Simulation of Dam Break Hydraulics in Natural Flood Plain Topography

A Thesis
Submitted in Partial Fulfillment of the Requirements for
The Degree of
Doctor of Philosophy

By:

Mimi Das Saikia

Department of Civil Engineering
Indian Institute of Technology Guwahati
Assam, India

July 2007

Dedicated to My

Parents and Husband

Acknowledgements

Author is highly pleased to express her profound gratitude and sincere thanks to **Dr. Arup Kr. Sarma**, **Professor, Department of Civil Engineering**, Indian Institute of Technology-Guwahati, for his invaluable guidance and full hand co-operation throughout the all aspects of this work, which made author's inquisitiveness a lot more easy. I also admire his patient explanation of concepts and basic principles.

I am grateful to **Dr. A Dutta, Head of Civil engineering** for necessary facilities and support.

I am indebted to **Dr. S Talukdar, Professor, Department of Civil Engineering, Dr Anoop Das Professor, Department of Mechanical Engineering and Dr G Braua, Associate Professor Department of Civil Engineering** for their valuable help and suggestions.

Author gratefully acknowledges the help **of C.V. Bellos** for his experimental data for the required numerical comparison in this study.

Thanks are also extended to **Dr. Faye Hicks, Professor Department of Civil & Environmental Engineering University of Alberta Edmonton, Alberta, Canada** for forwarding experimental data of C.V. Bellos.

Author acknowledges the help and logistic support extended by **National Hydro Power Corporation (NHPC) and National Productivity Council (NPC)** during the field investigation. It would not have been possible to acquired field data without the support of NHPC, INDIA.

Author gratefully acknowledges the help **Professor D.I.H Barr (PhD, Dsc) Retired Professor, Civil Engineering Department (Water resources engineering) University of Strathclyde, Glasgow,UK** and **Dr. M M Das Retd. Director, Technical Education, Assam and Emeritus Fellow AICTE,** India, for their experimental data for the required numerical comparison in this study

Author gratefully acknowledges help of **Sandra Soares, Université Catholique de Louvain, Belgium , editor CADAM proceedings ,**for providing the valuable data of Malpasset dam, the CADAM' s real dam break test case and **the bench mark problem for real dam break** in complex natural topography.

I express my sincere thanks to the **staff members of the civil engineering Department**, IIT Guwahati for their Kind cooperation during my work.

Author takes the opportunity to convey sincere thanks and gratitude to all her friends specially **Maya**, well-wishers and family members specially **my Parents, my Mother in law** and **Brother Bhargab**.

Finally, I have great pleasure to convey sincere thanks and gratitude to **my Husband L.P. Saikia** who took the pain of completion of my research work at IIT Guwahati.

Mimi Das Saikia

Abstract

Catastrophic floods caused by dam failures always lead to a great amount of property damage and even loss of human and wild life. Several dams are proposed in North East region of India, to harness the existing hydropower to facilitate electricity, irrigation, water supply, navigation, recreation and to achieve flood and erosion control. While multi-objective dam can provide immense benefit, its failure or improper operation may lead to devastating flood downstream. One of the major challenges to overcome for economic development of North East India is the management and planning against such devastating flood. This damage will be more destructive when a dam fails suddenly. The outcome of such occurrence will be a great danger to human and wild life causing significant economical and environmental damages. Readiness to face such dam break flood disaster can be attained when prior knowledge of the flood movement is acquired through simulation model. Hence the main aim of the study is to develop mathematical models for simulating dam break hydraulics using different forms of governing equations and adopting different numerical schemes so as to select the best one to simulate the dam break flood in complex real river floodplain.

The simulation models have been used to predict and analyze the flood due to the instantaneous failure of the proposed large dam in the Himalayan River Dibang. Significant variation in bed slope, bed width and resistance characteristic along the channel length are the typical characteristics of Dibang River. The proposed dam is a high concrete gravity dam.

Gradually varied unsteady flow equations for open channel i.e., continuity and momentum equations both in conservative and non-conservative forms have been used for 1-D formulations. 2-D formulation has been done with 2-D continuity and momentum equations in conservative form. The first order diffusive finite difference (F.D.) scheme, second order modified two-step Predictor Corrector F.D. scheme and MacCormack scheme with Total Variation Diminishing (TVD) limiters have been used here for computing flood movement due to failure of the proposed Dibang dam. The 2-D model has been developed using two-step modified predictor corrector scheme.

To assess applicability and validity of the numerical models for sub-critical, super-critical and sub/super critical mixed flow conditions, results obtained by the conservative formulation of the dam break flow with diffusive scheme have been compared with laboratory data. The model has been found to be quite capable of handling sub-critical, super-critical and mixed flow conditions.

In many practical applications, the steep slopes of the natural river terrain may result in strong source terms in the system of governing equations. Hence, to test whether the numerical models developed provide stable flow profiles in any type of Natural river valley, another dam break flood due to failure of the proposed dam in another Indian River Kynsy is also simulated. Considerable variations are there in the source term in different river sections in a particular instant of time as well as in the same section in different time periods. The conservative formulation with consideration of the natural terrain of both the rivers, one in steep mountainous valley and another in a comparatively flat terrain with a channel in a wide floodplain the developed model provides stable flow profiles

The simulated output depends a lot on the terrain or channel assumptions. Therefore, to get insight into the effect of channel characteristic on the flood prediction three 1-D models and one 2-D model have been formulated considering the natural ground terrain of Dibang. The proper prediction of the possible extend of inundation downstream of the dam is quite important, as it consists of villages, roads, dense forest etc. and therefore, for such practical purposes, for proper prediction of the dam break flood, it may be quite realistic and logical to select the computational channel in such a manner that it takes into account the wide floodplain when the river enters the floodplain.

The graphical user interface (GUI) software has been developed in Visual Basic. The visual basic has been used as programming language as it has enabled the author to get graphical output in desired form without any additional graphical package.

Dam-break hydraulics of natural rivers is indeed complicated, and involves not only water flow, but also morphological change in the riverbed. Here a simulation model has been developed for Dibang dam break flood in considering the bed of the channel as mobile. There is a considerable change in the riverbed at the dam site under the highly transient flow condition occurring due to the failure of a large dam. The rate of bed deformation signifies the need for the coupled modeling of the strongly interacting flow-sediment-morphology system under dam break flow.

Comparison of the outputs shows that once the depth of flow crosses the depth of the original river channel, it will start flowing to the nearby floodplain. The assessment of the models developed considering the natural topography of Dibang for dam break flood prediction has been done by using the model to simulate real world dam break benchmark problem. Malpasset dam break (1959 France) has been considered as the only real dam break benchmark problem by CADAM (Concerted Action on DAm-break Modeling) Project (1996-1999). Two sets of data are available; first the real dam break data collected by police at the time of dam break in 1959 at France and second the data available from the physical model which was built with a scale of 1/400 to study the dam-break flow in the laboratory of EDF, France (1964). Data are provided by CADAM so that the models developed here can be assessed. Maximum flood predicted by the developed model and the actual dam break flood level has shown satisfactory agreement. The predictions of the flood depth are slightly higher and peak arrival times are lower in model outputs.

In this study available historical flood data for the state of Assam, India from 1953 to 2005, containing information regarding inundated area, population affected, crop damage and total loss, has been used. Multiple Regressions analysis has been done to develop a generalize damage estimation model, suitable for flood damages estimation in the northeastern region of India. In case of instantaneous failure of Dibang dam considering the predicted area of inundation, the population affected and crop area affected the estimated damage has been found. Considering the high value of estimated damage several mitigation measures have been suggested.

Contents

List of Figures

List of Tables

List of Notation

a	= Position of wave front
aa	= Length of the longest, intermediate, and shortest axis of the particle
A	= Cross-sectional area of flow
$\bar{A}$	= Average cross-sectional flow area
AP	= Predicted Cross sectional area
AC	= Corrected Cross sectional area
b	= Cross-sectional width
bb	= Length of the intermediate axis of the particle
b_m	= Mean cross sectional width(m)
B_{avg}	= Breach width(m)
c_x	= Celerity of wave propagation
cc	= Length of the shortest axis of the particle
c	= Flux-averaged volumetric sediment concentration
$\bar{c}$	= Average wave celerity
C	= Non-dimensional celerity
C_r	= Courant number
d	= Size of sediment in millimeters
D	= Artificial dissipation term
D_d	= Sediment deposition fluxes across the bottom boundary of flow, representing the sediment exchange between the water column and bed
E_e	= Sediment entrainment fluxes across the bottom boundary of flow, representing the sediment exchange between the water column and bed
f	= Failure time
F	= Flux vector
FP	= Predicted flux vector
g	= Acceleration due to gravity

h	= Depth of flow
h_o	= Depth at dam site
h_b	= Height of breach
h_w	= Depth of water above breach at time of failure
i	= Represents node position in x-direction
I_1,I_2	= Cross-sectional moment integrals
k	= Constant of proportionality
K	= Overtopping multiplier
K_s	= Total Manning-Strickler coefficient
$K_s{}'$	= Grain Manning-Strickler coefficient
m	= Exponent
n_m	= Manning's roughness coefficient
n	= Time level
n_e	= Equivalent Manning's for a composite channel section subdivided into N sub areas
n_i	= Manning constant (i =1,2,..,N)
p	= Bed sediment porosity
P_i	= Wetted perimeter
Q	= Discharge
$\bar{Q}$	= Average discharge
Q_p	= Peak flow
q_s	= Sediment discharge rate
$\bar{\mathbf{r}}$	= Approximate Jacobian matrix eigenvectors
S	= Vector containing source terms
S_c	= Corey shape factor
SP	= Vector containing source terms
S_f	= Friction slope
S_0	= Bed slope
S_{ox}	= Bed slope in x direction

S_{oy}	= Bed slope in y direction
T	= TVD term
TN	**= N**on-dimensional time
t	= Time
Δt	= Computation time step
Δt	= Time step
UN	**= N**on-dimensional velocity
UP	= Predicted vector of flow variables
UC	= Corrected vector of flow variables
V	= Depth averaged flow velocity
V_x	= Flow velocity in x direction
V_y	= Flow velocity in y direction
V_w	= Volume of water stored at the time of failure
v	= Velocity of flow
VP	= Predicted velocity
VC	= Corrected velocity
x	= Direction parallel to the river
XN	**= N**on-dimensional distance
z	= Bed elevation
ΔS_{1D}	= Mean bed surface evolution (erosion or deposition) at that particular cross-section
ΔS_{2D}	= Change in surface in the lateral direction
Δx	= Space difference in x-direction
$\Delta Z\ j$ initial	= Change in bottom elevation in point j
$\Delta Z\ j$ final	= Final evaluation of the channel surface at any point j at a cross-section
α	= Weighting coefficient in diffusive scheme
β	= Weighting coefficient in Lerat and Peyret's family of Lax-

	Wendroff schemes
δw	= Characteristic variables
η	= Integration variable representing vertical distance to bottom of section
$\theta\theta$	= Weighting coefficient in Lerat and Peyret's familyof Lax-Wendroff schemes
$\bar{\lambda}$	= Approximate Jacobian matrix eigenvalues
ρ	= Ratio on which limiters F depend
τ	= Ratio $\frac{\Delta t}{\Delta x}$
ϕ	= Limiter allowing TVD conditions
ρ	= Density of water-sediment mixture
ρ_o	= Density of saturated bed
ρ_w	= Density of water
ρ_s	= Density of sediment
ω_o	= Settling velocity
$\alpha\alpha$	= Coefficient=min [2, (1-p)/c]
αc	= Near bed concentration of sediment which is assumed to be proportional to the depth-averaged concentration
θ	= Shields parameter
θ_c	= Critical Shields parameter
ν	= Kinematic viscosity of water
ζ	= Roughness parameter
τ_j^*	= Shear stress in point j obtained by M.P.M.
τ_{jc}^*	= Critical shear stress in point j

