breach failure occurs. This parameter is most of uncertainty. For this reason the final bottom width of breach in this study is taken as 20 meter, 40 meter, 60 meter and 100 meter (see **Table VI-1**). The final elevation of breach bottom is at elevation +208.00.

Side slope of breach is side slope of the reservoir breach form. This value is the horizontal component corresponding to the vertical component of 1. This is uncertain condition so to know the sensitivity of this parameter in this study, the side slope of breach value is taken ranging from 1:0.5 to 1:4 with increments of 0.5. The various side slopes of breach combined with other breach parameters are given in **Table VI-1**.

Breach bottom elevation is set up at elevation +208.00. It is elevation of dead storage of reservoir. Dead storage of reservoir corresponds to 50 years life of reservoir. The maximum deposit of sediment in the reservoir will be up to +208.00 after 50 years.

Time taken for the full breach formation is time elapsed from the beginning of breach formation until the breach has reached its maximum size at elevation of breach bottom. One of important breach parameter is time taken for breach formation, according to the dam data condition and breach scenario in this study its set up in various value of time. There are 4 hour, 5 hour and 6 hour (see **Table VI-1**).

(III) Spillway data

Spillway data includes spillway-rating curve, spillway crest elevation, coefficient of discharge of uncontrolled spillway, coefficient of discharge of crest of dam, and constant discharge from dam such as discharge through turbines. The spillway type of Batutegi Dam is ungated spillway with crest elevation of +274.00. Coefficient discharge of spillway assumed as 2.1 and for crest of dam with overtopping event assumed as 1.8. Constant discharge from other outlet structure is 180 m^3/sec, which is discharge through turbines.

2. Data Pertaining To The Downstream Routing Reach

Down stream routing reach data contain information, such as detail cross sections of river valley downstream of dam site, hydraulic resistance coefficient (Manning's roughness coefficients) and initial conditions in the downstream channel.

Detail cross section of river valley downstream of dam site, Batutegi Dam is located on Way Sekampung River. Distance from dam site to river mouth is 180 km. Cross sections in a reach of about 164 km from dam site to river mouth are available. This reach is divided in more than 60 of cross sections.

Hydraulic resistance coefficient. A comprehensive representation of channel resistance to flow in river is difficult, and direct measurement is not possible. Hydraulic resistance coefficient (Manning's Roughness) usually estimated by field observation, and calibrated using observed high water marks from historical flood events. No information about it is available data from previous studies. Therefor for this study reference is made to standard texts.

Initial conditions in the downstream channel. The Dambrk model demands for the channel routing analysis a steady-flow situation throughout the channel reach before routing the inflow hydrograph. Therefore, the initial flow in the channel at the beginning of analysis is assumed as 180 m^3/sec, which is the discharge release for turbine operation.

6.3. MODEL RUNS

The model is run for two scenarios.

6.3.1. Overtopping Scenario

Batutegi Dam is designed not to be overtopped. But it can happen in worst condition. Hence this scenarios has been analysed. Four type of flood inflow hydrographs are applied in this scenario. No inflow condition is also analysed. Various model runs show the maximum peak discharge occurs with PMF inflow hydrograph through the breach formation which occur in a period of 4 hours with breach width of 100 m and side slope of 1:4. It occurs 1 hour before the occurrence of second PMF inflow peak. The maximum peak discharge through the breach formation is 78,610 m^3/sec. The formation of overtopping breach is shown in **Figure VI-1**.

The model result summary of peak flows (in cumecs) for overtopping scenario is given in **Table VI-2a** to **Table VI-6d**. **Figure VI-2a** to **Figure VI-2e** show the out flow hydrograph for different flood inflow hydrographs applied and also for no inflow condition.

6.3.2. Piping Scenario

Piping process is a result of removal of soil particles from the body of the dam by erosive action of the seepage flow. In this scenario a piping failure is simulated as a rectangular orifice breach that grows with time and is centered at a specified elevation within the dam (refer to **Figure VI-3**). In this study center elevation of piping location is assumed at +260.00, for all conditions of flood inflow hydrographs to the reservoir. The summary of results of piping scenario is given in **Table VI-7**. The tables show the maximum peak flow through a breach occurring in 4 hrs with final breach width of 100 m. In overtopping scenario (same breach parameters adopted) the peak discharge through piping breach formation is 62,990 m^3/sec, which is less than peak flow from overtopping failure.

6.4. SENSITIVITY ANALYSIS OF BREACH PARAMETERS

Sensitivity analysis has been carried out for the breach parameters such as breach development time, breach width and breach slope to investigate the effect of change in values of each parameter on water levels, peak discharge and travel time of peak flow through the river.

Sensitivity analysis of each dam break variable on the resulting dam break flood discharge on the downstream of the dam, various runs have been made using Dambrk Model. Analysis has been repeated by applying four design inflow hydrographs and no inflow to the reservoir condition with various values of three dam breach parameters. Four of flood inflow hydrographs and no inflow condition are combined with three parameters of dam breach are described on **Table VI-1**.

The characteristic of Probable Maximum Flood design for Batutegi Dam is shown in **Figure V-3.** It has two peaks discharge, first occurs at 4 hr and second at 40 hr from the beginning of flood hydrograph. According on this condition the starting time of dam failure was decided at four different times : 38 hr, 39 hr, 40 hr and 41 hr, so that the effect of that particular time of dam failure on the resulting flood wave can be evaluated and its sensitivity analysed.

The result of peak outflow hydrograph in various conditions is given in **Figure VI-4a** to **Figure VI-7c.** These figures show different effects in inflow hydrograph, final breach formation time, time to dam failure and breach side slope.

6.4.1. BREACH SLOPE

Six values of breach slope applied are shown in **Table VI-1**. By increasing the breach slope, each flood inflow hydrograph, has shown increase in peak outflow from the breach formation. It is clear from the **Table VI-2d** and **Figure VI-8a**. (in case no inflow hydrograph condition, final breach width 100 m and time to failure 4 hours), that by doubling the breach slope from 1:0.5 to 1:1 the peak discharge is found to increase from 61,430 m^3/sec to 62,635 m^3/sec, the peak discharge increases by 1,205 m^3/sec or 1.96% only. **Figure VI-8b** (for breach width of 20 m) shows that increase of breach slope has more significant effect, upto a slope of 1:1.0. The peak discharge increases from 32,742 m^3/sec to 43,466 m^3/sec, for an increase in slope from 1:0.5 to 1:1.0. However, in this case the effect of inflow discharge is insignificant.

The effect of breach slopes can be summarised as below :

1. An increase in breach slope has an increasing effect on peak discharge. The increase in peak outflow is more significant when breach slope increased from 1:0.5 to 1:1.0.
2. The impact of breach slope is found insignificant on peak outflow in case the breach width is small. The results are presented in figure VI-6a and Figure VI-6b.

6.4.2. FINAL BREACH WIDTH

It is observed from the **Table VI-2a** to **Table VI-6d**, that change in the final breach width has an effect on peak discharge through the breach formation. **Figure VI-9a** shows the peak discharge through the breach at different inflow hydrographs for different final breach width. Observed from the **Figure VI-9a** that increase in the final breach width from 20 m to 100 m, there is an increase in the peak discharge from 61,332 m^3/sec to 66,157 m^3/sec. **Figure VI-9b** however shows that change in outflow peak is affected by final breach width but is not affected by inflow pattern when the breach slope is steep (1 :0.5).

Figure VI-10a and **Figure VI-10b** show that change in final breach width have a slight effect on peak discharge along the longitudinal profile of the river, particularly after 60 Km from dam site. **Figure VI-10b** shows at location 5 Km downstream of dam site the peak flow for final breach width 20 m and 100 m as 71,959 m^3/sec and 78,113 m^3/sec respectively (difference 6,154 m^3/sec). At same graph, after 60 km from dam site peak flow showing 17,968 m^3/sec for final breach width of 20 m and 18,318 m^3/sec for 100 m final breach width (difference 350 m^3/sec). Similarly flood levels along the longitudinal profile of the river, show insignificant effect of final breach width.

The effect in change of the final breach size is summarised as follow :

1. Increase in the final breach size has an increasing effect on peak discharge flow through breach formation.

2. The effect of final breach size based on four value of final breach size (20 m, 40 m, 60 m and 100 m) shows that peak discharge at close location from dam site is significant but as the distance from dam site increases the effect decreases.
3. With steep breach slope (1:0.5). change of final breach size has a significant effect on peak discharge, but inflow pattern has no significant effect.

6.4.3. BREACH DEVELOPMENT TIME

It is seen from the **Table VI-2a** to **Table VI-6d** that the peak outflow discharge through the breach formation decreases with increase in breach development time. The effect is shown on **Figure VI-11.** However the flood levels at different sites along the river remain nearly the same with increase in the breach development time.

It is evident that an increase in breach development time has a decreasing effect on peak discharge through breach formation.

6.4.4. FLOOD INFLOW HYDROGRAPH

Four various return period flood inflow hydrographs and one condition of no inflow to the reservoir were adopted for sensitivity analysis. The output of various runs are as shown in **Table VI-2a** to **Table VI-6d**. It is found that an increase in flood inflow hydrograph has generally an increasing effect on peak outflow discharge passing through breach formation.

As explained in paragraph 6.2 the PMF hydrograph of the river at Batutegi Dam is an unique hydrograph, as its has two peaks. First peak flow of 1,860 m^3/sec occurs at 4 hr and the second of 5,350 m^3/sec at 40 hr from the starting of flood hydrograph. Therefore according to this characteristic, 5 values of time to dam failure were adopted (0 hr, 38 hr, 39 hr, 40 hr and 41 hr) for sensitivity analysis. To work out the maximum peak outflow hydrograph through the breach formation, the model was run with different breach development time, which are 4 hr, 5 hr and 6 hr at same final breach width. The outflow hydrographs for different timings to dam failure and for breach development time of 4 hour are given in **Figure VI-12**. The summary of peak discharge passing through breach formation for various timings to dam failure are plotted in **Figure VI-13a** to **Figure VI-13c**.

It is observed from **Figure VI-12** and **Figure VI-13a** to **Figure VI-13c**, the maximum peak outflow occurs when dam failure occurs at 39 hour for breach formation time of 4 hours. The peak discharge is 78,610 m^3/sec. If breach formation time increases the peak outflow discharge decreases.

6.5. SUMMARY OF THE RESULT OF BREACH PARAMETER

The result of sensitivity analysis of breach parameters shows, that all the breach parameters are sensitive with regard to peak outflow discharge. An increase in flood inflow hydrograph, in final breach width, and in the breach slope has an effect to increase the peak outflow discharge. However an increase in breach development time has an effect to decrease the peak outflow discharge through breach formation.

6.6. MODEL APLICATION

Flood plain analysis has been based on results of breach parameters sensitivity analysis. Following breach parameters are selected for model application to Batutegi Dam failure.

1. Flood inflow hydrograph used Probable Maximum Flood (PMF) hydrograph
2. Slope of breach formation 1 : 4
3. Final Breach size is 100 m
4. Breach Development Time is 4 hour
5. Time to failure is 39 hour or 1 hour before the second peak of flood hydrograph occurs.

6.6.1. PEAK DISCHARGE

Peak outflow discharge hydrograph at various cross sections along the longitudinal river is shown in **Figure VI-14.** The figure shows that peak discharge at first 10 km downstream of dam site changes slightly. Just below dam site peak outflow is 78,610 m^3/sec and it becomes 77,153 m^3/sec at 11 km from dam site. The rapid change occurs in the reach of 12 km to 20 km from dam site. The peak discharge in this reach changes from 76.060 m^3/sec to 52,970 m^3/sec From 20 km to around 60 km peak discharge changes gradually to 18,320 m³/sec. Thereafter it gradually reduce to 5,000 m³/sec at Km 163.650.

6.6.2. TIME TO PEAK DISCHARGE

The time of arrival for peak flood stage at different locations along the river is shown in **Figure VI-15**. Time is accounted from the start of the dam break. It takes about 44.83 hours for the peak stage to travel from the dam site to the downstream around 20 km from river mouth. The figure shows that peak flow through breach formation (0 km) occurs at 44.53 hr, which is 5.53 hours after the time of the beginning of dam breach occurrence (time to dam failure 39 hr).

6.6.3. WATER SURFACE ELEVATION

The result of the run made by using PMF inflow hydrograph to know the maximum water surface elevation along the river below Batutegi Dam site is shown **Table VI-8** and **Figure VI-16**. The change of flood depth along the river is closely related to the geometry and size of the river channel. In the upper reach the river course is narrow, while in the lower reach, the river becomes wider, with increased width all the way to the downstream up to 20 km before river mouth.

Figure VI-16 shows that in most of reaches along the river the flooding elevation is over the banks. This indicates that at these locations, the river flow will over the bank. The duration time of river flow over the bank at each location along the river is different. **Figure VI-17a** to **Figure VI-17e** show various locations with range of flooding duration time. Details of flooding duration time all along the river valley are given in **Table VI-8**.