Chapter-1

Introduction

1.1 Introduction

The catastrophic floods of extreme magnitude in river valleys have been occurring around the world due to sudden failure of dam caused by piping, foundation failure, seismic load, overtopping, erosion etc. Similar sudden devastating floods also occur owing to failure of river dike, sudden huge landslide over a big reservoir, breaching of embankment or flood wall by sea tides, sudden release of water from a reservoir due to erroneous operation of reservoir. These floods of extreme magnitude in all the above situations are known as dam-break floods in the field of water resources engineering. This dam-break flood is highly unsteady associated with events of extreme magnitude since it starts to release a huge wall of water through the breach with such velocity that the valley below the barrier becomes a placid lake instantaneously. The treatment of the hydraulic aspects of this unsteady flood flow problem is identified as hydraulics of dam-break flood analysis. These catastrophic floods caused by dam failures always lead to a great amount of property damage like buildings, crop land, roads, railways, forest and even loss of human and wild life. Some of the examples of major dam-break flood disasters are St. Francis dam, U.S.A. (1928), Eildon dam, Australia (1938), Malpasset dam, France (1959), Vajont dam (1963) and Stava dam (1985), Italy, Baldwin Hill dam, U.S.A (1963), Banqiao dam, China (1975), Lake Ha! Ha! Quebec (1996). In India some examples of devastation due to dam failure are Tigra (1917), Poona (1961), Pachet Hill dam (1961), Khadak Wasla dam (1964), Nanak Sagar (1967), Machhu II (1979), Hirakud (1980). After these dam-break disasters, in most of the countries in the world, regulations were laid down for the dam owners to undertake dam break flood analyses. Several dams are proposed to be built in North East region of India, to harness the existing hydropower to facilitate electricity, irrigation, water supply, navigation, recreation and to achieve flood and erosion control. While multi-objective dam can provide immense benefit, its failure or improper operation may lead to devastating flood downstream. One of the major challenges to overcome for economic development of North East India is the management and planning against such devastating flood. This damage will be more destructive when a dam fails suddenly. The outcome of such occurrence will be a great danger to human and wild life causing significant economical and environmental damages. Readiness to face such dam break flood disaster can be attained when prior knowledge of the flood movement is acquired through simulation model.

1.2 Difference of dam break flood from natural flood

The flood caused by a dam failure is different from natural floods due to several reasons (CADAM final report 1999), namely:

(i) Water levels rises very quickly unlike gradual rising of water levels in a natural flood

(ii) The magnitude and extent of flooding will be much larger

(iii) Areas not liable to flooding will be also affected (e.g. forests, urban areas)

(iv) The route of flood flow will not be dictated by the river channel but more by the overall valley topography

(v) Flood warning times may be minimal or non existent

(vi) Chances of occurrence of structural damage is quite high (i.e. bridges, embankments, housing etc. are likely to be destroyed)

(vii) Large quantities of sediment may be transported leading to deposition as well as erosion.

(viii) Great difficulty arises in calibrating the numerical models (the variables are outside the usual range)

The dam break modelers must therefore, make a number of modelling decisions which may significantly affect the output results produced and in particular the prediction of the flood wave arrival time (Graham, 1998).

1.3 Aim and objective of the study

The aim of the study is to predict and analyze dam break flood in real river topography with an objective to suggest mitigation measures to minimize the expected damage. To formulate suitable flood hazard mitigation measures, prior knowledge of the flood movement i.e., the inundated area, the peak flood depths, and velocities and travel time of the flood waves is essential. All these can be obtained through simulation model developed to understand the dam break hydraulics in real complex flood plain topography. Hence the main aim of the study is to develop mathematical models for simulating dam break hydraulics using different forms of governing equations and adopting different numerical schemes so as to select the best one to simulate the dam break flood in complex real river floodplain. Faced with this real problem, series of reservoirs /dams are on the way of construction in north-eastern region of India. The hypothetical flood due to the failure of the proposed dam in a Himalayan river Dibang has been considered. Therefore, prime objective of the study is to develop a simulation model to predict and analyze the flood due to the instantaneous failure of the proposed large dam in this Himalayan River. To test applicability of the developed model in any natural river, another river Kynsy of Northeast India, having completely different topographic characteristics than that of the Dibang river, has also been considered. Also attempt has been made to analyze the dam-break flood in a river with mobile channel bed, in which significant work has not yet been done.

Therefore, objectives of the present study can be broadly categorized as:

1) To develop models to simulate dam-break flood in natural flood plain topography.
2) To assess the applicability of the developed models in any kind of real river valley.
3) To use the developed models to predict, analyze the depth, velocity, time of arrival of flood wave, inundation due to the failure of the proposed dam in the river Dibang.
4) To develop a dam-break model considering mobile channel bed.

5) To use the mobile bed dam-beak model in Dibang dam failure case to analyze the change in the original river bed and channel cross-sections of the same river after its failure.

6) To test the validity of the models by comparing the computed results with:

   (i) Laboratory data under sub, super and mixed flow conditions.

   (ii) Assessing the proposed model by real dam-break benchmark problem i.e. data of Malpasset (1959) dam failure.

7) To develop a model for damage estimation using available limited data of flood damage of Assam, India. The developed model has been used to compute the damage due to the failure of Dibang dam.

1.4 Organization of the Work

With an objective to make a systematic approach towards the problem, the proposed work has been divided into different phases and dealt separately in different chapters as outlined below.

At the very outset in chapter-2, a detailed discussion on the previous works conducted on various topics, relevant to this study has been made in a systematic way.

In chapter-3, the field data acquired with the assistance of National Hydro Power Corporation (NHPC) and National Productivity Council (NPC), India, for the river valley features and topographic characteristics for Dibang Dam Project have been presented.

Chapter-4 deals with the development of simulation models. Governing equations used for the purpose have been presented along with critical analysis regarding suitability of these equations in computing dam break flow. Then the numerical formulations of the governing equations are presented. A brief description of the user friendly computer program developed for the purpose has also been presented in this chapter. Finally performance of the different models developed in this chapter has been tested by applying the model in Dibang River valley and a critical analysis of the works carried out is presented.

In chapter 5, Validation of the developed numerical models is ascertained by comparing their outputs with laboratory data under all flow conditions i.e. sub critical, supercritical and sub-super mixed flow conditions. Robustness of the model has been assessed by applying the simulation model in another Indian river Kynsy having completely different topographic characteristics than that of the Dibang River.

In case of complex river topography, computed flow may vary significantly depending on how the downstream flow channel is represented in the model. In chapter 6, three 1-D models and one 2-D model, developed by considering downstream computational channel of Dibang river in different ways, have been presented. Simulated outputs obtained by implementing these models are critically analyzed to suggest the best one from practical point of view.

In chapter-7, dam break flow simulation model has been developed considering the river as mobile bed channel. The developed model has been applied in the Dibang River and variation of flow profiles and change in bed profile due to consideration of mobile bed has been analyzed.

The dam break simulation model was found as most suitable one among the models developed in this study, i.e., the 1D Simplified Flood Plain Model, has been assessed by comparing the model result with the real dam break bench mark problem data of Malpasset Dam and are presented in chapter-8.

The prime objective of dam break study is to take necessary disaster mitigation measures to minimize devastating effect of dam break flood. To estimate possible damage due to dam failure, a dam break flood predictions model and a damage estimation model is essential. In chapter 9, the proposed 1D model has been used for deriving inundation map and for estimating maximum depths, maximum discharges, maximum velocities and time of attaining maximum depth at any sections downstream of the dam. A flood damage estimation model developed using available limited data of flood damage of Assam, India has also been presented in this chapter. Finally estimated damage due to the failure of Dibang Dam has been presented in this chapter along with necessary mitigation measures.

In the concluding chapter i.e. in chapter-10, a comprehensive conclusion and general discussion on various works performed in this study have been presented. Some suggestions have also been put forward for further extension of author’s works.

Chapter-2

Review of Literature

2.1Introduction

Over the recent decades, there have been continuing efforts to enhance the understanding of dam break hydraulics. A brief review of the previous works found in literature on different situations of dam-break problem is presented here with the aim to focus the following main points:

(i) **Governing equations** i.e., mathematical formulation of the unsteady free surface flow

(ii) **Analytical solution** in the dam break situation

(iii) **Numerical simulation** of unsteady flow with special emphasis on dam break flood

(iv) **Simulations of dam break** flow considering **real field problems**

(v) Dam breaks hydraulics considering **mobile channel bed**

(vi) **Dam breach** modelling technique

2.2 Basic mathematical formulation of unsteady flow situation

The three basic fundamental laws of conservation of mass, momentum and energy can represent the free surface unsteady flow. As two equations are sufficient to represent the unsteady flow, either mass momentum couple or mass energy couple of conservation law is used depending on the physical situation .For discontinuous flow like the dam-break problem, mass-momentum couple is generally used. The continuity and momentum equations derived on the basis of Saint-Venant hypothesis (1871) can be regarded as the first valid step towards the mathematical representation of unsteady gradually varied flow.

2.3 Previous works on analytical and graphical solution of dam break problem

Ritter (1892) made the first attempt on this problem analytically. Starting from the characteristics equations, he solved the one dimensional St.Vanant equation for a frictionless horizontal channel.

Depth of flow "h" and velocity of flow "v" at time "*t*" at a distance x from dam site are represented by Ritter equation as:

$$h=\frac{1}{9g}(2\sqrt{gh_o}-\frac{x}{t})^2 \qquad \ldots(2.1)$$

$$v=\frac{2}{3}(\frac{x}{t}+\sqrt{gh_o}) \qquad (2.2)$$

Where h_o is depth at dam site.

But this classical solution of Ritter fails to describe accurately the physical flow as the friction at the wave tip is not negligible.

Craya (1946) developed a graphical approach based on characteristics to solve the Saint-Venant equations. His method was later modified by Re (1946) and Levin (1952) in their study of dam break analysis.

An attempt had been made by Dressler (1952, 1954) to solve the non-dimensional equation for rectangular channel the considering resistance term.

The non-dimensional form of the equation

$$\frac{\partial UN}{\partial TN}+U\frac{\partial UN}{\partial XN}+2C\frac{\partial CN}{\partial XN}+R(\frac{UN}{CN})^2=0 \qquad (2.3)$$

$$CN\frac{\partial UN}{\partial XN}+2\frac{\partial CN}{\partial TN}+2U\frac{\partial CN}{\partial XN}=0 \qquad (2.4)$$

$$\text{where } XN=\frac{x}{h_o}, TN=t(\sqrt{\frac{g}{h_o}}), UN=\frac{v}{\sqrt{gh_o}}, \quad CN=\frac{c}{\sqrt{gh_o}} \qquad (2.5)$$

Whitham (1955); in his paper suggested that in solution of dam break problem the resistance effect must be considered at the wave front where frictional resistance and resulting turbulence dominate the flow. So the tip region was treated as a definite boundary layer, in which the St.Venant equation ceases to be applied because the resistance effect is no longer negligible there. He has obtained from experiments that the velocity at wave front varies very little with distance hence at the tip region. Thus, he proposed it as the function of time only, approximate velocity is given as:

$$\frac{da}{dt} = \sqrt{gh_o}(2 - 3.452(kt\sqrt{g/h_o})^{1/2}) \qquad (2.6)$$

Where a =position of wave front and k=constant of proportionality

SU and Barnes (1970) modified Dressler's method and more general form of solutions is obtained for different shapes of channel i.e. for rectangular ,parabolic ,triangular cross sections with and without friction which is presented in form of some graphs.

Hunt (1982, 1984) obtained relatively simple closed form of solutions of the dam break problem. The reservoir outflow was calculated by using a quasi steady flow approximation and the downstream flood depths are calculated by using the kinematics wave imitations to obtain closed form solution for failure of dam upon dry, sloping channel and results were compared with previously calculated numerical solutions and experimental results. However investigation has shown that constraints much be placed for the application of the models, such as for the outflow of the reservoir model the ratio of the breach width to reservoir width should be less than 0.37 and for downstream flood the ratio of downstream flow depth to the channel width should be small enough to allow the approximation of hydraulic radius to the flow depth.

Sarma and Das (2003) have developed a new characteristic-based analytical solution of flood for the situation where flood water, being released through an opening in the river dike, moves over a valley.

The flow depth "h" and flow velocity "v" at time "t" at a distance "x" form the dike, are represented as:

$$h = \frac{1}{9g}\left[3\sqrt{gh_o} - \frac{x}{t}\right]^2 \qquad (2.7)$$

$$v = \frac{2}{3}\left[\frac{x}{t} + \frac{3}{4}\sqrt{gh_o}\right] \tag{2.8}$$

2.4Previous Works on Numerical Simulation of Unsteady Flow

Numerical simulations of dam break flood started with simple cases such as rectangular frictionless channels and remain a topic of interest till date for mathematical simulation of dam break flood with complex channels and floodplains. Some examples of 1D models are: Sakkas- Streloff (1973,1976), Das (1978), Barr and Das (1980, 1981), Katopodes (1984a,b), Garcia, Kahawita (1986), Bellos and Stakkas (1987), Fennema and Choudhury (1989) , Fennema and Choudhury (1990), Bellos, Soulis and Sakkas (1992) , Garcia et al(1992), Hicks et al(1992) , Yang et al (1993), Jha et al (1994), Zhao (1994), Nujic (1995), Rahman and Choudhury (1998), Tseng and Chu (2000), Zoppou and Roberts (2000), Aureli, (2001), Sanders (2001), Macchione and Morelli. (2003), Zoppou, Roberts (2003), Zhou et al (2004), Macchione, Viggiani (2004), Saikia and Sarma (2006). Simulation with 2D numerical model are: Katopodes (1984), Hromadka (1985) , Akanbi, et al (1988), Fennema and Choudhury (1989, 1990) , Zhao and et al (1996), Sarma (1999) , Zoppou and Roberts (2000), Lin et al (2002), Shige-edam and Akiyama (2003), Aureli and Mignosa. (2004)

“Gasper Monge” developed a graphical procedure known as the method of characteristics (MOC) in 1789 at Paris for the integration of partial differential equations. The first application of M.O.C, to the field of hydraulics engineering is credited to “Massau” **(**1889). M.O.C technique is for converting the partial differential equation into solution of ordinary differential equations. Issacson, Stoker and Froesch (1954) utilized this method to investigate the propagation of flood wave in open channel.

Lax (1954) presented a simple explicit finite difference scheme known as Lax diffusive scheme which gives satisfactory results in typical hydraulics engineering problems.

Faure and Nahas (1969) developed a numerical solution on the basis of the Whitham approximations using characteristics method.

Shugrin and Sudobicher (1968) developed a numerical solution for breached wave on the dry bed on moving grid system of finite difference form.

Sakkas and Strelkoff (1973, 1976) used non dimensional form of the characteristic equations derived from equation (2.1) and the generalized Ritter solution as the initial condition for starting the computation. The characteristics equations were solved numerically on a characteristic grid using Euler formula and the trapezoidal rule in a predictor corrector scheme. This method fails at the tip region and therefore, the solution was extended to the wave tip by using simplified form of momentum equation, derived from consideration of the physical situation.

Xanthopoulos (1976) developed solution of flood propagation in a two dimensional domain. The study was characterized by small values of water depth and velocity, compared to the flood propagation in a narrow valley and the nonexistence of a wave front with the characteristic shock properties.

Katapodes and Strelkoff (1978) developed simulation of two dimensional propagation of dam break flood wave by using method of characteristics. The bore was isolated and tracked explicitly along the downstream channel.

Das (1978) studied the effect of resistance on dam break flow, through numerical solution. Barr and Das (1980, 1981) have presented the theoretical and experimental studies of the effect of resistance on the unsteady free surface following the dam break situations in 1-D flow. Solution was based on finite difference moving grid system in x-t plane where provisions were made to calculate friction factor, at any position and at any time, by an approximate formula .The experiment have been undertaken in the laboratory flumes and acceptable agreements with theoretical data have been obtained. The effect of resistance in the free surface profile has been found pronounced as shown in the figures.

Initial condition to start the numerical solution, Dressler's solution modified by Su and Barness has been taken. Down stream boundary condition at tip is from Sakkas and Strelkoff, which is based on Whitham approximation at tip region. Upstream boundary condition is simple with infinite reservoir with horizontal bed.

Matsutomi (1983) used the leapfog scheme to compute dam break flow profiles over dry bed. He used shock fitting to track the bore as used by Sakkas and Strelkoff and Ritter's solution was used for the first few time steps.

Katopodes (1984) proposed a variance in the Galarkin method for both one dimensional and two-dimensional flow that exhibits highly selective damping characteristics as classical Galerkin formulation produces very poor result when applied to discontinuous flow. The dissipation affects only the numerically generated high frequency parasitic waves while maintaining remarkable accuracy in the approximation to the true solution. It has been shown as the generalization of the Lax Wendroff F.D. method, it poses the best features of the finite difference scheme which is mostly used in open channel flow field and in addition it has the advantage of handling the complex boundary and domain conditions.

Hromadka (1985) developed a simple two-dimensional dam break model where the governing flow equations are approximately solved as a diffusion model coupled with the equation of continuity, and applied it to a hypothetical dam failure situation of the river "Owens" of California. Hromadka considered also diffusion model for one dimensional channel flow and concluded that for low to moderate velocity flow regimes (i.e., less than 25 feet/sec), the diffusion model is a reasonable approximations of the full dynamic wave equation. The one dimensional dynamic wave implicit dam break model K-634 developed by Land, was used to route the flood hydrograph initially through a reservoir till the dam fails. After breach the dam break flood was routed downstream though a canyon by solving the one dimensional St. Venant equation by nonlinear implicit F.D algorithm. As negligible attenuation of the flood wave peak was observed, in the subsequent study the two-dimensional plains were assumed to have the dam break located at the downstream point of the canyon. One-dimensional model of K-634 was also used for prediction of flow depth in the flood plain. Comparison of the one dimensional and two dimensional model reveals that the two dimensional model produces more accurate prediction in case of a broad and flat plain, especially when the flow can break out the main channel and then returns at other downstream reaches.

Garcia and Kahawita (1986) used the MacCormack scheme in development of a two-dimensional hydraulic simulation model that solves the St.Venant Equation. They have shown that the proposed model is able to treat the rapidly varying flow as well as gradually varied flow problems in hydraulics. It has been used for one dimensional elementary problem; to which analytical solution exists, to verify some fundamental characteristics of the numerical scheme such as phase and amplitude errors, stability and shock capturing ability. More complex two dimensional problems were then attempted to assess the performance of the model and its response to the variation of different parameters involved in the solutions.

Bellos and Stakkas (1987) simulate the dam break flow over a dry bed in one dimension using McCormack scheme.

A two dimensional simulation model for the shallow water flow specially, flood waves propagating on dry bed were presented by Akanbi et al (1988). The governing equations are transformed to an equivalent system valid on a deforming coordinate system and are solved by a dissipative finite element technique. Whitham equation was used for the initial condition. The accuracy and the stability of the model were examined by comparing the results of the model with observed data from an experimental field test.

Fennema and Choudhury (1989, 1990) proposed explicit Mac Cormack scheme and implicit Beam and Warming scheme to simulate two-dimensional unsteady free surface flow

Francisco et al (1992) applied the flux difference splitting technique for solution of St.Venant equation in case of channel having arbitrary cross section. This technique provides a mean for the use of the high-resolution schemes based on flux limiter. Thus non-oscillatory solutions were obtained while retaining the second order accuracy in smooth flow region.

Hou Zang et al (1992) developed an implicit numerical method for the solution of the complete one dimensional shallow water wave equation.

Garcia-Navarro et al (1992) combined MacCormack explicit scheme and the TVD technique to simulate unsteady open channel flows.

Yang et al (1993) presented various characteristics based on high-resolution non-oscillatory shock capturing finite difference schemes and Petrov-Gelerkin finite element method for computation of one-dimensional free surface flow resulting from a dam failure. One-dimensional FD schemes like upwind scheme, second order TVD scheme, second order non-oscillatory scheme and third order non-oscillatory scheme were presented in their works. Analytical solution for sudden formation of bore wave was used for verification of the numerical results.

Savic and Holly (1993) proposed a new algorithm based on Godunov method which is capable to overcome the existing discontinuity in the flow field, as simple methods cannot treat it appropriately and on the other hand the shock fitting techniques can handle it properly but difficulties arise in tracking the shock in natural channels. The proposed algorithm copes efficiently with mixed flow regimes and strong shocks in non-prismatic channels.

Jha et al (1994) proposed a modification to the well known Beam and Warming implicit scheme for computing one-dimensional unsteady free surface flow to incorporate an important point that while using the numerical methods for computing free surface flow in super critical region, one should use information from upstream only and in case of sub critical flow in formations from both upstream and downstream should be used. The modification was based on the concept of conservative splitting flux technique which allows automatic switching of space difference operators when the flow changes from sub critical to super critical and vice versa. This enables correct modeling of flow situations where both super critical and sub critical flow exist simultaneously. Computational results are compared with the solutions of the original beam warming scheme as well as the analytical solutions. The modified method significantly improves the accuracy of the results but requires a little more computational time as compared to the original scheme.

Zhao et al (1994) developed a two –dimensional unsteady flow model based on the finite volume model with a combination of unstructured triangular and quadrilateral grids in a river basin. The attractive feature of this model is that it calculates the mass and momentum flux across each side of the elements as a Riemann problem, which is solved using Osher scheme. This feature enables this model to deal with the wetting and drying processes of the floodplain; dam break flood involving discontinues flow, super and sub critical flow and other such complex cases. The Oshar scheme used is a method based on characteristic theory. It is a shock capturing, monotone upwind, and high-resolution numerical scheme. Kissimmee river basin was used as their experimental area and comparative study of Riemann solvers FVS (Flux vector splitting), FDS (Flux Difference Splitting) and Osher scheme were conducted by simulating dam break flow in that area. Authors claimed that it could be suitably used for simulating flow through hydraulic structures like weirs, gates bridges, etc. However, Nujic (1996) pointed out that the numerical treatment taken for the bottom of the slope as improper as this term may lead to great inaccuracies, if strong variations in bottom topography are present.