Table VI-1. Flood Inflow Hydrograph Variables and Breach Parameters

Flood Inflow	Final Breach Width (m)	Time to Failure (hrs)	Side Slope of Breach Formation (1 : Z)
No Inflow	20 , 40 , 60 , 100	4 , 5 , 6	0.5; 1.0; 1.5; 2.0; 2.5; 3.0; 3.5; 4.0
10 Yr	20 , 40 , 60 , 100	4 , 5 , 6	0.5; 1.0; 1.5; 2.0; 2.5; 3.0; 3.5; 4.0
100 Yr	20 , 40 , 60 , 100	4 , 5 , 6	0.5; 1.0; 1.5; 2.0; 2.5; 3.0; 3.5; 4.0
500 Yr	20 , 40 , 60 , 100	4 , 5 , 6	0.5; 1.0; 1.5; 2.0; 2.5; 3.0; 3.5; 4.0
PMF	20 , 40 , 60 , 100	4 , 5 , 6	0.5; 1.0; 1.5; 2.0; 2.5; 3.0; 3.5; 4.0

TABLE VI-2a. BREACH PEAK FLOW DISCHARGE
No Inflow to Reservoir Condition
Final Breach Base Width = 20 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	32,742	31,392	29,558
1 : 1.0	43,466	40,591	36,830
1 : 1.5	50,386	45,913	40,289
1 : 2.0	54,556	48,596	41,477
1 : 2.5	56,731	49,650	42,215
1 : 3.0	57,698	50,443	42,858
1 : 3.5	58,538	51,144	43,439
1 : 4.0	59,239	51,746	43,928

TABLE VI-2b. BREACH PEAK FLOW DISCHARGE
No Inflow to Reservoir Condition
Final Breach Base Width = 40 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	42,976	40,543	37,273
1 : 1.0	50,756	46,630	41,325
1 : 1.5	55,429	49,690	42,598
1 : 2.0	57,867	50,774	43,201
1 : 2.5	58,831	51,437	43,731
1 : 3.0	59,533	52,010	44,196
1 : 3.5	60,168	52,565	44,635
1 : 4.0	60,730	53,040	45,022

TABLE VI-2c. BREACH PEAK FLOW DISCHARGE
No Inflow to Reservoir Condition
Final Breach Base Width = 60 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	51,050	47,312	42,377
1 : 1.0	56,294	50,809	43,832
1 : 1.5	59,030	51,954	44,234
1 : 2.0	59,997	52,461	44,632
1 : 2.5	60,564	52,952	44,989
1 : 3.0	61,138	53,404	45,361
1 : 3.5	61,621	53,825	45,695
1 : 4.0	62,067	54,207	46,003

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TABLE VI-2d. BREACH PEAK FLOW DISCHARGE
No Inflow to Reservoir Condition
Final Breach Base Width = 100 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	61,430	54,637	46,539
1 : 1.0	62,635	54,781	46,598
1 : 1.5	62,893	54,977	46,744
1 : 2.0	63,154	55,205	46,891
1 : 2.5	63,509	55,450	47,088
1 : 3.0	63,825	55,714	47,288
1 : 3.5	64,132	55,965	47,476
1 : 4.0	64,434	56,202	47,650

TABLE VI-3a. BREACH PEAK FLOW DISCHARGE
Q_{10} Yr Applied
Final Breach Base Width = 20 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	33,140	31,944	30,261
1 : 1.0	44,072	41,410	37,822
1 : 1.5	51,161	46,928	41,464
1 : 2.0	55,461	49,760	42,743
1 : 2.5	57,746	50,863	43,484
1 : 3.0	58,732	51,612	44,132
1 : 3.5	59,490	52,249	44,679
1 : 4.0	60,151	52,839	45,164

TABLE VI-3b. BREACH PEAK FLOW DISCHARGE
Q_{10} Yr Applied
Final Breach Base Width = 40 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	43,499	41,263	38,177
1 : 1.0	51,472	47,584	42,461
1 : 1.5	56,294	50,820	43,849
1 : 2.0	58,857	51,968	44,472
1 : 2.5	59,849	52,591	44,995
1 : 3.0	60,472	53,144	45,431
1 : 3.5	61,087	53,648	45,864
1 : 4.0	61,601	54,102	46,236

TABLE VI-3c. BREACH PEAK FLOW DISCHARGE
Q_{10} Yr Applied
Final Breach Base Width = 60 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	51,699	48,196	43,466
1 : 1.0	57,116	51,900	45,116
1 : 1.5	59,993	53,204	45,495
1 : 2.0	61,037	53,657	45,886
1 : 2.5	61,571	54,104	46,266
1 : 3.0	62,047	54,477	46,822
1 : 3.5	62,504	54,878	46,903
1 : 4.0	62,916	55,231	47,189

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TABLE VI-3d. BREACH PEAK FLOW DISCHARGE
Q_{10} Yr Applied
Final Breach Base Width = 100 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	62,331	55,846	47,837
1 : 1.0	63,678	55,969	47,900
1 : 1.5	63,876	56,107	48,009
1 : 2.0	64,131	56,338	48,166
1 : 2.5	64,376	56,521	48,306
1 : 3.0	64,662	56,723	48,464
1 : 3.5	64,931	56,938	48,630
1 : 4.0	65,192	57,142	48,816

TABLE VI-4a. BREACH PEAK FLOW DISCHARGE
Q_{100} Yr Applied
Final Breach Base Width = 20 m

Side Slope of Breach Form	Time of Breach Form (hr)		
	4	5	6
1 : 0.5	33,317	32,194	30,586
1 : 1.0	44,341	41,783	38,284
1 : 1.5	51,507	47,392	42,013
1 : 2.0	55,867	50,294	43,332
1 : 2.5	58,202	51,420	44,082
1 : 3.0	59,195	52,148	44,724
1 : 3.5	59,914	52,751	45,254
1 : 4.0	60,562	53,335	45,736

TABLE VI-4b. BREACH PEAK FLOW DISCHARGE
Q_{100} Yr Applied
Final Breach Base Width = 40 m

Side Slope of Breach Form	Time of Breach Form (hr)		
	4	5	6
1 : 0.5	43,730	41,590	38,595
1 : 1.0	51,791	48,020	42,991
1 : 1.5	56,682	51,338	44,451
1 : 2.0	59,302	52,531	45,065
1 : 2.5	60,304	53,119	45,583
1 : 3.0	60,922	53,674	46,026
1 : 3.5	61,494	54,138	46,432
1 : 4.0	61,992	54,581	46,797

TABLE VI-4c. BREACH PEAK FLOW DISCHARGE
Q_{100} Yr Applied
Final Breach Base Width = 60 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr (m^3/sec)	5 hr (m^3/sec)	6 hr (m^3/sec)
1 : 0.5	51,988	48,599	43,970
1 : 1.0	57,484	52,399	45,712
1 : 1.5	60,425	53,778	46,096
1 : 2.0	61,523	54,212	46,479
1 : 2.5	62,021	54,628	46,858
1 : 3.0	62,449	54,991	47,151
1 : 3.5	62,892	55,353	47,460
1 : 4.0	63,286	55,692	47,735

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TABLE VI-4d. BREACH PEAK FLOW DISCHARGE
Q_{100} Yr Applied
Final Breach Base Width = 100 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr (m^3/sec)	5 hr (m^3/sec)	6 hr (m^3/sec)
1 : 0.5	62,733	56,400	48,442
1 : 1.0	64,148	56,529	48,502
1 : 1.5	64,331	56,657	48,594
1 : 2.0	64,565	56,851	48,755
1 : 2.5	64,756	57,027	48,890
1 : 3.0	65,026	57,202	49,034
1 : 3.5	65,277	57,393	49,191
1 : 4.0	65,517	57,594	49,360

TABLE VI-5a. BREACH PEAK FLOW DISCHARGE
Q_{500} Yr Applied
Final Breach Base Width = 20 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	33,647	32,647	31,161
1 : 1.0	44,842	42,452	39,097
1 : 1.5	52,146	48,222	42,975
1 : 2.0	56,612	51,242	44,366
1 : 2.5	59,035	52,406	45,131
1 : 3.0	60,041	53,101	45,763
1 : 3.5	60,699	53,701	46,268
1 : 4.0	61,332	54,227	46,744

TABLE VI-5b. BREACH PEAK FLOW DISCHARGE
Q_{500} Yr Applied
Final Breach Base Width = 40 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	44,162	42,179	39,332
1 : 1.0	52,381	48,801	43,919
1 : 1.5	57,396	52,258	45,505
1 : 2.0	60,114	53,558	46,104
1 : 2.5	61,163	54,112	46,613
1 : 3.0	61,758	54,620	47,071
1 : 3.5	62,256	55,036	47,433
1 : 4.0	62,733	55,451	47,786

TABLE VI-5c. BREACH PEAK FLOW DISCHARGE
Q_{500} Yr Applied
Final Breach Base Width = 60 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	52,524	49,321	44,856
1 : 1.0	58,161	53,286	46,757
1 : 1.5	61,215	54,791	47,166
1 : 2.0	62,405	55,199	47,543
1 : 2.5	62,848	55,566	47,898
1 : 3.0	63,226	55,924	48,194
1 : 3.5	63,626	56,224	48,449
1 : 4.0	63,995	56,532	48,701

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TABLE VI-5d. BREACH PEAK FLOW DISCHARGE
Q_{500} Yr Applied
Final Breach Base Width = 100 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	63,471	57,382	49,512
1 : 1.0	65,005	57,555	49,557
1 : 1.5	65,200	57,644	49,643
1 : 2.0	65,371	57,773	49,792
1 : 2.5	65,533	57,939	49,921
1 : 3.0	65,725	58,095	50,047
1 : 3.5	65,948	58,258	50,185
1 : 4.0	66,157	58,423	50,326

TABLE VI-6a. BREACH PEAK FLOW DISCHARGE
PMF Applied
Final Breach Base Width = 20 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	40,523	38,979	36,833
1 : 1.0	54,144	50,656	46,051
1 : 1.5	62,601	57,062	50,022
1 : 2.0	67,346	59,898	51,265
1 : 2.5	69,464	61,012	52,164
1 : 3.0	70,543	61,933	52,944
1 : 3.5	71,513	62,768	53,601
1 : 4.0	72,380	63,498	54,232

TABLE VI-6b. BREACH PEAK FLOW DISCHARGE
PMF Applied
Final Breach Base Width = 40 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4 hr	5 hr	6 hr
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	52,619	49,847	46,082
1 : 1.0	62,627	57,698	51,269
1 : 1.5	68,322	61,293	52,679
1 : 2.0	70,960	62,385	53,372
1 : 2.5	71,942	63,161	53,992
1 : 3.0	72,736	63,848	54,564
1 : 3.5	73,468	64,471	55,049
1 : 4.0	74,164	65,044	55,545

TABLE VI-6c. BREACH PEAK FLOW DISCHARGE
PMF Applied
Final Breach Base Width = 60 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4	5	6
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	62,326	58,086	52,388
1 : 1.0	69,163	62,634	54,296
1 : 1.5	72,435	63,938	54,708
1 : 2.0	73,448	64,499	55,125
1 : 2.5	74,078	65,032	55,550
1 : 3.0	74,683	65,535	55,988
1 : 3.5	75,228	66,004	56,350
1 : 4.0	75,774	66,430	56,718

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TABLE VI-6d. BREACH PEAK FLOW DISCHARGE
PMF Applied
Final Breach Base Width = 100 m

Side Slope of Breach Formation	Time Taken Breach Formation		
	4	5	6
	(m^3/sec)	(m^3/sec)	(m^3/sec)
1 : 0.5	75,228	67,488	57,689
1 : 1.0	76,922	67,584	57,789
1 : 1.5	77,113	67,699	57,839
1 : 2.0	77,358	67,890	58,003
1 : 2.5	77,617	68,097	58,172
1 : 3.0	77,974	68,379	58,388
1 : 3.5	78,293	68,653	58,594
1 : 4.0	78,610	68,920	58,809

Table VI-7. Summary of Peak Outflow Due To Piping
Centre Elevation of Piping Location +260.00

Final Breach Width	Flood Inflow Return Period				
	No Inflow	10	100	500	PMF
	(m^3/sec)	(m^3/sec)	(m^3/sec)	(m^3/sec)	(m^3/sec)
Time To Failure 4 hr					
20.00 m	43,934	44,382	44,569	44,962	56,000
40.00 m	45,501	45,923	46,091	46,465	57,953
60.00 m	46,925	47,319	47,481	47,827	59,751
100.00 m	49,427	49,765	49,904	50,218	62,990
Time To Failure 5 hr					
20.00 m	38,483	39,023	39,263	39,746	49,636
40.00 m	39,853	40,363	40,586	41,035	51,360
60.00 m	41,090	41,580	41,786	42,209	52,965
100.00 m	43,295	43,712	43,885	44,283	55,822
Time To Failure 6 hr					
20.00 m	32,798	33,470	33,788	3,440	43,035
40.00 m	33,973	34,601	34,903	35,482	44,525
60.00 m	35,041	35,631	35,915	36,457	45,919
100.00 m	36,898	37,450	37,687	38,180	48,422

Table VI.8. Summary of Flood Parameters

Distance From Dam Site	Peak Discharge	Time To Peak	Time To Reach Top of Leave		Duration of Inundation Flood Area
(Km)	(x 1,000 m^3/sec)	(hr)	Beginning	End	(hr)
0.00	78.610	44.526	41.373	47.839	6.466
1.00	78.490	44.526	42.347	47.108	4.761
5.00	78.110	44.526	42.746	45.960	3.214
10.00	77.000	44.956	42.440	48.876	6.436
15.00	66.060	45.386	42.582	50.631	8.050
20.00	52.970	45.816	42.986	51.266	8.280
25.00	51.820	45.816	43.786	50.148	6.362
30.00	50.480	46.246	44.384	49.598	5.214
34.00	47.890	46.676	44.457	53.538	9.081
40.00	38.260	48.396	45.166	60.297	15.132
46.00	27.450	48.826	45.613	60.870	15.257
49.00	26.960	48.826	45.992	58.722	12.731
56.00	21.600	52.266	46.103	69.689	23.587
60.00	18.320	53.126	47.115	68.423	21.308
66.00	17.000	54.416	47.554	71.296	23.742
70.00	16.550	54.416	47.742	73.897	26.155
74.00	16.500	54.846	47.736	74.695	26.959
81.00	16.440	55.405	49.574	68.715	19.140
84.00	16.350	56.781	49.330	74.823	25.493
89.00	15.820	58.845	50.374	77.386	27.012
97.00	14.080	61.021	51.072	87.108	36.036
100.00	13.550	61.765	51.448	92.254	40.806
105.00	12.670	63.253	52.276	94.726	42.450
110.00	11.840	63.996	52.946	100.783	47.837
115.00	10.390	64.740	53.629	108.265	54.637
119.00	9.890	65.484	53.684	121.832	68.148
125.00	9.470	68.460	53.735	137.856	84.121
139.00	8.200	73.260	57.359	140.691	83.332
144.00	7.890	76.789	60.339	136.360	76.021
149.00	6.760	85.023	61.893	150.651	88.758
155.00	5.450	87.635	62.626	163.588	100.961
159.00	5.240	87.635	63.996	163.501	99.505
160.00	5.170	89.355	64.232	164.126	99.893
163.65	5.000	89.355	66.069	158.858	92.790