Alam and Bhuiyan in 1995 used collocation method in conjunction with Quintic Hermite Element to solve the system of flow equations. They have used Quintic Hermite Element on the contrary to the discontinuous weighting functions as used in the other finite element methods, which omit the requirement of the provision for using the arbitrary dissipative mechanism to control the spurious oscillations that lead to instability.

Jha et al (1995) made extensive investigation regarding application of first and second order flux difference splitting scheme to dam break problem. It had been concluded that although higher order schemes provide better shock resolution, Roe's (1981) first order schemes may be preferred for practical applications when computational time, overall accuracy and applicability are considered .In case of shock capturing scheme it is desired to keep the depth ratio as small as possible, the possible reduction of depth ratio in case of dam break simulation that has been studied using different existing shock capturing scheme.

Nujic (1995) presented two high resolution numerical schemes based on Lax-Friedrich and ENO (Essentially non-oscillatory) type of extrapolation. It has been successfully applied and shown that they are much easier to implement than the methods which use field to field decomposition and require less computational time. It has been stated that the scheme can be successfully applied to shallow water flows with variable bottom topography with an accuracy similar that to the other high-resolution schemes.

Zhao et al (1996) applied three approximate Riemann solvers, namely FVS (Flux vector splitting), FDS (Flux difference splitting) and Osher scheme and compared them according to theoretical developments, different schemes and applicability to hydraulic shock wave problems .These Riemann solvers based on the characteristic theory were used in the finite volume method for solving the two dimensional shallow water equations. They suggested that algorithms to estimate flux should appropriately handle the inherent directional property of signal propagation. Comparison of the numerical and analytical solutions indicates that all three approximate Riemann solvers observe very good agreement. For practical purpose all of them can satisfactorily simulate the hydraulic phenomenon in sub critical and supercritical flows as well and in smooth dissentious flows.

Rahman and Choudhury (1998) remarked that proper grid distribution in numerical solution can play an important part in the prediction of the solutions. They used an adaptive grid, which adjusts itself as the solution proceeds. They claimed that that their solution takes care of both sub critical and supercritical flows. Their solution is limited to only wide rectangular, horizontal and frictionless channel. It is perhaps the simplest case of 1-D flow, which has not much relevance to the practical field problems. Most of the practical problems are with friction.

Liska and Wendroff (1999) presented a composite finite difference scheme to solve the two-dimensional shallow water wave equations, where the Lax Wendroff (LW) and Lax-Friedrich (LF) are combined. As both of these methods have some drawbacks i.e. the LW scheme is second order accurate but the scheme is oscillatory closed to the shocks and LF scheme is non oscillatory but excessively diffusive so they are combined to remove their deficiencies where several LW steps follow by one diffusive LF step that serves as a consistent filter removing the unwanted oscillations. The composite scheme was applied to the circular dam break problem and they stated that although the speed comparison was not done with other methods these simple schemes result in a rather fast numerical algorithm.

Sarma (1999) simulated two dimensional flows propagating from the opening in a river dike by modified predictor corrector scheme. The computed results were in a good agreement with the experimental values which have been taken in experimental set up for three different discharges with different slopes.

Tseng and Chu (2000) presented a Finite difference predictor-corrector TVD (total variation diminishing) scheme for the computation of unsteady one-dimensional dam-break flows. The proposed algorithm modified the widely used MacCormack scheme by implementing a conservative dissipation step to avoid any unphysical oscillation in the vicinity of strong gradients in the numerical solution. The accuracy and robustness of the numerical scheme are verified with an analytic solution and experimental data. The good agreement between the computed and measured water depth ascertains that the proposed TVD scheme is capable of dam-break flow simulations. Furthermore, a sensitivity study is carried out to investigate the accuracy of four different versions of the predictor-corrector schemes. It is found that the numerical scheme will have less computational error and higher efficiency when the direction of the predictor-corrector step is the same as the direction of the shock

Wang et al (2000) investigated a second-order hybrid type of total variation diminishing (TVD) finite-difference scheme for solving dam-break problems. The scheme is based upon the first-order upwind scheme and the second-order Lax-Wendroff scheme, together with the one-parameter limiter or two-parameter limiter. A comparative study of the scheme with different limiters applied to the Saint Venant equations for 1D dam-break waves in wet bed and dry bed cases shows some differences in numerical performance. An optimum-selected limiter is obtained. The scheme is extended to the 2D shallow water equations by using an operator splitting technique, which is validated by comparing the present results with the published results, and good agreement is achieved in the case of a partial dam-break simulation. Predictions of complex dam-break bores, including the reflection and interactions for 1D problems and the diffraction with a rectangular cylinder barrier for a 2D problem, are further implemented. The effects of bed slope, bottom friction, and depth ratio of tail water/ reservoir are discussed simultaneously.

Zoppou and Roberts (2000) described a numerical model for the solution of the two-dimensional dam break problem which is based on a second-order approximate Riemann solver with a van Leer type limiter used to solve the shallow water wave equation on a Cartesian grid. The shallow water equations include source terms which account for resistance to the flow and the influence of the bed slope. The wetting and drying process is also considered. The model is verified by comparing the model output with documented results. A simple example of the collapse of a water supply reservoir in a narrow valley is used to demonstrate the capability of the model. It is capable of resolving shocks, handling complex geometry, including the influence of steep bed slopes and simulating the wetting and drying process.

Aureli (2001) presented numerical simulation using based on the well-known McCormack shock-capturing scheme. In implementing the numerical scheme the source terms of the equations so treated that they are compatible with the resolution of shocks introducing artificial dissipation terms or Total Variation Diminishing (TVD) corrections. To verify the numerical model under severe test conditions, laboratory experiments have been carried out in which shock formation, reverse flow and wetting and drying conditions in the flow field were induced.

Mingham et al (2001) presented a simple update to a well-known classical MacCormack scheme with a total variation diminishing term appended to the corrector step which eliminates spurious numerical oscillations which often arise when the convective terms are discretized using classical central difference schemes, for solving the non-linear shallow water equations. The scheme is explicit, second-order in space and time and able to resolve accurately both sub critical and supercritical flows. They found that their scheme can be used to solve problems involving high spatial gradients such as bore waves and hydraulic jumps which are areas where classical schemes have been found to perform badly or fail to perform at all.

Sanders (2001) presented a scheme to model open channel flow over wet and dry beds in non-rectangular and non-prismatic channels. The scheme solves the St.Venant equations using a Godunov-type finite volume method. Mass and momentum fluxes are computed using a Roe-type Riemann solver, the MUSCL (Monotone Upwind Scheme for Conservation Laws) approach is applied for second-order spatial accuracy, and a treatment is introduced to model the hydrostatic pressure force exerted by the channel walls in the stream wise direction. The treatment permits momentum fluxes and the channel wall force to be balanced to numerical precision, preventing the artificial acceleration of the flow. Comparisons between model results, exact solutions, and experimental data have shown good agreements.

Venutelli (2001**)** proposed Padé–Galerkin solution for dam-break simulations .The fractional time-step integration of Saint-Venant equations is obtained by a three-stage Padé implicit approximation, whereas the spatial discretization is obtained by the standard Galerkin finite-element method. The model is unconditionally stable, becomes dissipative with the formulation of the unknown quantity mass matrix in lumped form. Results, in terms of water profiles and depth hydrographs, following dam-break phenomena, are investigated in one- and two-dimensional problems.

Caleffi et al (2002) simulated flood in natural channels by a finite volume method based on the Godunov approach. The HLL Riemann solver is used. MUSCL (Monotone Upwind Scheme for Conservation Laws) predictor–corrector techniques achieve a second order accuracy in space and time respectively. The high-resolution requirement is ensured by satisfaction of TVD property. Particular attention is posed to the numerical treatment of source terms. Accuracy, stability and the reliability of the code are tested on a selected set of study cases. A grid refinement, analysis is performed. Numerical results are compared with experimental data, obtained by the physical Modeling of a submersion wave on a portion of the Toce river valley, Italy.

Frazão and Zech (2002) presented an experimental study of a dam-break flow in an initially dry channel with a 90° bend, with refined measurements of water level and velocity field. The measured data are compared to some numerical results computed with finite-volume schemes associated with Roe-type flux calculation.

Lin et al (2002) proposed Four finite-volume component-wise total variation diminishing (TVD) schemes for solving the two-dimensional shallow water equations. In the framework of the finite volume method, a proposed algorithm using the flux-splitting technique is established by modifying the McCormack scheme to preserve second-order accuracy in both space and time. Based on this algorithm, four component-wise TVD schemes, including the Liou–Stefen splitting, van Leer splitting, Steger–Warming splitting and local Lax–Friedrichs splitting schemes are developed. These schemes are verified through the simulations of the 1-D dam-break, the oblique hydraulic jump, the partial dam-break and circular dam-break problems. It is demonstrated that the proposed schemes are accurate, efficient and robust to capture the discontinuous shock waves without any spurious oscillations in the complex flow domains with dry-bed situation and bottom slope or friction.

Bradford and. Sanders (2002) developed a model based on the finite-volume method for unsteady, two-dimensional, shallow-water flow over arbitrary topography with moving lateral boundaries caused by flooding or recession. The model uses Roe's approximate Riemann solver to compute fluxes, while the monotone upstream scheme for conservation laws and predictor-corrector time stepping are used to provide a second-order accurate solution that is free from spurious oscillations. It has been presented to simulate the movement of a wet/dry boundary without diffusing it.

Zoppou and Roberts (2003) examined the performance of 20 explicit numerical schemes used to solve the shallow water wave equations for simulating the dam-break problem. Results from these schemes have been compared with analytical solutions to the dam-break problem with finite water depth and dry bed downstream of the dam. Most of the numerical schemes produce reasonable results for sub critical flows. Their performance for problems where there is a transition between sub critical and supercritical flows is mixed. The majority of first-order schemes examined produce nonphysical solutions. Second-order schemes avoid the generation of expansive shocks; however, some form of flux or slope limiter must be used to eliminate oscillations that are associated with these schemes. These limiters increase the complexity and the computational effort required, but they are generally more accurate than their first-order counterparts. The limiters employed by these second-order schemes will produce monotone or total variation diminishing solutions for scalar equations. Some limiters do not exhibit these properties when they are applied to the nonlinear shallow water wave equations. This comparative study shows that there are a variety of shock-capturing numerical schemes that are efficient, accurate, robust, and are suitable for solving the shallow water wave equations when discontinuities are encountered in the problem.

Wang et al (2003) presented finite-volume total variation diminishing (TVD) scheme for modeling unsteady free surface flows caused by dam-breaks in branch channels. To extend the finite-difference TVD scheme to finite-volume form, a mesh topology is defined relating a node and an element. The solver is implemented for the 2D shallow water equations on arbitrary quadrilateral meshes, and based upon a second-order hybrid TVD scheme with an optimum-selected limiter in the space discretization and a two-step Runge–Kutta approach in the time discretization. It has been used to predict the characteristics of free surface flows due to dam breaking in branch channels, for example, in a symmetrical trifurcated channel and a natural bifurcated channel, on coarse meshes and fine meshes, respectively.

Rao (2003) in his work presented a multiple grid algorithm coupled to an explicit finite difference scheme for solving the two-dimensional open channel flow equations. In a multiple grid approach, for a fixed time level the equations are solved for at nodes across different spatial domains. This ensures maximum propagation of the disturbance with minimum computational effort. Focus in this work is laid equally both on its formulation and its reliability for simulating two-dimensional transient open channel flows.