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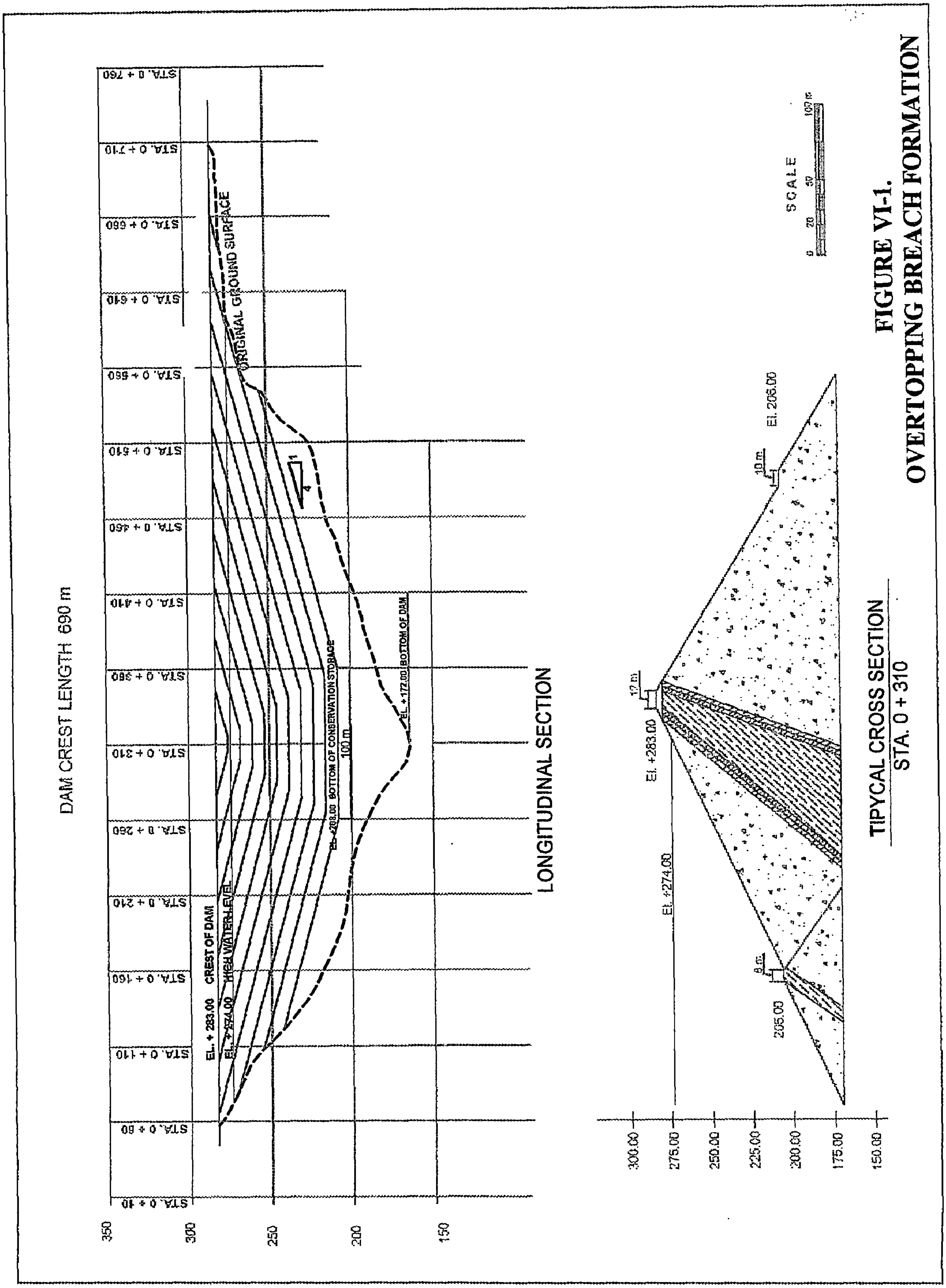
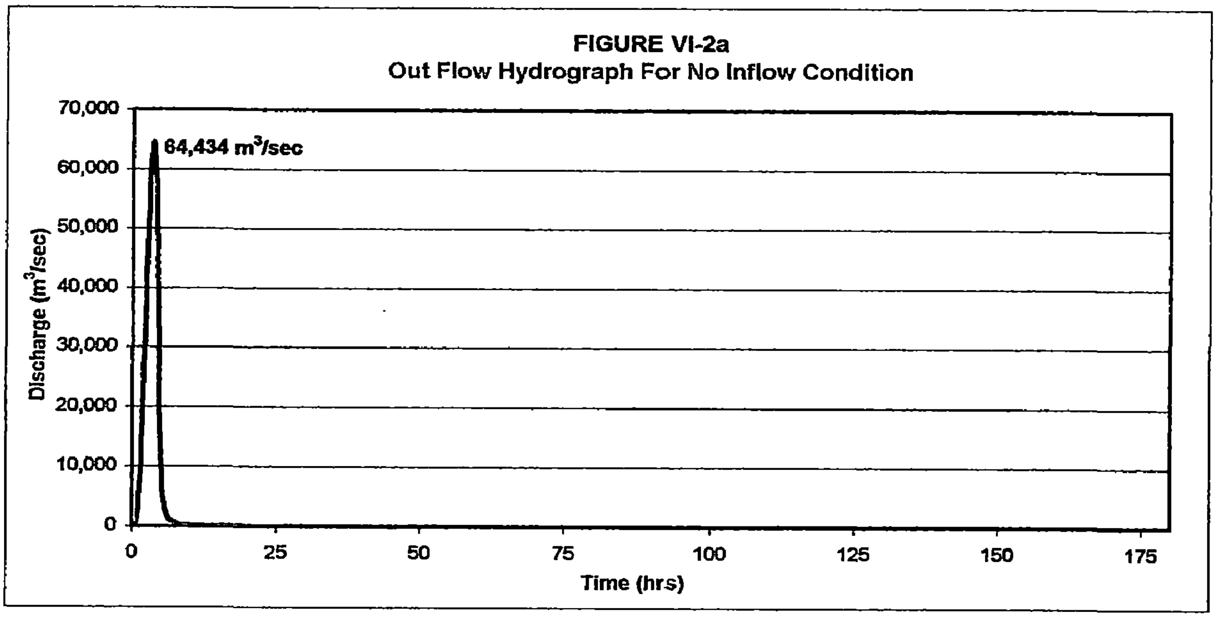
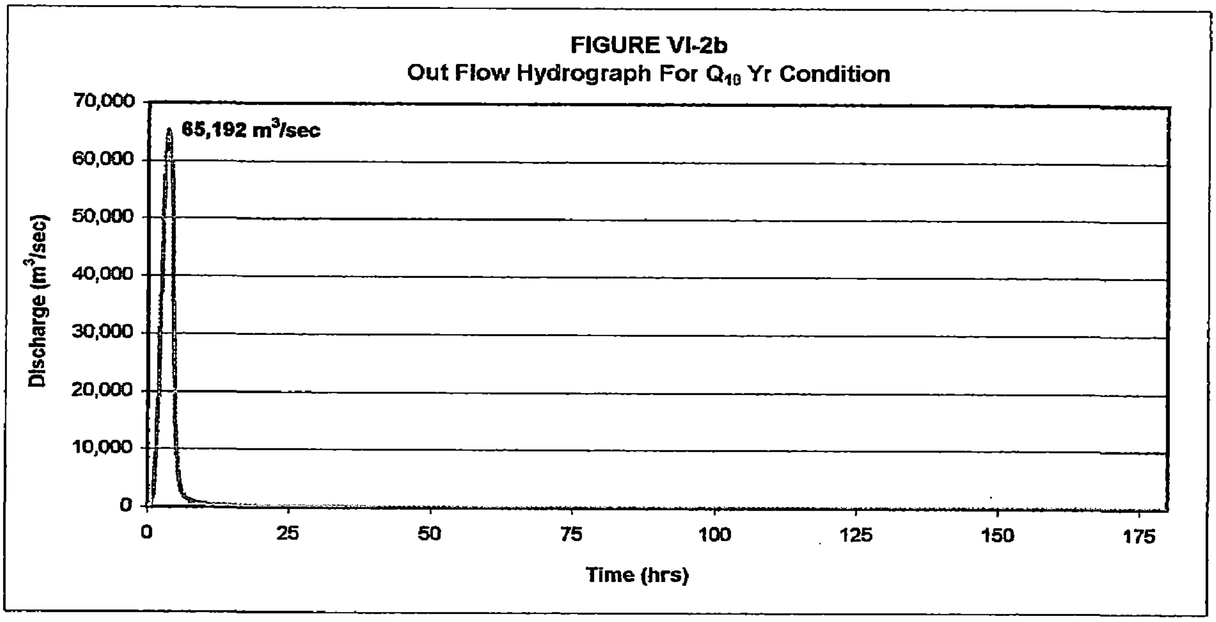


FIGURE VI-1.
OVERTOPPING BREACH FORMATION





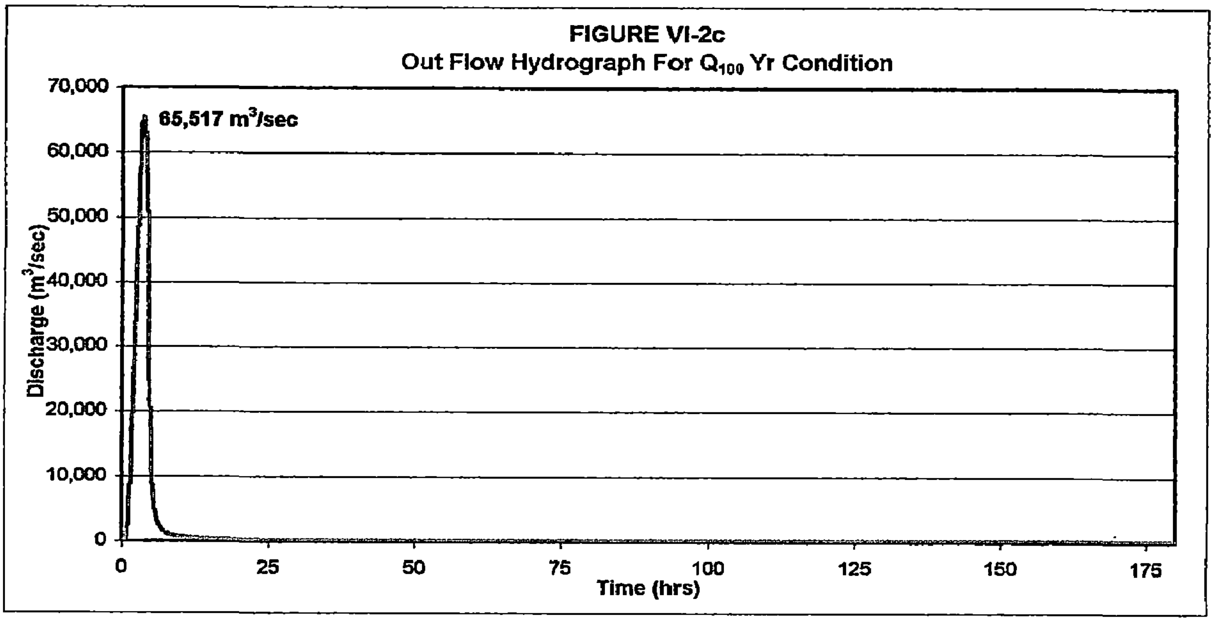


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FIGURE VI-2d.
OUTFLOW HYDROGRAPH FOR Q_{500} Yr CONDITION

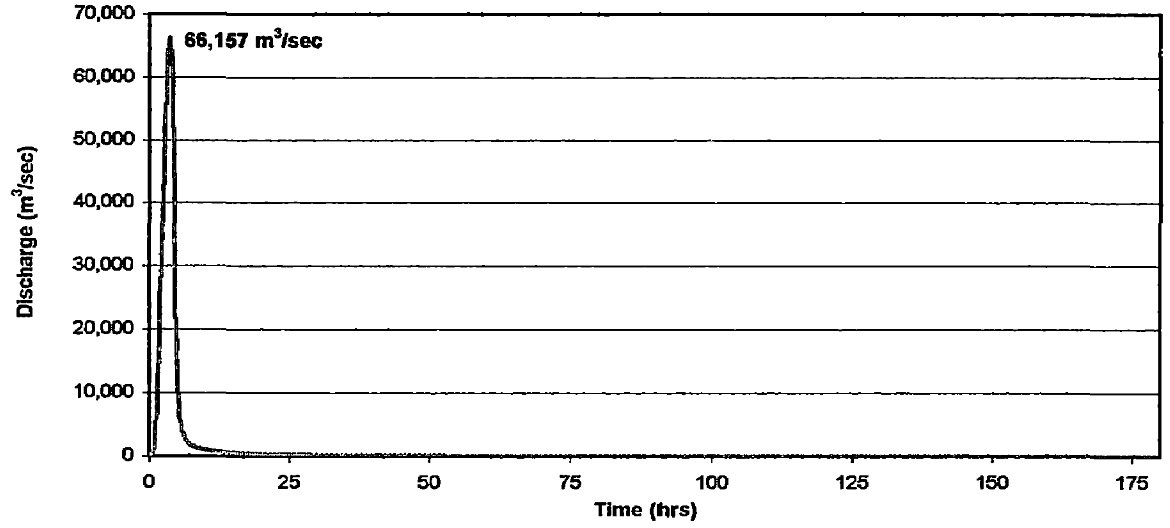
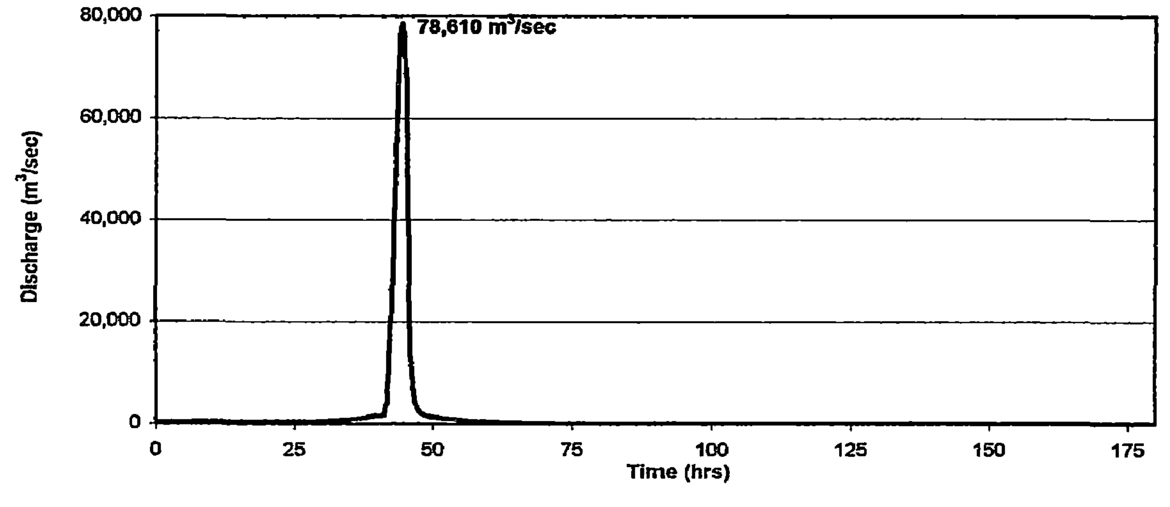


FIGURE VI-2e.
OUTFLOW HYDROGRAPH FOR Q_{PMF} CONDITION



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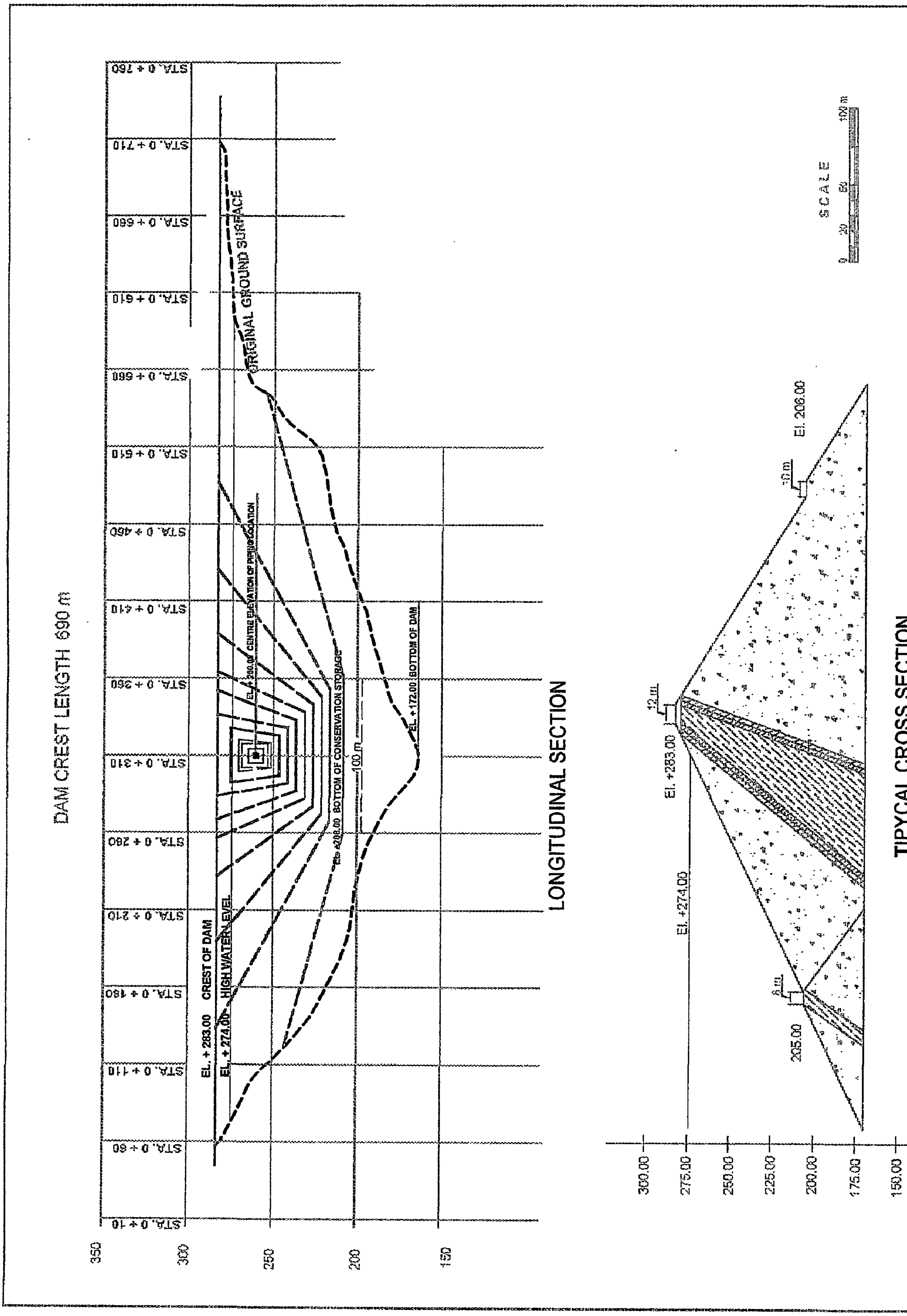
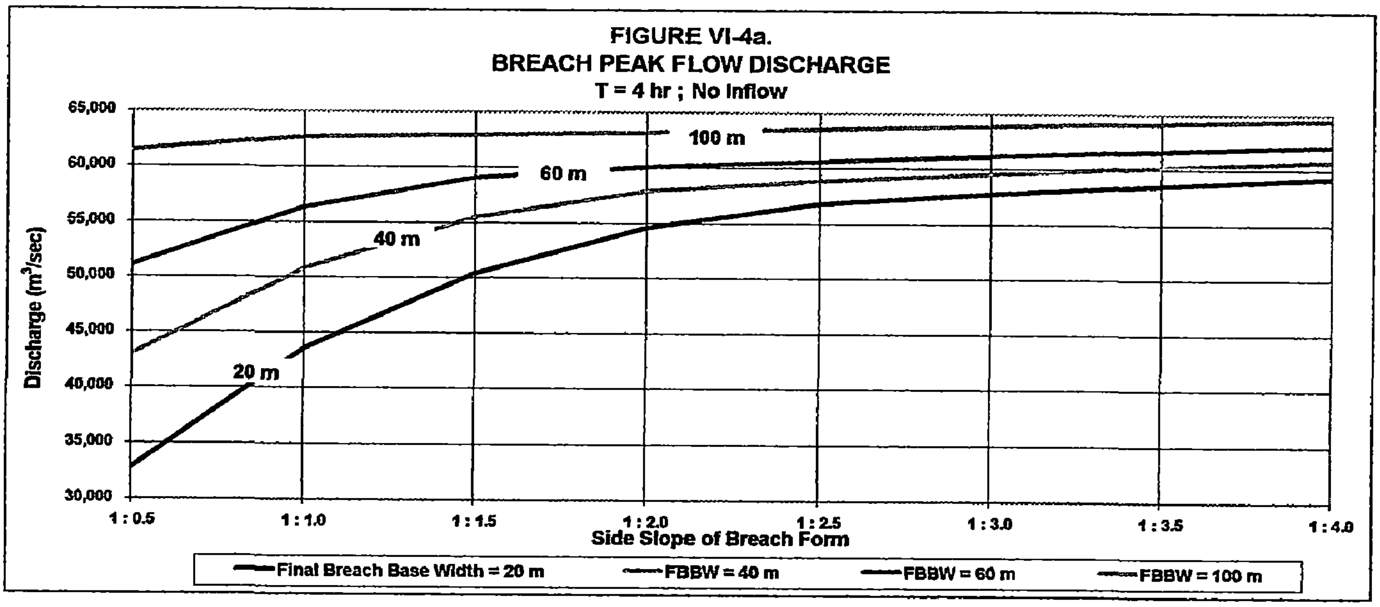
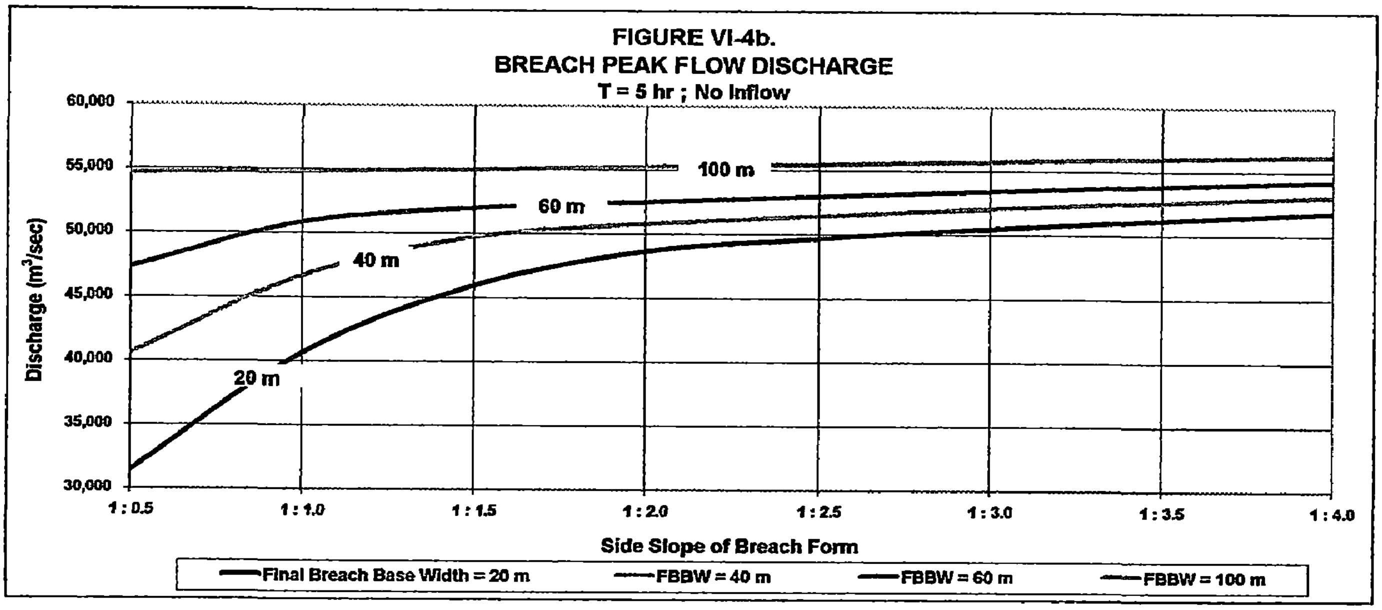


FIGURE VI-3.
PIPING BREACH FORMATION





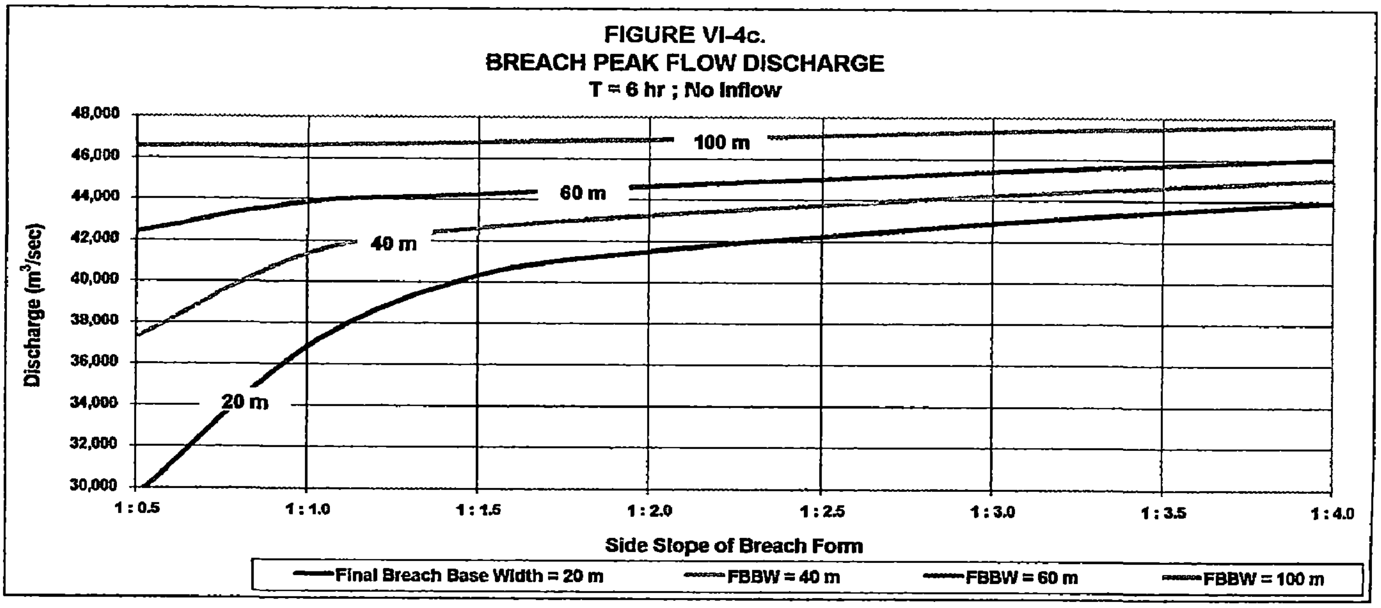


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FIGURE VI-5a.
BREACH PEAK DISCHARGE
T = 4 hr ; Q_{10} Applied