Shige-eda .and Akiyama (2003) investigated the behavior of two-dimensional (2D) flood flows and the hydrodynamic force acting on structures numerically and experimentally. Numerical simulations are performed using a model based on the finite-volume method with an unstructured grid system and the flux-difference splitting technique. Experiments on flood propagation in a flood plain, with and without structures were conducted so as to obtain a comprehensive verification of the model. Front positions, depths, and surface velocities of flood flows as well as hydrodynamic forces on structures were observed.

Guinot (2003) presented a Riemann solver that converts two-dimensional Riemann problems into one-dimensional equivalent Riemann problem (ERP) which is derived by applying the theory of bicharacteristics at each end of the interface and by performing a linear averaging along the interface. The proposed approach is tested against the traditional one-dimensional approach on the classical circular dam problem. Use of the two-dimensional solver with a first order scheme may give better results than the use of a second order scheme. The theory of the bicharacteristics is also used to discuss the boundary conditions and he found that, when the flow is sub critical, the number of boundary conditions affects the accuracy of the solution but not its existence and uniqueness. When only one boundary condition is to be prescribed, it should not be the velocity in the direction parallel to the boundary. When two boundary conditions are to be prescribed, at least one of them should involve the component of the velocity in the direction parallel to the boundary.

Calenda et al (2003) simulated Tiber River flood of Rome during the December 1870 by a depth-integrated two-dimensional varied flow model, using a constant inflow equal to the peak discharge of the flood. The model fits satisfactorily to the observed water elevations, both in the river and in the flooded areas.

Macchione and Morelli (2003) compared the performances of first-order and total variation diminishing second order upwind flux difference splitting schemes, first-order space-centered schemes, and second-order space-centered schemes with the TVD artificial viscosity term. The schemes were applied in a dry frictionless horizontal channel; in a dry, rough and sloping channel; and in a nonprismatic channel. Among first-order schemes, the diffusive scheme provides only slightly less accurate results than those obtained by the Roe scheme. For TVD second-order schemes, no significant differences between the upwind scheme and central schemes are reported. In the case of a dam break in a dry frictionless horizontal channel, the second-order schemes were two- to five-fold more accurate than the diffusive scheme and Roe’s scheme. These differences in scheme performances drastically reduce when the results obtained for the rough sloping channel test and for the nonprismatic channel test are analyzed. In particular, the accuracy of the diffusive and Roe’s schemes is similar to second-order schemes when such features of dam break wave, relevant from an engineering viewpoint, like wave peak arrival time and maximum water depths, are considered.

Gottardi and Venutelli (2004) have used a new second-order central scheme, proposed by Kurganov and Tadmor (KT) for the solution of the two-dimensional dam-break problem. The KT scheme in its semi-discrete second-order form has been extended for taking into account the presence of a source term. Using a third-order TVD Runge–Kutta scheme, time integration has been performed. The solution obtained by this model for a dam-break problem of one-dimensional flow has been compared with the corresponding analytical solution; moreover, the solutions obtained for two test cases of two-dimensional flow have been compared with the experimental results.

Zhou et al (2004) simulated Dam-break flows in general geometries with complex bed topography numerically using a high-resolution Godunov-type cut cell method. The model is based on the shallow water equations with appropriate source terms. A vertical step in the bed is treated efficiently and accurately with the surface gradient method. For dam-break flows occurring in complicated geometries, the Cartesian cut cell method together with transmissive boundary conditions is incorporated. Verification of the method is carried out by predicting dam-break flows typical of practical situations, i.e., dam-break flows over a vertical step into bent channels and a dam-break flow over a bump in a bed with both transmissive and reflective boundary conditions at the channel end. The results are compared with experimental data showing good agreement.

Schwanenberg and Harms (2004) presented a total variation diminishing Runge Kutta discontinuous Galerkin finite-element method for two-dimensional depth-averaged shallow water equations. The scheme is well suited to handle complicated geometries and requires a simple treatment of boundary conditions and source terms to obtain high-order accuracy. The explicit time integration, together with the use of orthogonal shape functions, makes the method for the investigated flows computationally as efficient as comparable with finite-volume schemes. For smooth parts of the solution, the scheme is second order for linear elements and third order for quadratic shape functions both in time and space. Shocks are usually captured within only two elements. Several steady transcritical and transient flows are investigated to confirm the accuracy and convergence of the scheme. The results show excellent agreement with analytical solutions. For investigating a flume experiment of supercritical open-channel flow, the method allows very good decoupling of the numerical and mathematical model, resulting in a nearly grid-independent solution. The simulation of an actual dam break shows the applicability of the scheme to nontrivial bathymetry and wave propagation on a dry bed.

Fagherazzi et al (2004) presented a discontinuous Galerkin method for the solution of the dam-break problem. The scheme solves the shallow water equations with spectral elements, utilizing an efficient Roe approximate Riemann solver in order to capture bore waves. A projection limiter that eliminates spurious oscillations near discontinuities enhances the solution. The one-dimensional dam break results are verified by comparison with analytical solutions. The application to a two-dimensional dam-break problem shows the efficiency and stability of the method.

Yoon and Kang (2004) developed a numerical model based upon a second-order upwind finite volume method on unstructured triangular grids for solving shallow water equations. The HLL approximate Riemann solver is used for the computation of in viscid flux functions, which makes it possible to handle discontinuous solutions. A multidimensional slope-limiting technique is employed to achieve second-order spatial accuracy and to prevent spurious oscillations. To alleviate the problems associated with numerical instabilities due to small water depths near a wet/dry boundary, the friction source terms are treated in a fully implicit way. A third-order total variation diminishing Runge–Kutta method is used for the time integration of semi discrete equations. The developed numerical model has been applied to several test cases as well as to real flows.

Macchione and Viggiani (2004) considered the complex problem of dam failure and subsequent flood estimation. Numerical models are required and distinctive features of natural rivers such as friction and real topography have to be considered. They represent, a very easy to implement scheme, the diffusive scheme. Stability and accuracy of the numerical solution are analyzed and the performances in terms of water depths are tested. The Malpasset dam-break case (France, 1959) is referred to as a test case. Numerical results are compared with the depth measurements of a physical model and with the results of two other numerical models available in the literature

2.4.1 Observations form the Works on Numerical Simulation:

Bunch of numerical simulations of unsteady flow are available in literature with almost all types of numerical techniques ranging from the Finite difference, Finite element and Finite volume. But the performances of most of the schemes are tested in relatively simple channels. Some exceptional simulation cases are also reported e.g. 1D simulation models (Sanders (2001) ,Macchione and Viggiani (2004))are considered as may be in real world cases such as with changing cross sections ,complex bed topography, and with distinctive features of natural rivers such as friction and real topography, etc. Some 2 D model simulation models (Katopodes (1984), Hromadka (1985) Akanbi, et al (1988), Fennema and Choudhury (1989, 1990), Zhao and et al (1996), Sharma (1999) Zoppou and Roberts (2000), Lin et al (2002)) are also found representing the interaction of flow in the main channel and in simple flood plain ,due to breaking of a levee or due to release of flood water through a lateral gate or an opening

Some Simulations (Wang et al 2000, Bradford. and. Sanders (2002) Shige-eda etal2003, Zoppou (2003)) are also reported for simulation of flow over the floodplain having rectangular obstruction, floodplain with structures (concrete pillars are used in the experimental flow plain), shallow-water flow over arbitrary topography.

But Simulations of dam break flow in a river valley with the highly nonprismatic real river channel , with a high variations in bed slope, bed friction with a high dam on it as that may be in a real case , are rare to found in literature compared to the simulations in laboratory channels.

2.5 Dam Break Hydraulics considering Mobile Channel Bed

Among the literature found for dam break hydraulics, very few numerical models are proposed for mobile beds and applications were only in simple laboratory channels. Most of the models have been developed for fixed bed cases (with few exceptions as described later in this section) without considering the undoubtedly strong eroding capability of the highly transient dam break flow and the resultant morphological change of the channel bed.

Capart and Young. (1998) studied the highly unsteady flow due to the collapse dam over a loose granular bed. In contrast to the earlier works of dam breaks where dam break flow exhibit a continuous, monotonic depression in the free surface for up stream to the down stream, they have shown that sediment particles are swiftly eroded by the flow. In turn, the bed also induces changes in the flow. This interaction leads to the formation of a hydraulic jump in the center of the dam break wave. The above conclusion they have supported both numerically and experimentally.

Ferreira and Leal(1998) used 2nd order Mac-Cormack TVD and a 1st order modified Beam and Warming schemes and compared their solutions with the data of the experiments which was carried out at the Hydraulics Laboratory of Technical University of Lisbon.

Fraccarollo and Armanin (1998) proposed a theoretical approach to determine the solution to the initial-value problem represented by the dam break over a movable bed .This solution is controlled by the attitude of the bed to be eroded by the flow, expressed by a grain-mobility parameter. Turning the grain mobility parameter to zero, it is shown that the solution does coincide with the Ritter solution.

Paquire and Balayn(1998) , Paquire (2002)used a second-order Godunov-type explicit scheme in their proposed model.

Fraccarollo and Capart (2002) have examined the sudden erosion initiated by the release of a dam break over a loose sediment bed. Accounting for bed material inertia, a transported layer of finite thickness is introduced and a sharp interface view of the morphologic boundary is adopted. The wave structure features two smoothly varied waves and a special type of shock: erosion bore forming at the forefront waves. Profiles are constructed through a semi-analytical procedure, yielding geomorphologic generalization of the Stoker solution for dam break waves over rigid bed. For most of the flow properties, the predictions of the theoretical treatment compare reasonably with experimental tests visualized using particle imaging techniques.

In the recent analytical solution for sediment transport due to dam-break by Pritchard and Hogg (2002), the velocity field derived from the idealized clear-water flow over frictionless and rigid bed is presumed. The strong interaction between flow, sediment, and morphological evolution is ignored without any justification. Thus it may only find very limited use.

Cao et al. (2004) presented one of the detailed studies on mobile bed hydraulics of dam-break flow. A model was built using the total-variation-diminishing version of the second-order weighted average flux method in conjunction with the HLLC approximate Riemann solver and SUPERBEE limiter. The model was applied in a rectangular channel and several new basic observations were presented.

Wu and Wang (2006) proposed a 1-D finite-volume model with the first-order upwind scheme for Dam-break flow over movable beds .The model is then compared with two sets of experimental data observed in Taipei (University of Taiwan) and Louvain-la-Neuve (Université catholique de Louvain) reported by Capart and Young(1998) and Fraccarollo and Capart (2002) . Both sets concerned small-scale dam-break waves over movable beds in prismatic channels.

Saikia and Sarma (2006) numerically simulated the flow and bed profiles due to the hypothetical failure of the proposed dam in a Himalayan river Dibang.

In the recent literature, Wu and Wang (2007) have taken two experimental test cases as reported by Capart and Young (1998) and Fraccarollo and Capart (2002) respectively conducted in Taipei University of Taiwan and Louvain-la-Neuve Université Catholique de Louvain. They have also applied their model developed to the hypothetical large-scale dam-break flow case as assumed by Cao (2004).The model predicts that the flows in case of large dam break:

(i) Rapidly increases to their maximum values and then gradually decrease.

(ii) Bed erosion intensity and sediment concentration near the dam-break wave front are much larger than in other locations.