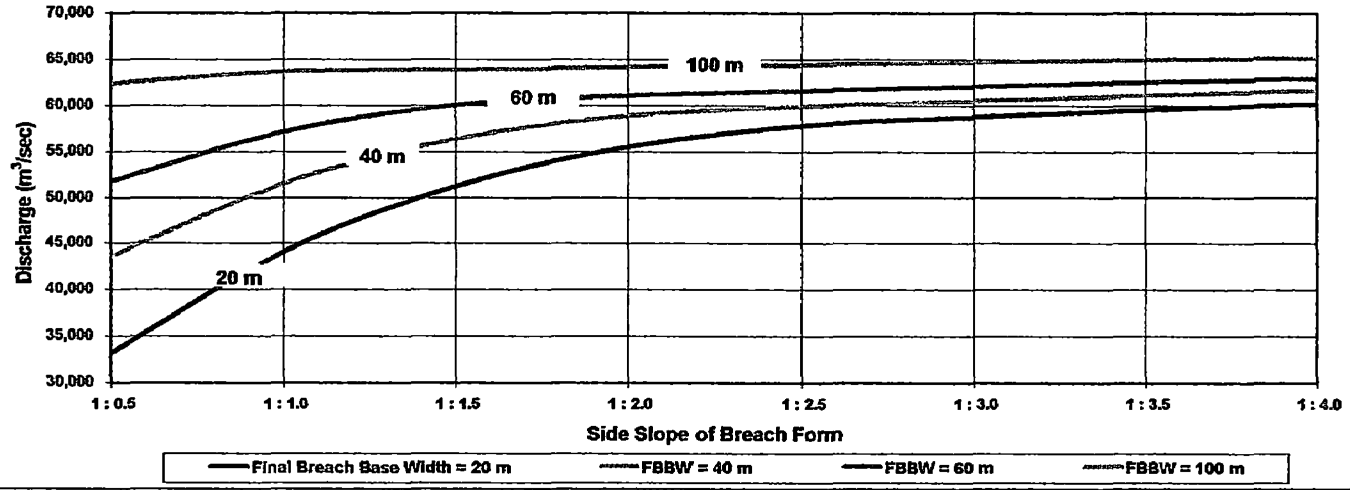


FIGURE VI-5b.
BREACH PEAK DISCHARGE
T = 5 hr ; Q_{10} Applied

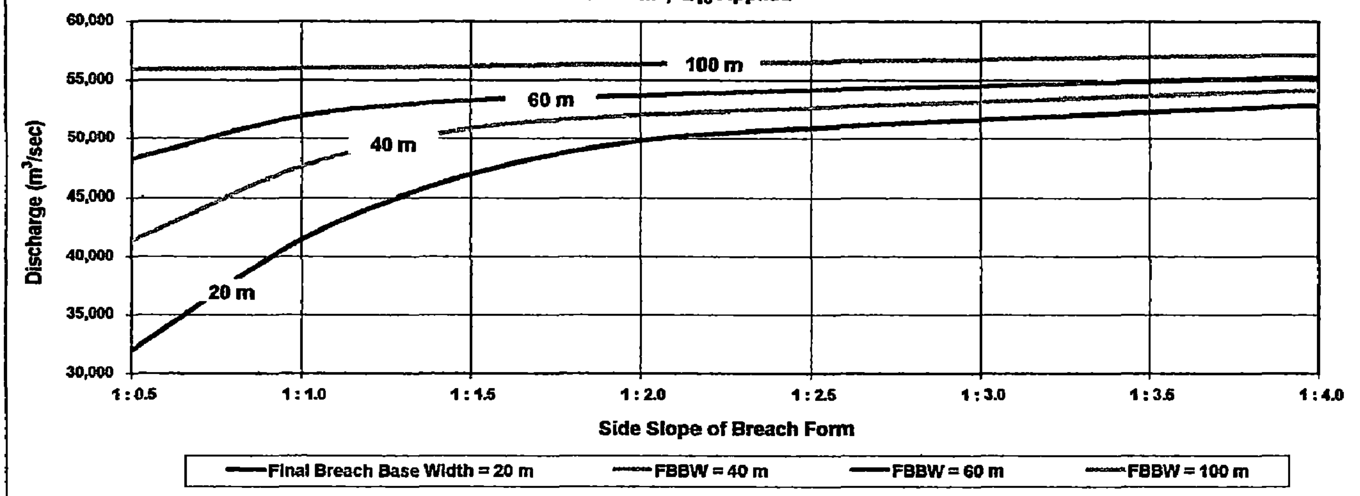
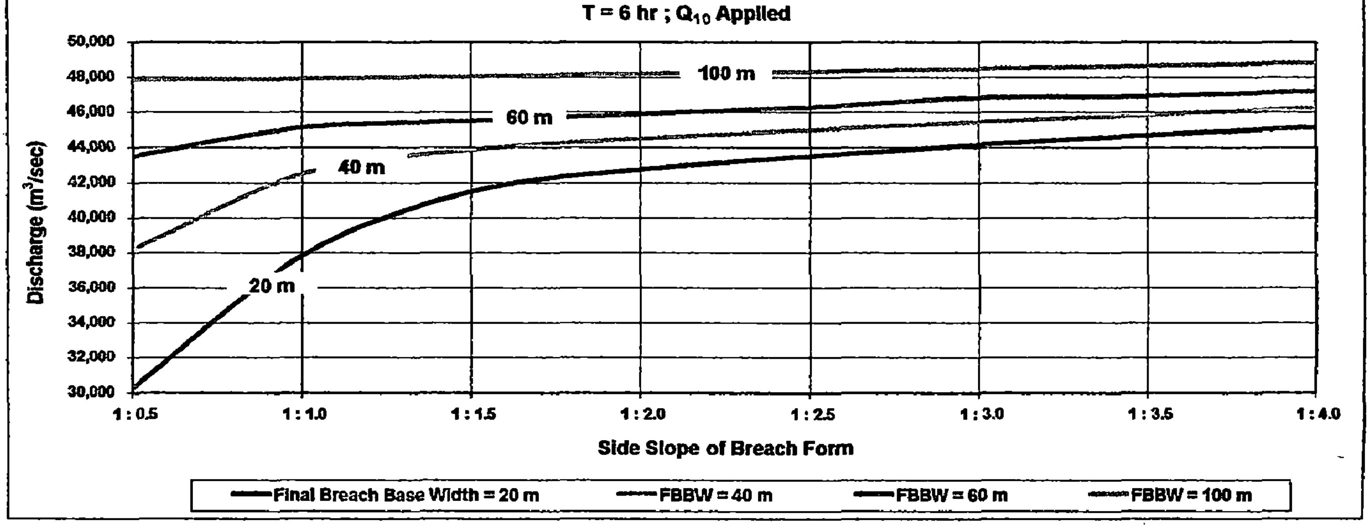


FIGURE VI-5c.
BREACH PEAK DISCHARGE
T = 6 hr ; Q_{10} Applied



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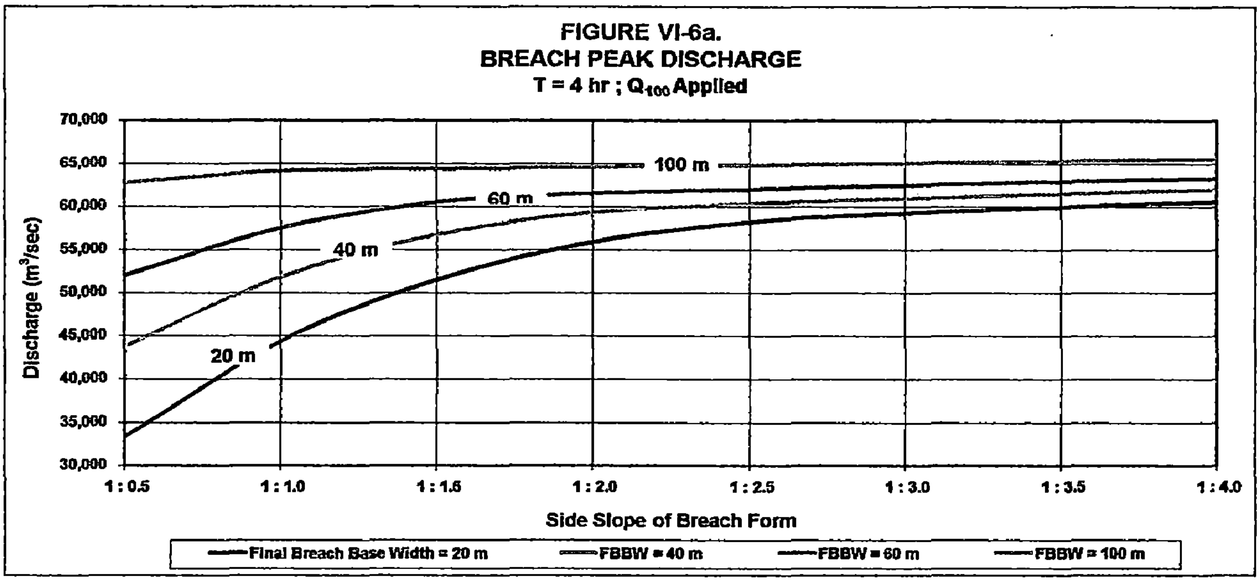

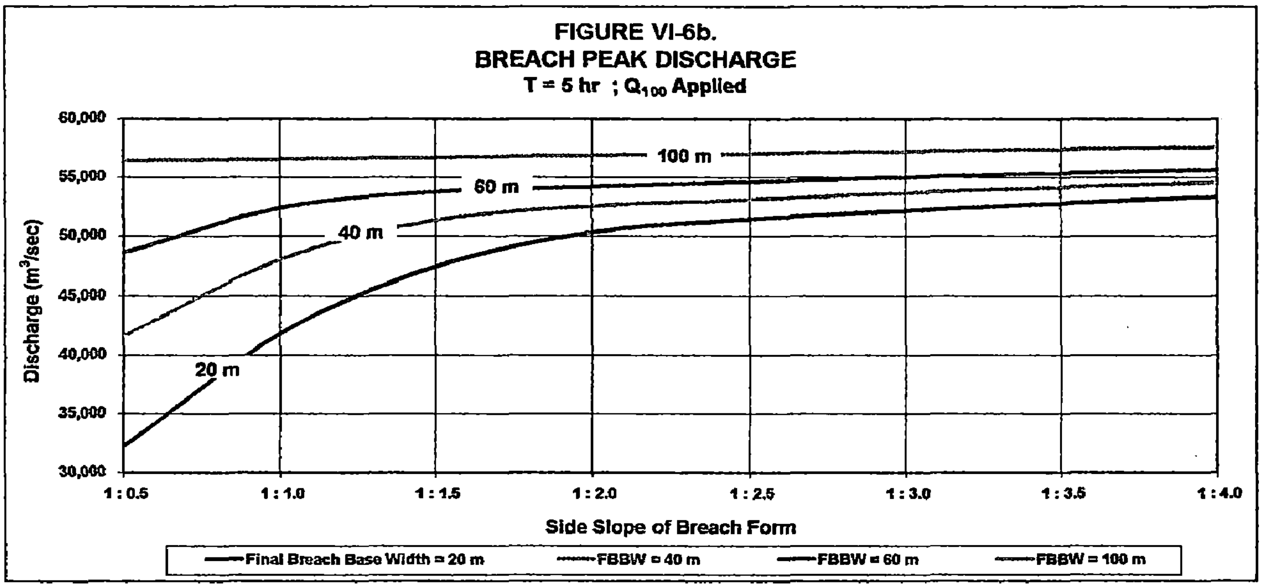

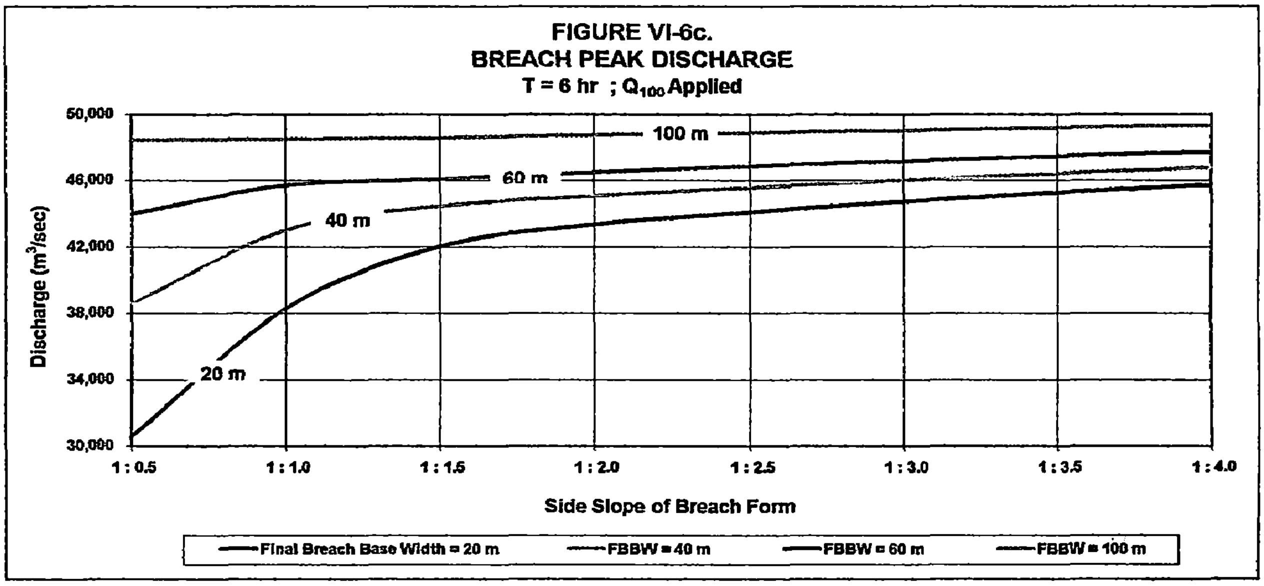

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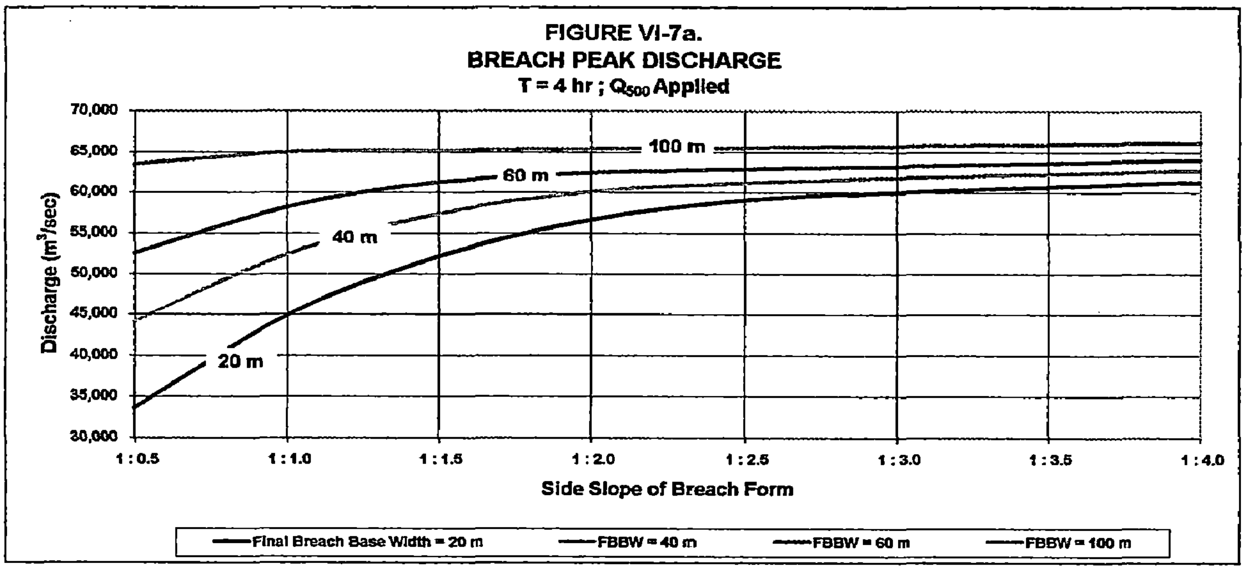

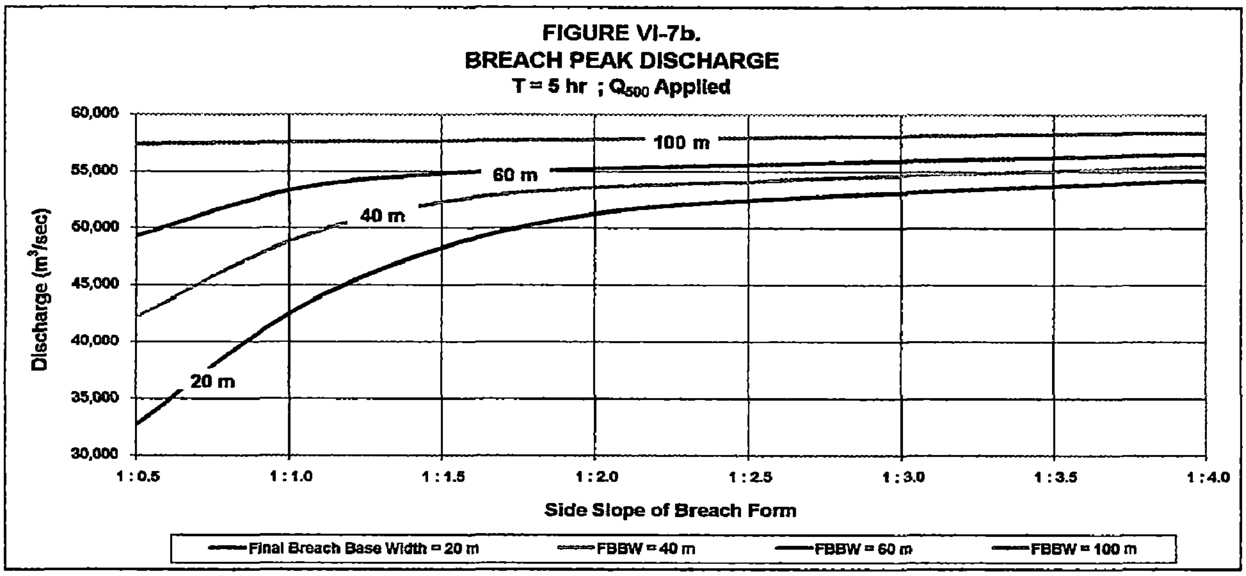

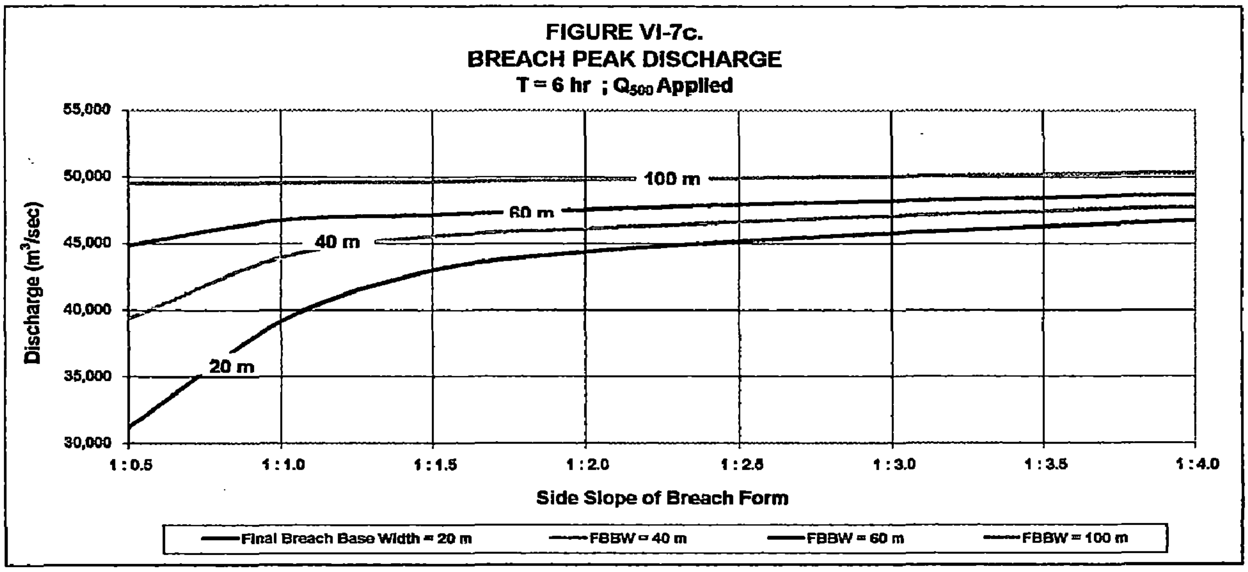

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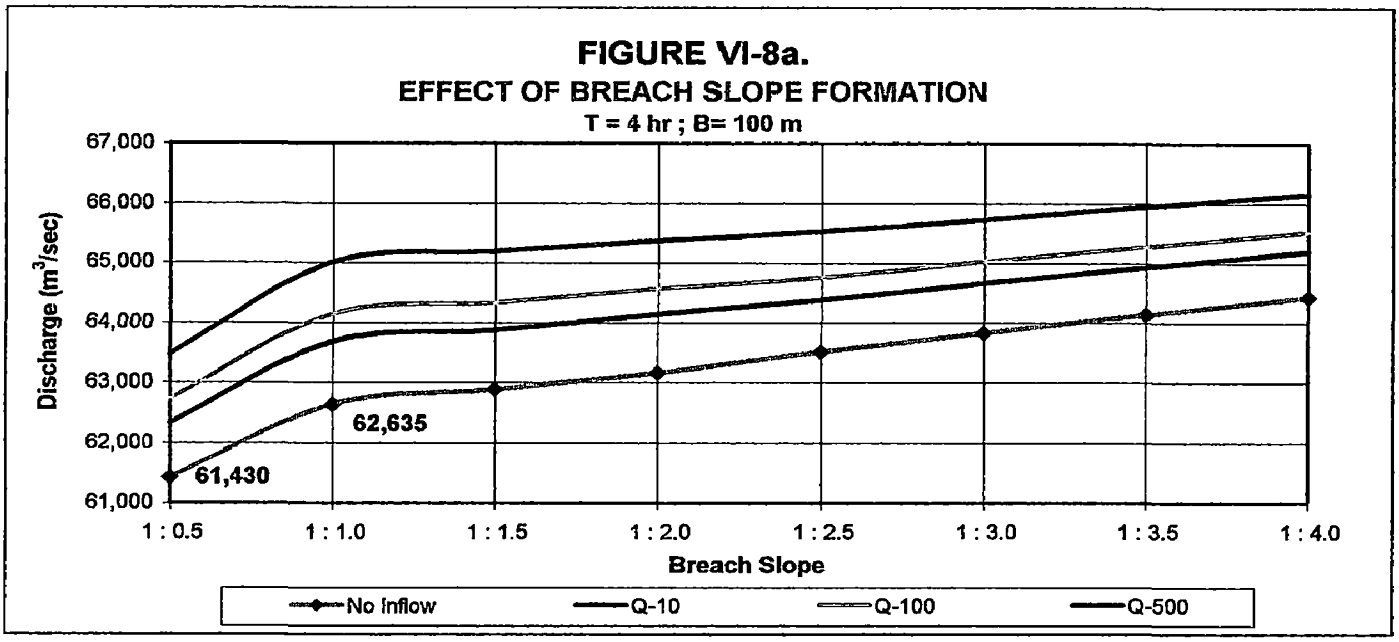

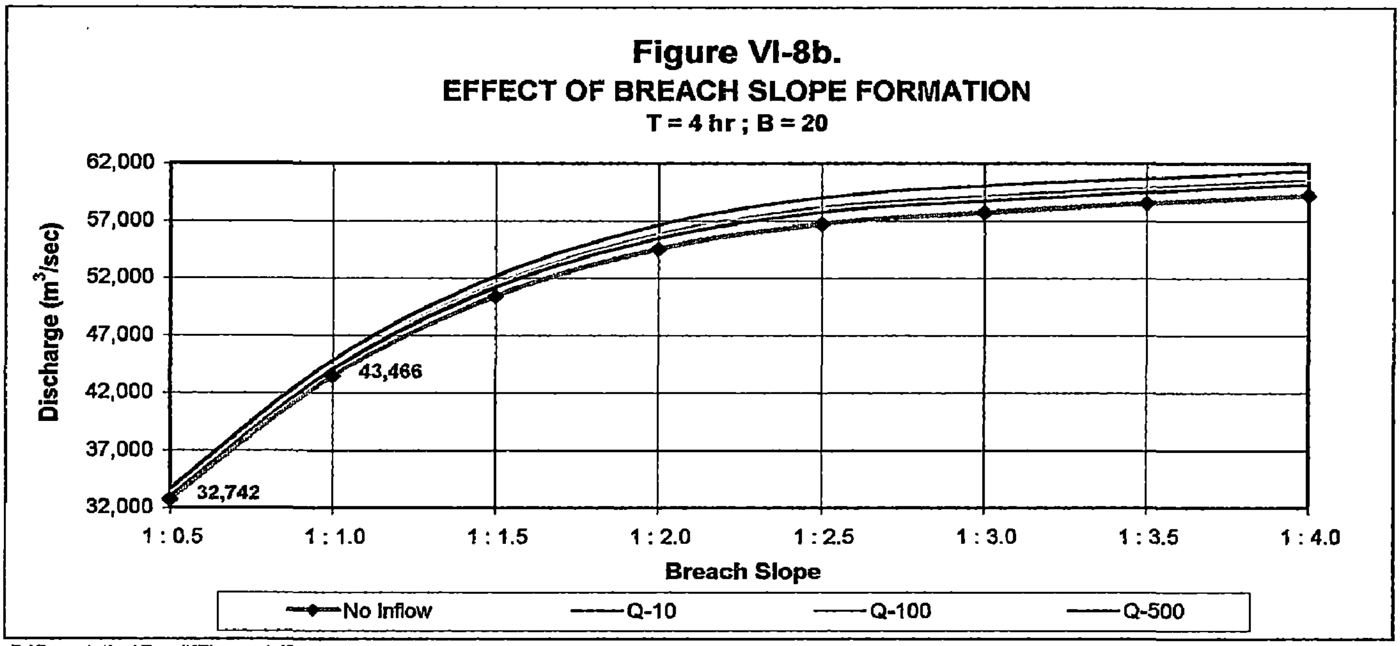

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FIGURE VI-9a.
EFFECT OF FINAL BREACH WIDTH
S = 1 : 4

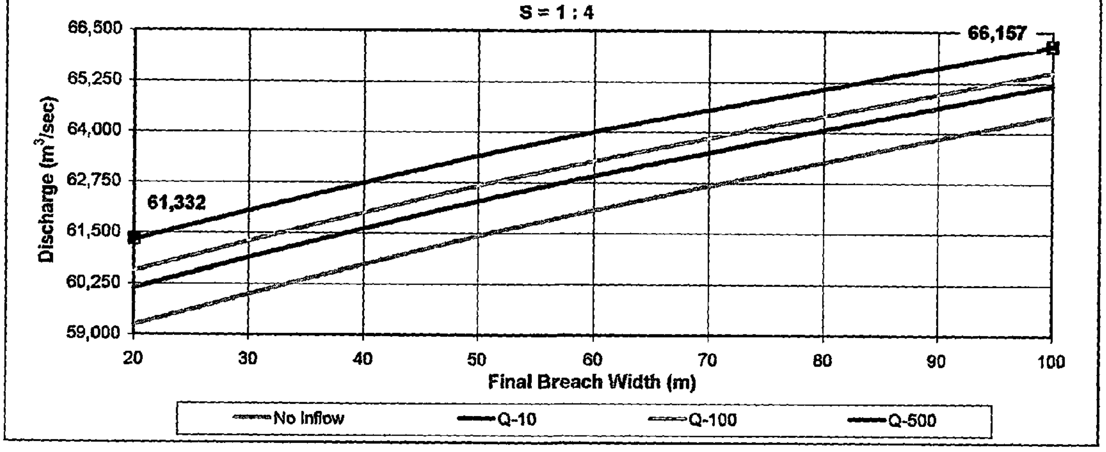
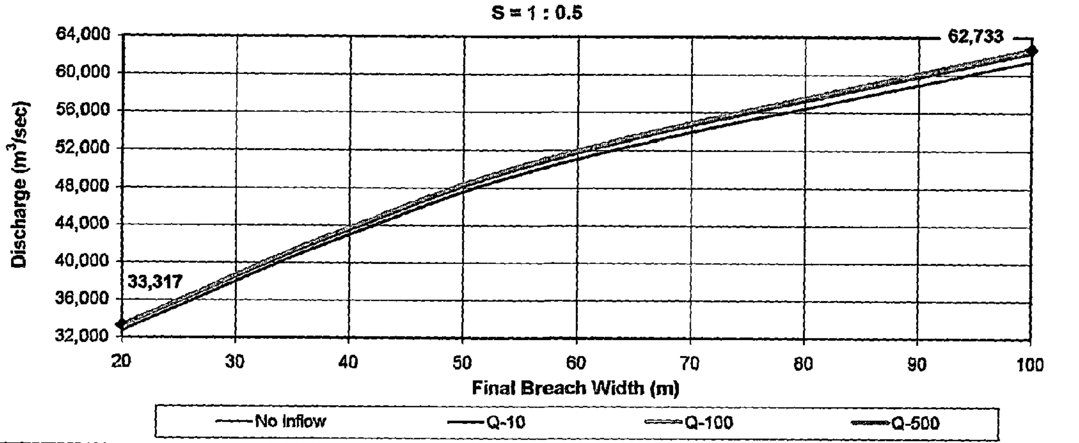


FIGURE VI-9b
EFFECT OF FINAL BREACH WIDTH
S = 1 : 0.5



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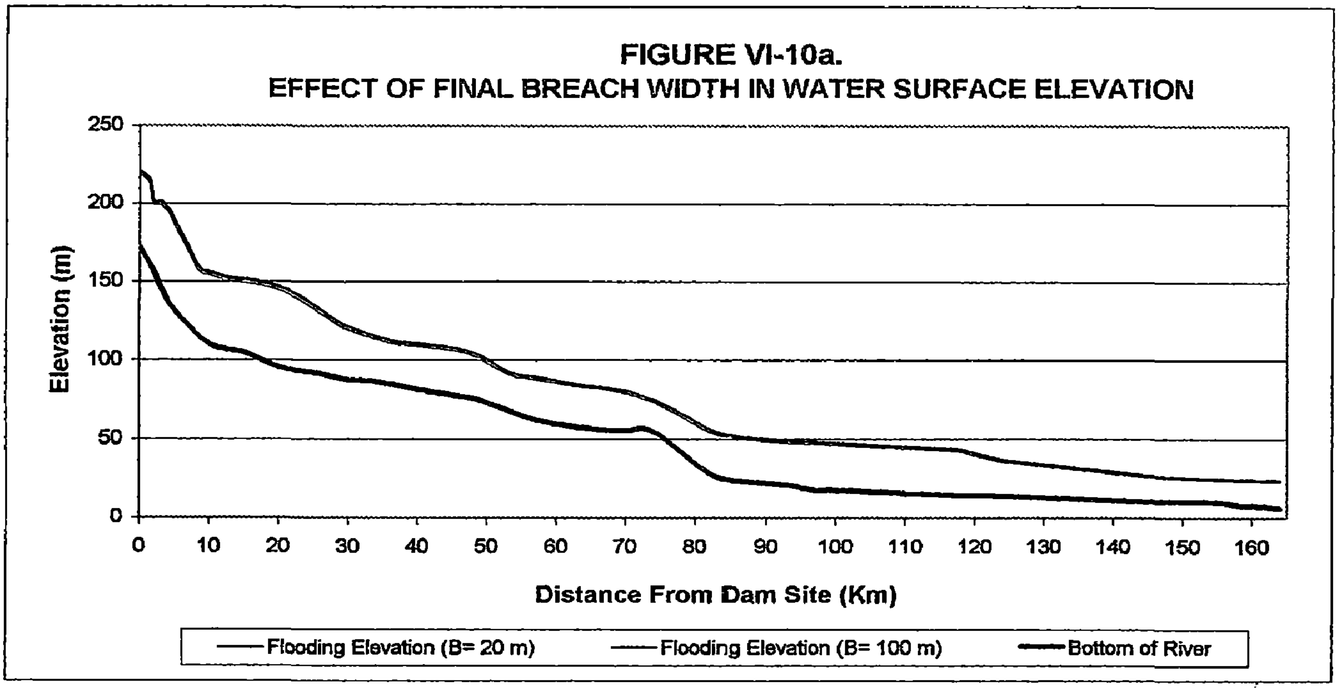