(iii) most of the erosion occurs as the wave front passes unlike in the small-scale case

2.6Previous works on Dam breach modeling technique

Cristofano's (1965) work is perhaps the first attempt to simulate the growth of a breach in an earthen dam. Using his geotechnical principles Cristofano equated force of water flowing through breach to resistive shear strength acting on the bottom surface of the over flow channel. Thus the rate of change of erosion was related to the rate of change of water flowing through the overflow channel. He assumed that the breach top width would remain constant over time and that the breach would maintain a trapezoidal shape throughout the failure process, and the side slopes of the breach equal to angle of repose of the bank material and the bottom slope of the over flow channel equal to the angle of friction of the bed material.

Fread and Harbaugh (1973) assumed a triangular gap of constant central angle whose bottom point travels either at constant or variable downward velocity. This allows simulation in case of failure of homogeneous dam or one with retarding layer.

Brown and Rogers (1977) showed that the assumption that instantaneous failure causes a positive wave in the downstream direction and the negative wave in the upstream direction is likely to be far from reality in case of gradual failure of earth dam. They documented a control section that formed just upstream of the breach and described how the reservoir level dropped uniformly upstream of this section, preventing the negative wave from propagating upstream.

Ponce and Tsivoglou (1981) formulated, developed a simulation model of gradual failure of an earth embankment and tested with real life data. The principles of unsteady open channel flow, sediment transport mechanism and channel morphology were successfully combined in the proposed mathematical model of breach enlargement and the ensuing flood wave. Unsteady flow elements of the simulation are an implicit numerical solution of the complete Saint Venant equations coupled with a sequential sediment routing technique. It was tested using data from the Huaccoto Dam failure, which occurred in central Peru in June 1974.

Singh and Snorrasan (1982) observed that the durations of the earth breaches can vary from 15 mints to more than 5 hrs. Ponce (1982) pointed out that this duration could last from 3 to 12 hrs. In case of non-engineered embankment with mild slopes, the failure may be even last 24 –48 hrs.

MacDonald and Monopolis (1984) collected data of number of historical dam failures, analyzed and developed graphical relationship for predicting breach characteristics. Erosion type breaches were considered for analysis as the predominated mechanism of breaching for earth fill dams is by erosion of embankment materials by the flow of water either over or through the dam.

Singh and Scalatos (1988) collected all necessary and possible data of dam failure from 52 historical dam failure cases. Analyzing these data they concluded that a simple lumped model for the breach formation, as suggested by Singh et al 1996) can be developed, including many of the relevant parameters and processes.

Froehlich (1995, 1995b) presented empirical relations based on regression analysis of the data collected from numerous published and unpublished sources of 22 dam failures for predicting the time of failure ,the average breach width and the peak outflow through the breach as:

Breach width

$$B_{avg} = 0.1803 \; K_0 V_w^{0.32} h_b^{0.19}$$

Failure time

$$f = 0.00254 \; V_w^{0.53} h_b^{-0.9}$$

Peak flow

$$Q_p = 0.607 \; V_w^{0.295} h_w^{1.24}$$

Where: K is the overtopping multiplier, 1:4 for overtopping, 1 for piping ;Vw is the volume of water stored at the time of failure, h_b is the height of breach and h_w=depth of water above breach at time of failure.

Singh (1996) summarized on the basis of his continuous study that some mathematical models of comprehensive nature are available which can include breach morphology such as sediment transport, breach shape and side slopes collapsing as modeling components while modeling dam break flood.

Walder and O'Connor (1997) revised the straightforward regression relations that prediction of peak outflow with the water volume released or drop in lake level and asserted that they are of limited utility as they fail to portray the effect of breach-formation rate as a function of various dam and/or reservoir parameters, with the relations developed from analyses of case study data from real dam failures. In contrast, to the other statistical methods, Walder and O'Connor's method is based upon an analysis of numerical simulations of idealized cases spanning a range of dam and reservoir configurations and erosion scenarios. An important parameter in their method is an assumed vertical erosion rate of the breach; for reconnaissance-level estimating purposes, they suggest that a range of reasonable values is 10 to 100 m/hr, based on an analysis of case study data. The method makes a distinction between so-called large-reservoir / fast-erosion and small-reservoir/slow-erosion cases. In large reservoir cases, the peak outflow occurs when the breach reaches its maximum depth, before there has been any significant drawdown of the reservoir. In this case, the peak outflow is insensitive to the erosion rate. In the small-reservoir case, there is a significant drawdown of the reservoir as the breach develops, and thus the peak outflow occurs before the breach erodes to its maximum depth. Peak outflows for small-reservoir cases are dependent on the vertical erosion rate and can be dramatically smaller than for large-reservoir cases. The determination of whether a specific situation is a large- or small-reservoir case is based on a dimensionless parameter incorporating the embankment erosion rate, reservoir size, and change in reservoir level during the failure. Thus, so-called large-reservoir/fast-erosion cases can occur even with what might be considered ‘‘small’’ reservoirs and vice versa. This refinement is not present in any of the other peak flow prediction methods.

Ghose and Sasikumar (1998) presented a new deterministic approach of analysis for simulating outflow hydrograph resulting from a gradual earth dam failure. They suggested that the initial and final dimension of the breach and the breach development time are function of embankment material and flow characteristics. During the process of breach development, initially erosion rate increases and attains a maximum value and finally at the end of breach development time there will be negligible erosion.

Coleman, Andrews and Webby (2002) constructed homogeneous small-amplitude embankments in flumes from a range of uniform non cohesive materials and they were breached by overtopping flows under constant reservoir level conditions. Embankment erosion evolves from primarily vertical to pre-dominantly lateral in nature. The breach channel initially erodes the downstream face of the embankment with an invert slope parallel to the face, the breach invert slope then progressively flattening to a terminal value by rotating about a fixed pivot point along the base of the embankment, the location of this pivot point being a function of the size of the embankment material. The breach channel is of a curved shape in plan.

Ponce et al. (2003) presented an analytical model of flood wave propagation to study the sensitivity of dam-breach flood waves to breach-outflow hydrograph volume, peak discharge, and downstream-channel bed slope.

Wahl (2004) presents an analysis of the uncertainty of many of the relations to make predictions of basic geometric and temporal parameters of a breach or the estimation of peak breach outflows which is required in studies of failure of embankment dam. Many of the relations most commonly used to make these predictions were developed from statistical analyses of data collected from historic dam failures. In his work an analysis of the uncertainty of many of these breach parameters and peak flow prediction methods, making use of a previously compiled database (Wahl 1998) of 108 dam failures were presented. The four methods for predicting breach width (or volume of material eroded, from which breach width can be estimated) and all had absolute mean prediction errors less than one-tenth of an order of magnitude, indicating that on average their predictions are on target. The uncertainty bands were similar ($\pm$0.3 to $\pm$0.4 log cycles) for all of the equations except the MacDonald and Langridge- Monopolis equation, which had an uncertainty of $\pm$0.82 log cycles. The five methods for predicting failure time all under predict the failure time on average, by amounts ranging from about one-fifth to two-thirds of an order of magnitude. Conservatively predict fast breaches, which will cause large peak outflows. The uncertainty bands on all of the failure time equations are very large, ranging from about $\pm$ 0.6 to $\pm$1orders of magnitude, with the Froehlich (1995a) equation having the smallest uncertainty. Most of the peak flow prediction equations tend to over predict observed peak flows, with most of the ''envelope'' equations over predicting by about two-thirds to three-quarters of an order of magnitude. The uncertainty bands on the peak flow prediction equations are about $\pm$0.5 to $\pm$1 order of magnitude, except the Froehlich (1995b) relation which has an uncertainty of $\pm$ 0.32order of magnitude. In fact, the Froehlich equation has both the lowest prediction error and smallest uncertainty of all the peak flow prediction equations. The case study presented here showed that significant engineering judgment must be exercised in the interpretation of predictions of breach parameters.

2.7Simulation of Dam Break Flood in Real Floodplain Topography

Simulations of dam break flow in real river valley are found to be very less in numbers compared to the simulation in laboratory channels. The projects and literatures related to simulation in real floodplain are as follows:

Test cases taken under CADAM (Concerted Action on DAm-break Modeling) Project (1996-1999) for Real dam-break case (Benchmark problem):

Malpasset dam failure (France): The dam failed in 1959, at 21:14 hours on 2nd December due to an exceptionally heavy rain. A total of 433 casualties were reported

Observations at the end of CADAM project:

(i) Modelers must try and compare with different assumptions.

(ii) 1D model may be used to simulate some 2D flow conditions; however, this requires considerable skill and experience if a reasonable level of accuracy is to be achieved.

(iii) 1D over estimated wave speed

(iv) 2D under estimated wave speed

(v) Accuracy of dam break models should not be compared to normal river models. Flow conditions are far more complex and data to validate the models are limited.

(vi) The accuracy of numerical models in predicting *general* hydrodynamic conditions is relatively good in comparison to other aspects of a dam break study.

(vii) Large scale movement of sediment is likely to occur during a dam break event leading to large variations in valley topography, particularly near to the dam.

IMPACT (Investigation of **E**xtre**M**e **Flood P**rocesses **A**nd **U**n**Ce**rtain**T**y**) Project (2000-2005):** The IMPACT project was carried out in order to improve scientific knowledge and understanding of extreme and aggressive flood flows following the catastrophic failure of water control structure. The objectives of the project are:

(i) Firstly, the movement of sediment

(ii) Secondly, the mechanisms for the breaching of embankment

(iii) Thirdly, the simulation of catastrophic inundation of valleys and urban areas

A key objective of the IMPACT project is to advance the understanding of uncertainty associated with the above processes.

Test case for real dam break: Tous Dam failure, Valencia region (Spain) is considered as the case of reference. The burst of Tous Dam occurred due to extraordinary heavy rain fall in 19-20 October 1982. The subsequent flood afflicted a large area; one hundred thousand people had to be evacuated, including the town of Sumacárcel (population 2000)

Malpasset Dam Break Simulation found in Literature

Some investigators (other than in CADAM) simulated Malpasset dam break flow are : Hervouet (2000), Valiani; (2000), Caleffi; and Zanni (2002), Macchione and Viggiani. (2004), D. Schwanenberg and Harms (2004), Ying and Wang (2005)

Numerical Simulation of Levee Breaking in the Middle of Po River

Aureli F. and Mignosa P. (2004) presented the results of mathematical modelling of three flooding scenarios due to levee breaking in the middle of Po River (northern Italy). In this area, some catastrophic floods that occurred in the nineteenth century caused the formation of several levee breaks and the inundation of wide plains (500 -600 km^2), with considerable water depths (6 -7 m). The inundation dynamics as a result of a levee break occurring at different locations along the right river bank of the Po River were modeled. The hypothetical breaches were located where the historical ones occurred more frequently or where the more severe consequences of the flooding were expected. Three scenarios were numerically simulated with a two-dimensional mathematical model based on the shallow water equations solved by means of the well-known MacCormack finite difference scheme. When possible, a calibration of the model parameters on the basis of the available information about recent and historical events was performed. The fairly good agreement between computed results and historical data suggests that the proposed approach is capable of reproducing the main characteristics of the phenomenon, giving support to the design of flooding maps for risk assessment

Numerical Simulation of Dibang and Kynsy

Saikia and Sarma (2006) numerically simulated hypothetical failure of two Indian Rivers namely Dibang and Kynsy. Advantages of using a simple Finite Difference Diffusive(FD DIFFUSIVE) scheme over a higher order numerical models in computing dam break flow in a real field situation with complex topography is investigated. Complexities arise when the flood wave due to dam failure moves over a natural flood plain topography having significant changes in bed elevations, bed slopes and bed frictions at each computational grid. For exploring merit of the proposed FD DIFFUSIVE scheme, it is compared with the TVD MacCormack scheme through their application in two hypothetical dam failure situations with complex topography; one located on the

Himalayan river Dibang, having wide flood plain on downstream, and the other on the river Kynsy, having steep narrow downstream channel. The simulation model with FD DIFFUSIVE scheme is found to be highly stable and well applicable for these complex hypothetical failure situations. The ease of implementation, less computational runtime and the ability of handling flows in sub critical, mixed and highly super critical flow conditions are the favorable features of the presented model. It provides accurate predictions for maximum flood depths, peak arrival time and flood wave travel time, which are the most important parameters for practicing field engineers and planners.