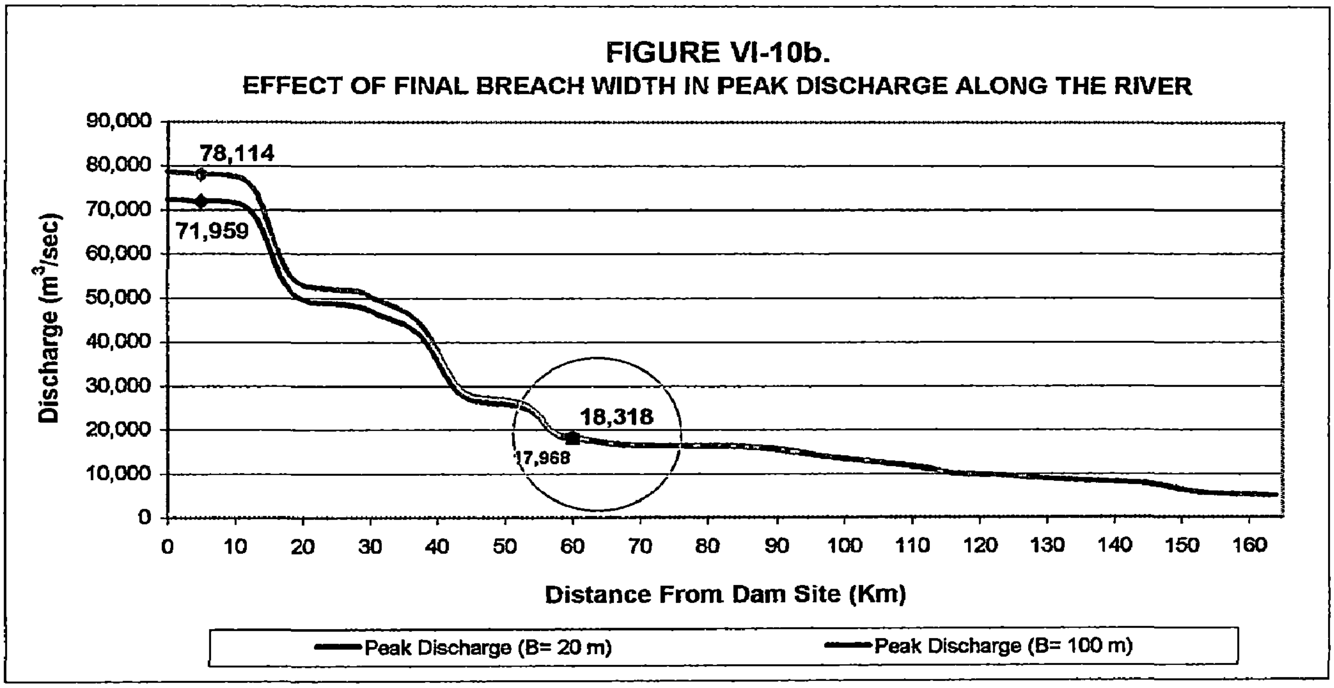

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FIGURE VI - 11.
EFFECT OF TIME DEVELOPMENT BREACH FORMATION

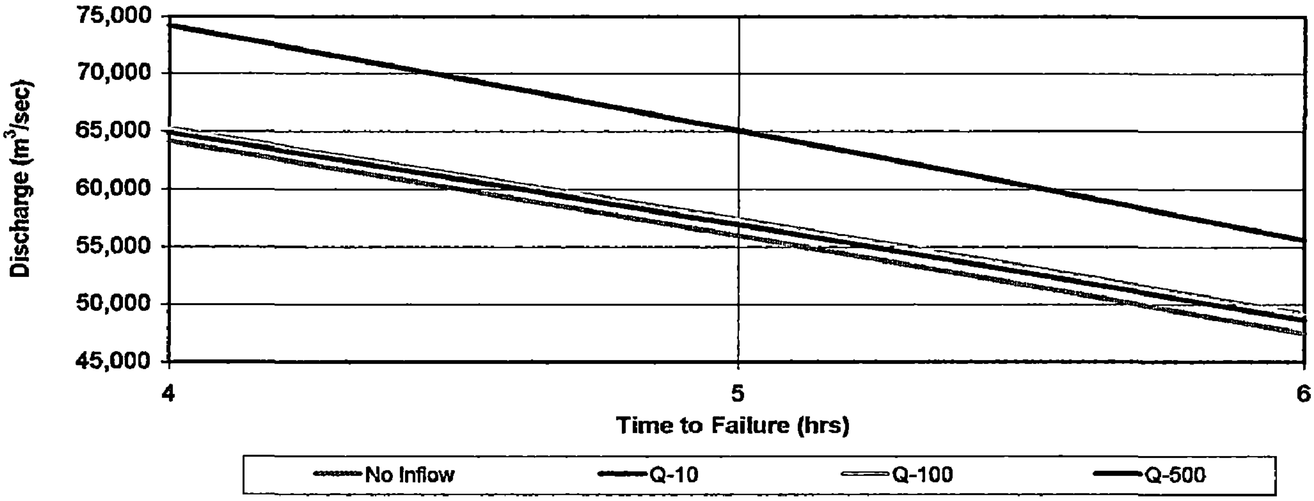

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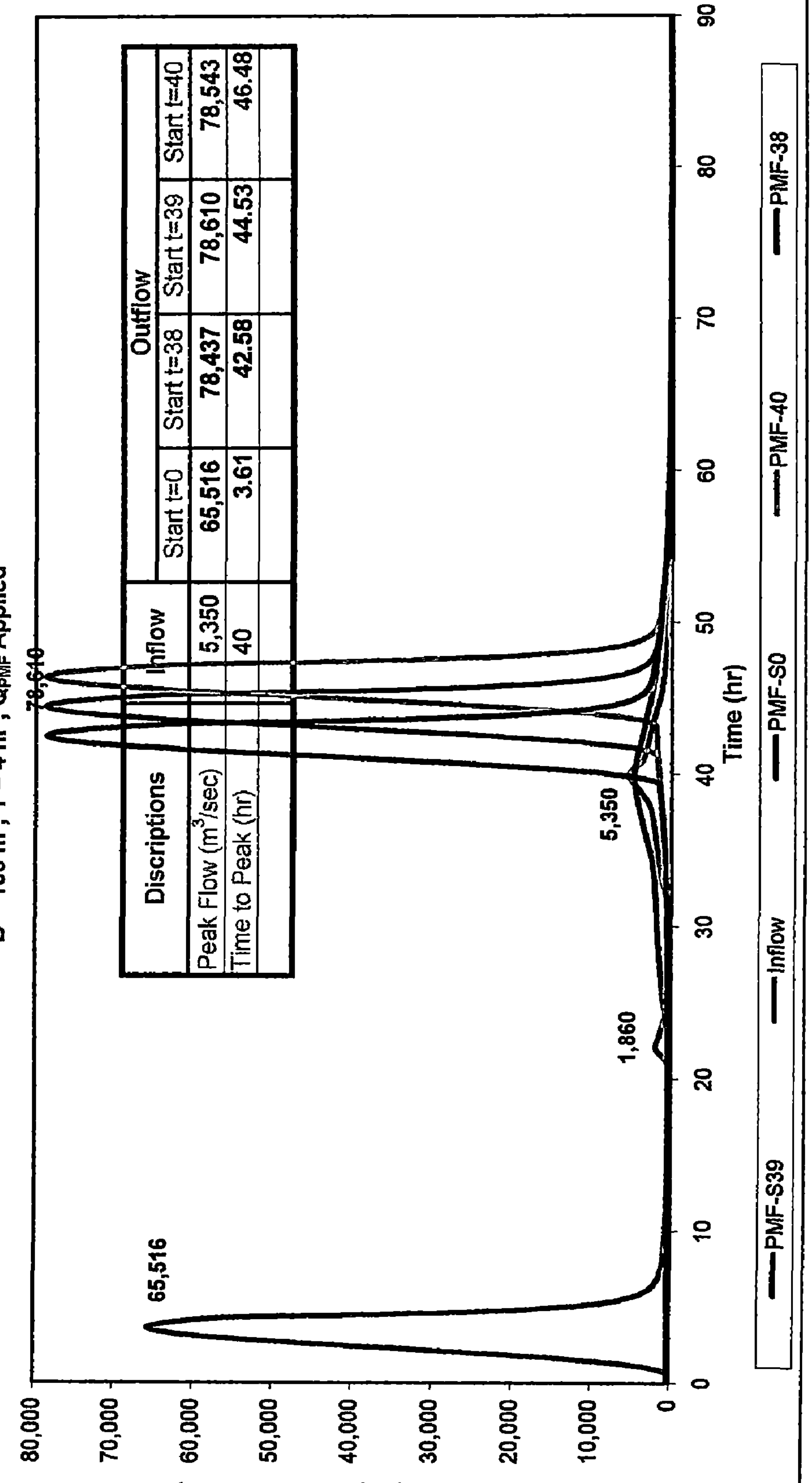
FIGURE VI-12.
SUMMARY OF PMF OUTFLOW FLOOD HYDROGRAPH
B = 100 m ; T = 4 hr ; Q_{PMF} Applied



Discriptions	Inflow	Outflow			
		Start t=0	Start t=38	Start t=39	Start t=40
Peak Flow (m^3/sec)	5,350	65,516	78,437	78,610	78,543
Time to Peak (hr)	40	3.61	42.58	44.53	46.48

FIGURE VI-13a.
PEAK FLOW Vs TIME TO DAM FAILURE
B = 100 m ; T = 4 hr ; QPMF Applied

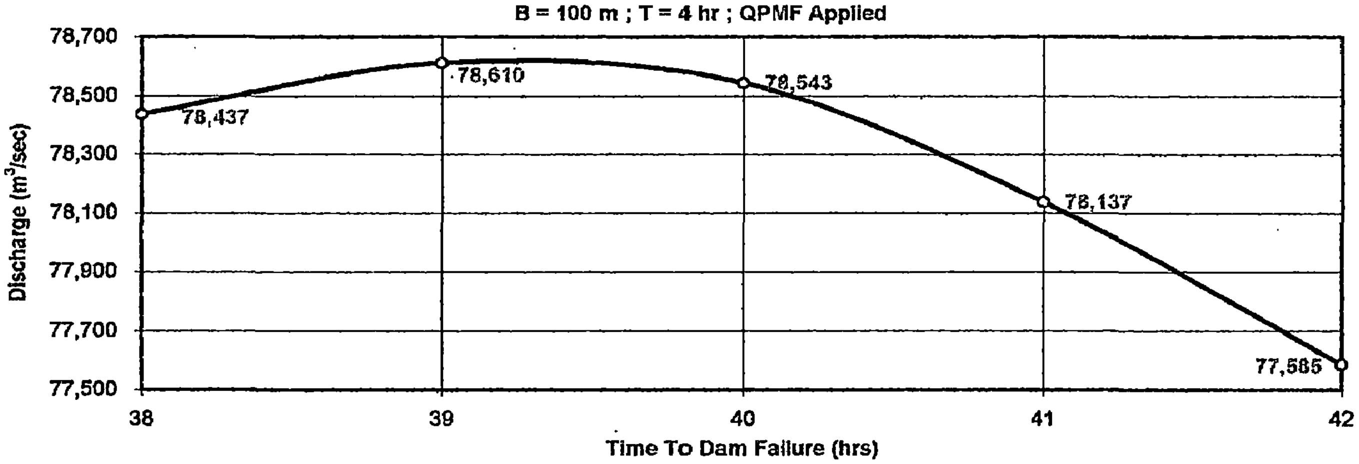


FIGURE VI-13b
PEAK FLOW Vs TIME TO DAM FAILURE
B = 100 m ; T = 5 hr ; QPMF Applied

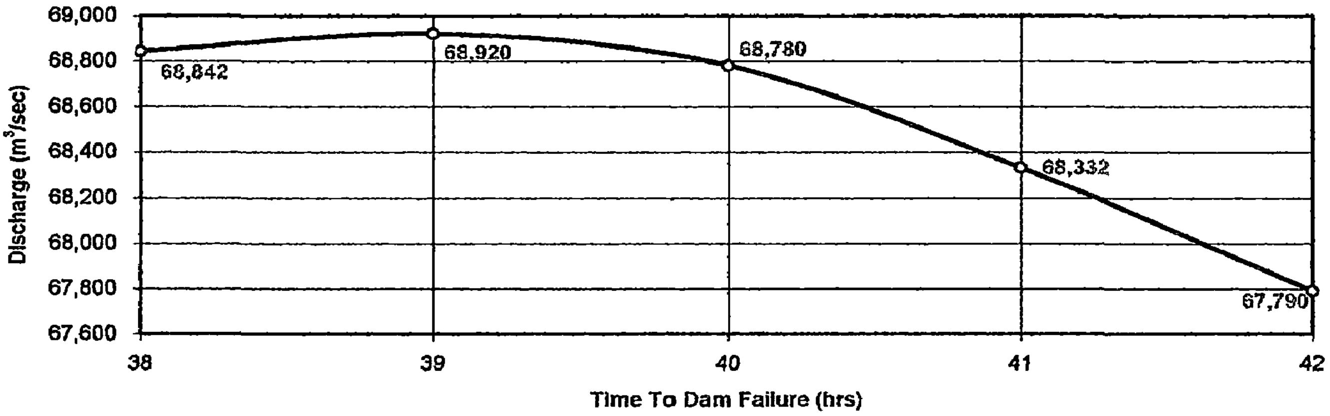


FIGURE VI-13c
PEAK FLOW Vs TIME TO DAM FAILURE
B = 100 m ; T =6 hr ; QPMF Applied