2.8Conclusion

From the review of the past works following observations have been made.

i. Dam break problem remains a topic of continued interest since 1892 which started with simple case such as rectangular frictionless channels, till date for mathematical simulation of a real dam break flood with natural complex channels and floodplains.

ii. Quite a number of investigators have worked on analytical , numerical and experimental works on the problem

iii. It appears that large number of numerical solutions both in 1-D and 2-D models have been developed

iv. Most of the developed solutions are compared mostly with simple laboratory data

v. Comparison of those developed model in real field data are relatively quite less

vi. Some of the developed models are quite complex although results are not very reliable predicting underestimation of output.

vii. Those developed complex models both in 1-D and 2-D have taken lot of computational runtime and they are quite difficult for field engineers to handle.

Therefore in this proposed study efforts will be made to development of simple numerical models which may predict quite reasonable results to a safer prediction and at the same time, it will aimed for field engineers to handle the models to predict dam break flood in any real field problem.

Chapter-3

Dibang: Its Important Features, Topographic Characteristics and the Proposed Dam

3.1 Introduction

The mighty River Brahmaputra has been a continuing source of beauty and bounty for the seven northeastern states of India, since time immemorial, more specifically to Assam and its neighboring Arunachal Pradesh. The four main Himalayan Tributaries Siang, Dibang, Lohit and Subansiri contribute as much as 60% of the total flow of the Brahmaputra. With a view to take advantage of the large hydro potential of these tributaries and to derive benefits of flood control, irrigation, navigation, several dams have been proposed in the Northeastern region of India. The hydropower potential of India is estimated to be 1, 50,000 MW and only 20% of which has been harnessed till now. The development of hydro-potential would speedily facilitate economic development of the Northeastern region of India. In these states the hydropower potential is reckoned as 31% of the county's total. One among the proposed dams is the 288m high Dibang dam with an installed capacity of 3000 MW.

In this study, the hypothetical flood in river Dibang due to the failure of the proposed large dam has been considered .The reasons for considering Dibang Dam project as the case study are due to the following important facts:

(i) The instantaneous failure of large dam inundates the downstream to a great extent.

(ii) In downstream, several villages of Arunachal Predesh and Assam, reserved forests and two roads Guwahati-Pasighat and Tinsukia-Roing are there.

(iii) Data for analysis are available.

(iv) NHPC, the implementing authority is yet to carryout the Dam break analysis.

3.2 Important Features of Himalayan Rivers in Dam Break Analysis

Significant changes occur in

(i) Bed Elevation and Bed Slope

(ii) Bed Materials: Frictional Characteristics

(iii) Channel Cross Sections

3.3 Salient Features of Dibang Dam

3.3.1. Location of the dam

(i) Country: India

(ii) State: Arunachal Pradesh

(iii) District: Lower Dibang valley

(iv) Dam Site: Latitude: 28°20’7” N

(v) Longitude: 95°46’ 38” E

3.3.2. Hydrology

(i) Catchment area: 11276 km^2

(ii) Location:

a) Latitude: 28°11’ 50” N to 29°25’ 59” N

b) Longitude: 95°14’ 47” E to 96°36’ 49” E

(iii) Average annual rainfall: 4405 mm

(iv) Maximum temperature: 45° C

(v) Minimum temperature: 2° C

3.3.3. Reservoir

(i) Maximum water level: EL 548 m (above mean sea level)

(ii) Full reservoir level: EL 545 m(above mean sea level)

(iii) Area under submergence in full reservoir level: 40.09 km^2

(iv) Length of reservoir: 43km

3.3.4. Dam

(i) Type: Concrete Gravity Dam

(ii) Top Elevation of Dam: 550 m(above mean sea level)

(iii) Height of Dam above deepest foundation level: 288 m(above mean sea level)

(iv) Length of Dam at Top: 816.3 m

3.3.5. Power Generation

Installed Capacity: 3000 MW

3.4 Structure Failure

As per guidelines of dam break analysis of CADAM (1996) "There is no methodology for predicting the growth of a breach through a concrete or masonry structure. In broad terms the failure will be quick relative to the formation of a breach through an embankment dam and failure time will be taken as instantaneous"

Hence, for simulating the dam break flood, total removal of Dibang dam is assumed due to instantaneous failure.

3.5 Topographic Characteristics

The topographic characteristic varies significantly within the computation domain. The river Dibang passes its course from hills of Arunachal Pradesh to the plains of Assam. The river passes through deep gorges, terrains with pebbles and boulders and then through alluvial plains. Most of the portions on the downstream of the dam lie in the plains. In the plain, multiple stream channels namely Deopani, Iphipani, Gango, Siba, Sisiri and Siang are flowing there in the terrain near by the main channel of river Dibang. In between the streams several villages, dense forest and two main roads namely Guwahati-Pasighat road and Roing-Tinsukia road are there. The cross sectional data have been obtained with the help of National Hydro Power Corporation (NHPC) and National Productivity Council (NPC) of INDIA up to 63km downstream of the dam at its confluence with Brahamaputra. The channel of River Dibang in hills and plains are graphically presented in Figure 3.1 and Figure 3.2. Some photographs of River Dibang in different terrain conditions taken during field visit are presented below from figure 3.3 to figure 3.8.