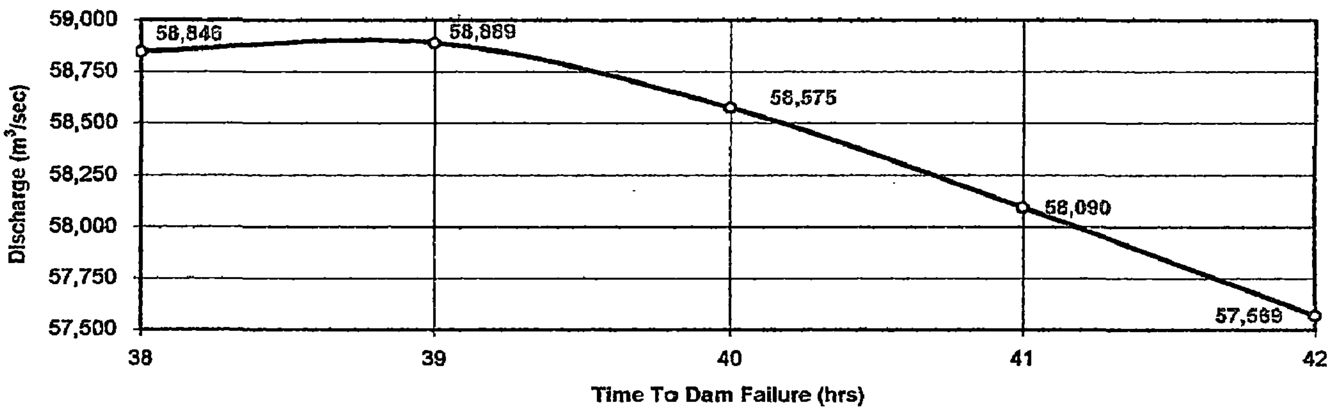


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FIGURE VI-14.
PEAK DISCHARGE ALONG THE RIVER CROSS SECTION

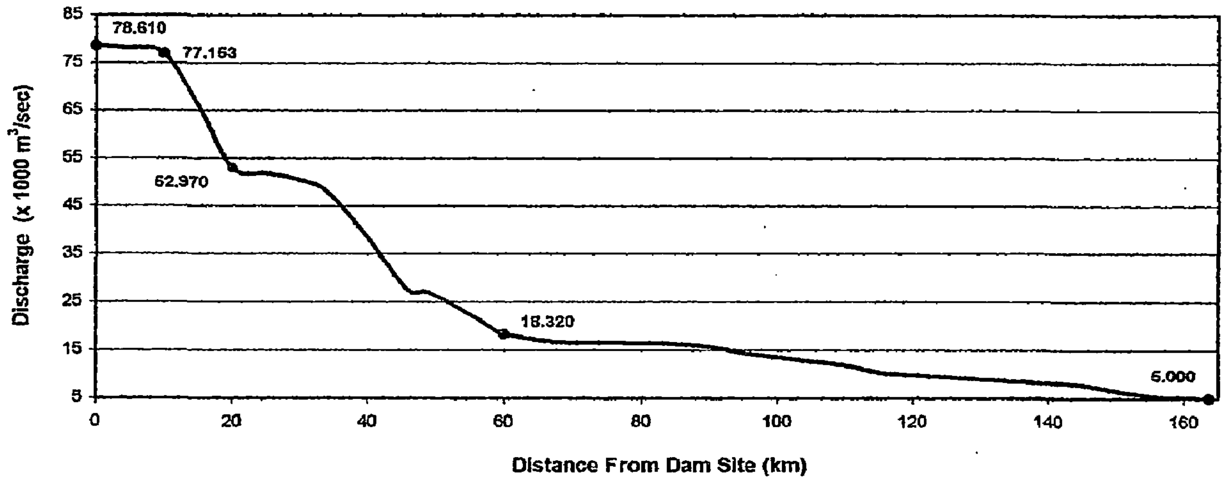


FIGURE VI-15.
TIME TO PEAK DISCHARGE

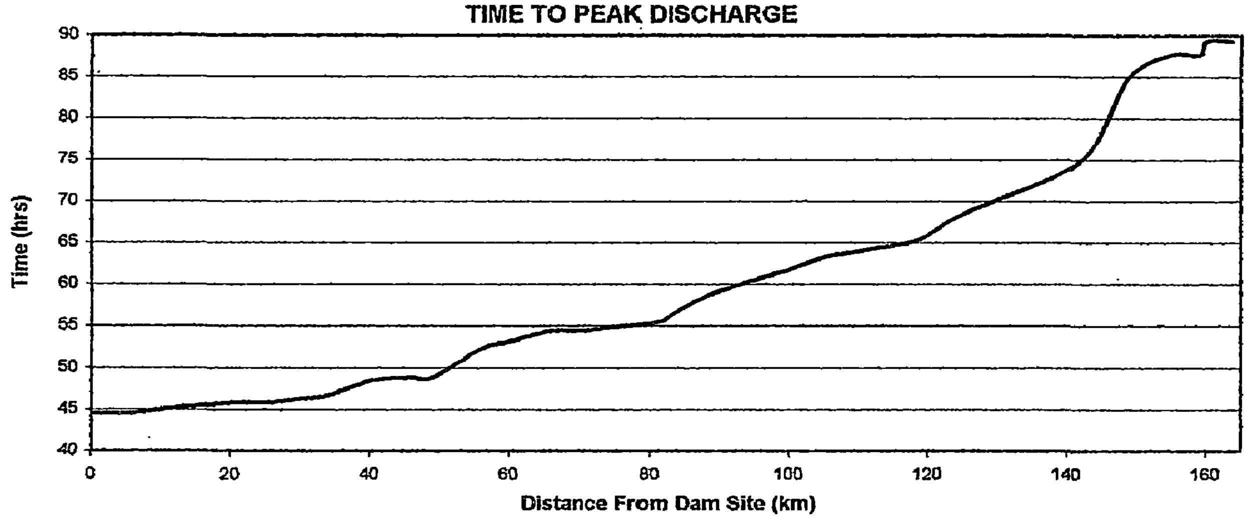
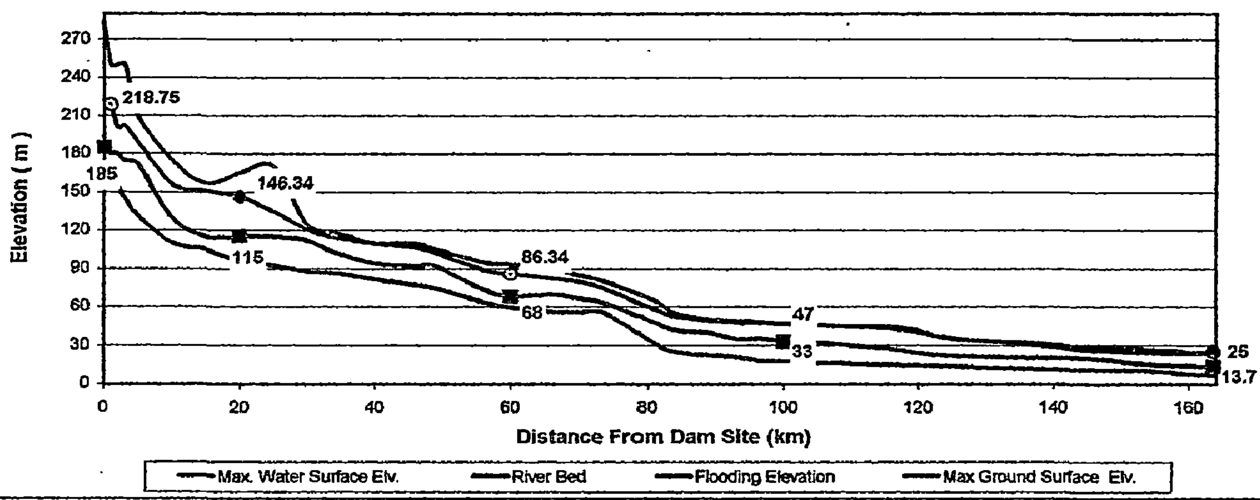


FIGURE VI-16.
MAXIMUM WATER SURFACE ELEVATION



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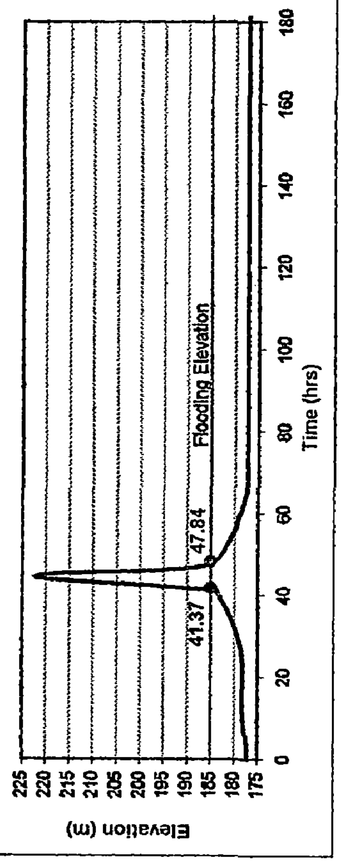

FIGURE VI-17a.
WATER SURFACE ELEVATION AT STA. 0.300 Km

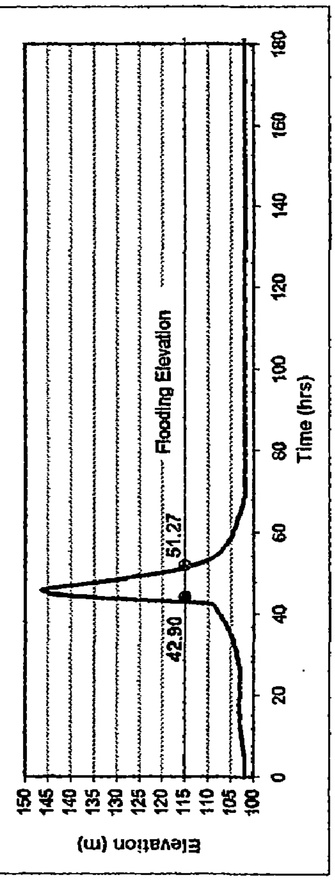

FIGURE VI-17b.
WATER SURFACE ELEVATION AT STA. 20 Km

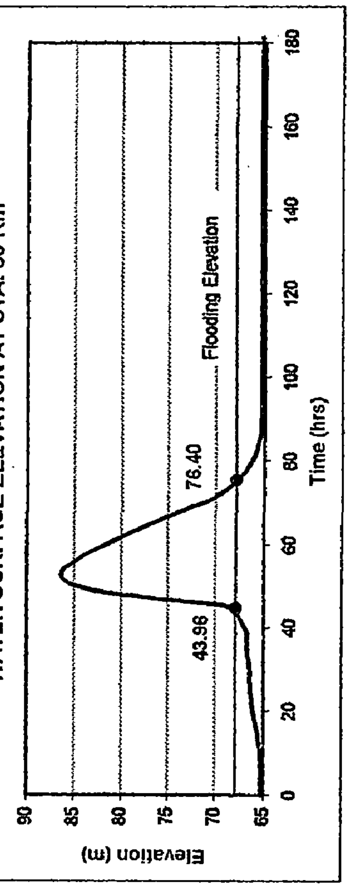

FIGURE VI-17c.
WATER SURFACE ELEVATION AT STA. 60 Km

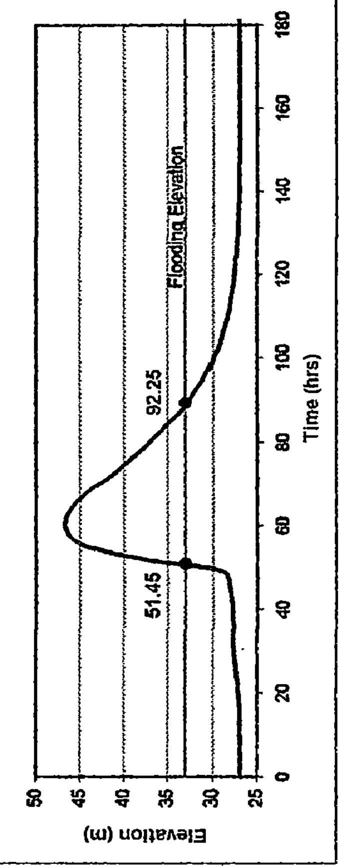

FIGURE VI-17d.
WATER SURFACE ELEVATION AT STA. 100 Km

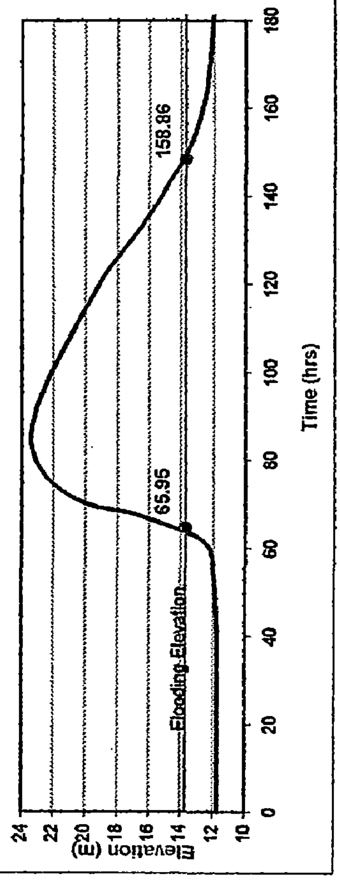

FIGURE VI-17e.
WATER SURFACE ELEVATION AT STA. 163.650 Km

CHAPTER VII
CONCLUSIONS

7.1. CONCLUSIONS

This study simulates the dam break flood wave propagation due to a hypothetical failure of Batutegi Dam in Lampung Province Indonesia, using DAMBRK Model. Dam break flood modeling is subject to several simplified assumptions. The sensitivity analysis has been done to minimize the impact of uncertain breach parameters. Result of sensitivity analysis shows the effect of each of breach parameters.

An effort has also been made to simulate various flood inflow hydrographs to the reservoir. From various studies, it is seen that changes in inflow hydrograph to the reservoir have insignificant/slight effect on the flood wave characteristics. It may be because of large surface area and storage capacity of the reservoir when compared to the total inflow to reservoir.

The results of the study are summarized as below :

1. Two scenarios of dam failure are applied. First is dam failure due to overtopping and second dam failure due to piping. The results of both scenarios show that dam failure due to overtopping has greater adverse effect on the down stream area than the piping failure. As is evident from results, the peak discharge through the breach formation is 33,140 m^3/sec for 10 years and 33,647 m^3/sec for 500 years return period of flood inflow hydrograph (refer to **Table VI-2a** and **Table VI-4a**).

2. Flood stage and peak discharge values are increasing along the river, when related to the increase in the breach parameters except the time taken in breach formation.

3. On the peak discharge through breach formation, breach width has a significant effect combined with the breach slope. The significant effect occurs only at cross section close to the dam site (see **Figure VI-10b**).

4. An increase in the value of side slopes has an increasing on the peak discharge (see **Figure VI-8a** and **Figure IV-8b.**).

5. Time taken in breach formation has an inverse effect on peak discharge. An increase in the time taken in breach formation reduces the peak discharge through the breach formation (see **Figure VI-11**).

6. Based on results of breach parameter sensitivity analysis, selected values of breach parameters are used to simulate the PMF inflow hydrograph to the Batutegi reservoir. The result shows the maximum peak discharge just below dam site as 78,610 m^3/sec and at the end of available cross section (about 163.650 km) downstream of dam only 5,000 m^3/sec (refer to **Table VI-8** and **Figure VI-14**).

7. According to the flood surface elevation along the river down stream of dam site, in most of the river reaches the water levels are more than the bank flooding levels. This indicates that at many locations, the river flood wave is over the bank (refer to **Table VI-8** and **Figure VI-16**).

8. The arrival time of peak flood stage at different locations along the river is accounted from the start of the dam break. It takes about 44.83 hours for the peak stage from the dam site the travel to the downstream around 163.650 km (refer to **Table VI-8**).

9. The time duration of flooding over the flood plain area along the downstream of dam site varies. The duration time varies from 3.2 hr to the maximum of 100.96 hr. The long duration of inundation time occurs in flat reaches of the river (refer to **Table VI-8**).

7.2. SCOPE FOR FURTHER STUDIES

The results obtained by this study may be used for the purpose of flood warning and flood inundation mapping. In this study breach parameters have been analyzed but another source of uncertainty such as river valley hydraulic parameters not analyse yet. The river valley hydraulic parameters cause several errors in dam break analysis. The range of variation in discharge, water depth and time to peak stage resulting from river parameter uncertainty should be examined in detail for a more reliable dam break flood simulation and contingency evacuation planing.

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