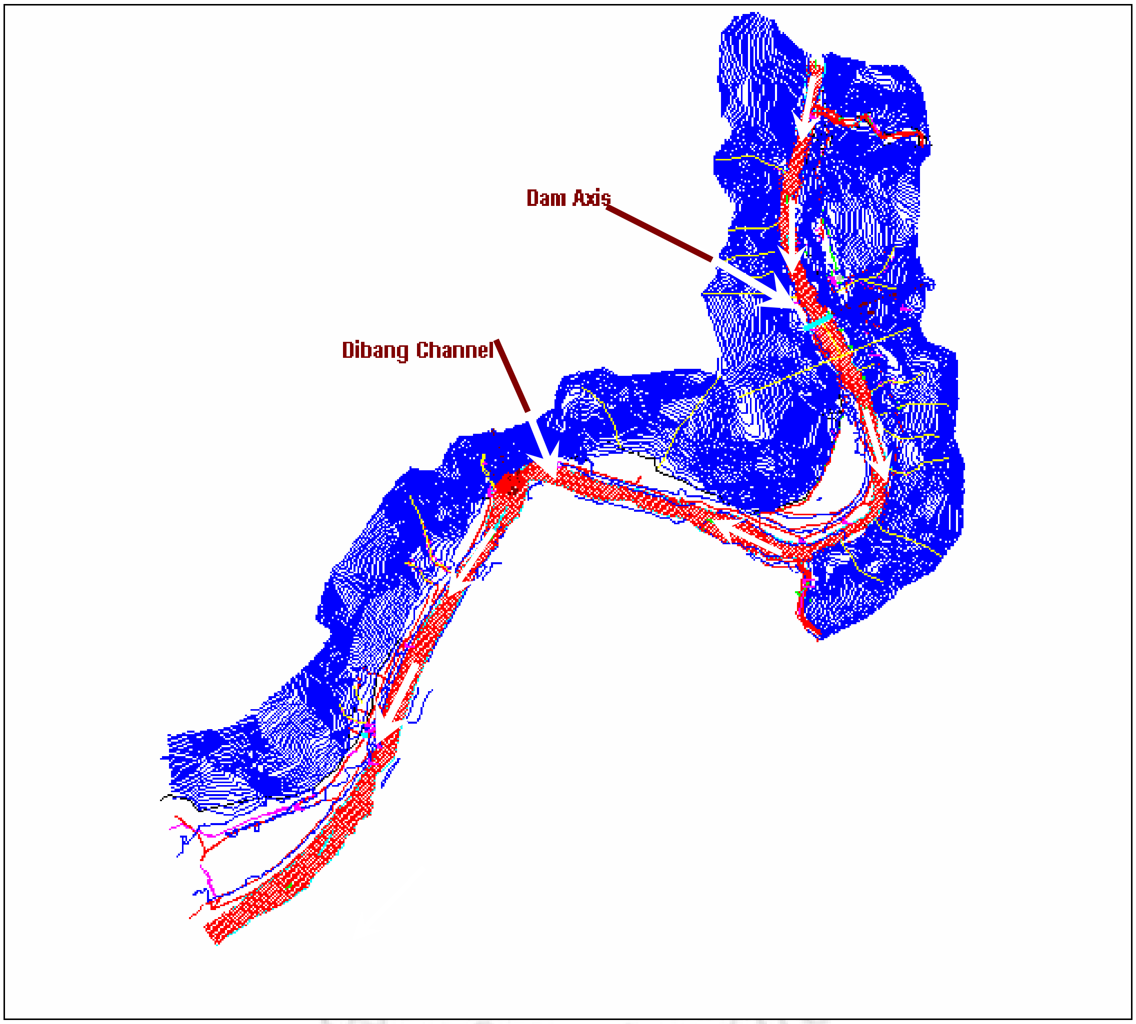

Figure3.1: The Channel of River Dibang in Hills

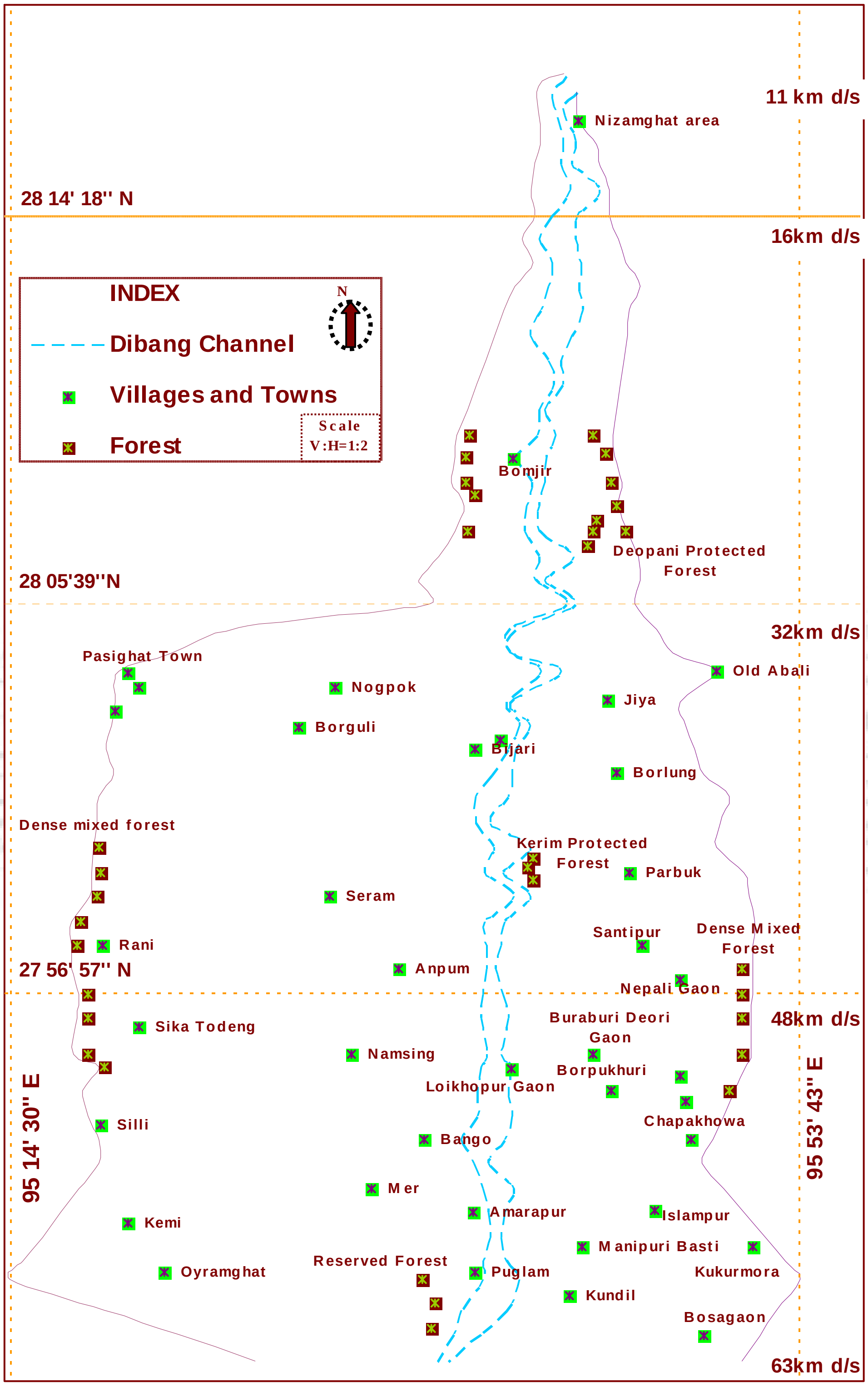

Figure3.2: The Channel of River Dibang enters Plains at 11 km Downstream the Dam

Figure3.3 Dibang in Deep Gorges

Figure 3.4 Dibang Approaching Wide Plains

Figure 3.5 Dibang Gradually Opens out at Foot Hills

Figure 3. 6 Dibang Gets Widen in the Plain


Figure3.7: Dibang Passing through Boulder Stages

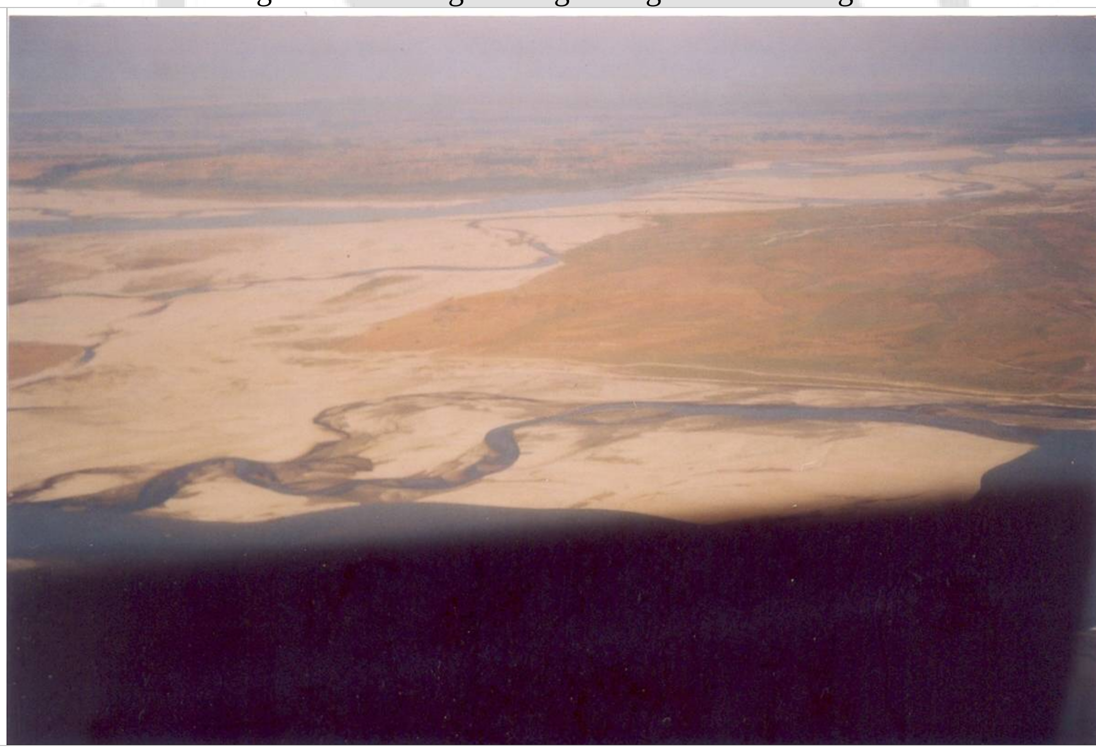
Figure3.8: Dibang Passing through Alluvial Plain

3.6 Conclusion

The field data presented on the above sections for features and topographic characteristics and through figures 3.1 to 3.8 indicate that the dam is quite high and the topography is very complex. It appears that the analysis of dam-break flood hydraulics of the proposed dam there will be interesting and challenging.

Chapter-4

Development of the Simulation Model

4.1. Introduction

Numerical simulation models are powerful tools to assess the impact of floods due to dam-failure events. However, one needs to take utmost care in developing and implementing these models in practical field, as possibility of having erroneous results exists (Viseu and Franco), particularly in the situations where:

(i) The dams are very high and the initial water difference between upstream and downstream is great

(ii) Sub critical and supercritical flows occur in the same routing river stretch

(iii) There are cross-section alterations (enlargements or narrowings)

(iv) The flood occupies a flood-plain

The proposed study taken here is the Dibang project where the reservoir extends up to 43km upstream of the dam and the channel meets the river Brahmaputra at 63 km downstream the dam, hence the computational channel is quite long i.e. 106 km. The elevation of the channel bed changes from 545m to 126.95m. The other most important features of Dibang dam are:

(i) Dam is very high (288m) and the initial water level difference between upstream and downstream is also high

(ii) Sub critical, mixed and supercritical flows are expected to occur in the same section at different time intervals or at different sections in the same time interval

(iii) The significant alterations of enlargement and narrowing of the cross-sections of the river exist as seen in the topographical features of the previous chapter.

(iv) The flood is going to occupy flood plain after the failure of the dam. This is also vivid from the topographical features.

Hence the mathematical representations and selections of the numerical schemes for simulation of the flood due to failure of such dam have to be made very carefully.

4.2. Mathematical Representations

4.2.1. Governing Equations

To obtain the basic equations of fluids in motion following philosophies are followed:

(i) To choose the appropriate fundamental physical principles i.e., conservation of mass, momentum and energy.

(ii) To apply these physical principles to suitably simulate the flow

(iii) To deduce the mathematical equations to represent the flow.

If a solid body is in translational motion, the velocity of each part of the body is same, but if the fluid is in motion, the velocity may be different at each location within the fluid. Hence to represent a moving fluid and to apply the fundamental physical principles one of the following models needs to be used:

a) **Non-conservative Form**

When the governing equations i.e. the continuity and momentum equations for the unsteady flow are deduced considering the fluid element which is moving along with the fluid such that same fluid particles are always inside it, the formulation is known as non-conservative form.

b) Conservative Form

In conservative formulation the mathematical model is represented considering the fluid element fixed in space with fluid moving through it.

4.2.1.1. One Dimensional Representation

The movement of the wave in the dam-failure situation is governed by gradually varied unsteady flow equations in open channel, i.e., the Saint-Venant (1871) equations.

This can be written in non-conservative form as follows:

$$\frac{\partial A}{\partial x} + A\frac{\partial V}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{4.1}$$

$$\frac{g}{b_m}\frac{\partial A}{\partial x} + V\frac{\partial V}{\partial x} + \frac{\partial V}{\partial t} = g(S_b - S_f) \tag{4.2}$$

And in conservative form as follows:

$$\frac{\partial U}{\partial t} + \frac{\partial F(U)}{\partial x} = S(U) \tag{4.3}$$

where:

$$U = \begin{Bmatrix} A \\ Q \end{Bmatrix} \tag{4.4}$$

$$F(u) = \begin{Bmatrix} Q \\ \dfrac{Q^2}{A} + gI_1 \end{Bmatrix} \tag{4.5}$$

$$S(u) = \begin{Bmatrix} 0 \\ gA(S_0 - S_f) + gI_2 \end{Bmatrix} \tag{4.6}$$

where x is direction parallel to the river, t is time, A is cross-sectional flow area, Q is discharge, V is the depth averaged flow velocity, b_m is mean cross sectional width, g is the acceleration due to gravity, S_0 is the bed slope, and S_f is the friction slope.

$$I_1 = \int_0^{h(x)} [h(x) - \eta]b(x,\eta)d\eta \tag{4.7}$$

$$I_2 = \int_0^{h(x)} [h(x) - \eta]\left[\frac{\partial b}{\partial x}\right]_{h=h_o} d\eta \tag{4.8}$$

I_1 and I_2 are cross-sectional moment integrals,

η is the integration variable representing the vertical distance to the bottom of the section,

b is the cross-sectional width at height η and h is the water depth above the bottom.

4.2.1.2. Two Dimensional Representations

In conservative form, the two dimensional representations are:

$$\frac{\partial U}{\partial t} + \frac{\partial G(U)}{\partial x} + \frac{\partial F(U)}{\partial y} = S(U) \tag{4.9}$$

Where: $$U = \begin{Bmatrix} h \\ V_x h \\ V_y h \end{Bmatrix} \tag{4.10}$$

$$G(u) = \begin{Bmatrix} V_x h \\ V^2{}_x h + gh^2/2 \\ V_x V_y h \end{Bmatrix} \tag{4.11}$$

$$F(u) = \begin{Bmatrix} V_y h \\ V_x V_y h \\ V^2{}_y h + gh^2/2 \end{Bmatrix} \tag{4.12}$$

$$S(u) = \begin{Bmatrix} 0 \\ gh(S_{ox} - S_{fx}) \\ gh(S_{oy} - S_{fy}) \end{Bmatrix} \tag{4.13}$$

Where h is the flow depth, V_x and V_y are the flow velocities in x and y directions respectively, g is the acceleration due to gravity S_{0x}, S_{oy} are the bed slopes in x and y directions respectively.

4.3. Numerical Scheme Formulation of 1-D Model

In modelling dam break floods in natural channels, the parameters of practical importance are the maximum water depths, peak arrival time and time of movement of the wave front. The ease of implementing a numerical scheme and runtime to simulate the flood are the important computational aspects. Numerical simulation using complex high-resolution scheme can bring finer computational accuracy. However, such accuracy is obtained at the cost of computational time, space and implementation efforts. Keeping these practical aspects into consideration, the performances of first order Diffusive finite difference (F.D.) scheme, second order modified two-step Predictor Corrector F.D. scheme and Total Variation Diminishing (TVD) MacCormack Predictor Corrector scheme are investigated here for computing flood movement due to failure of the proposed Dibang dam.

4.3.1. Conservative Formulation

4.3.1.1. First Order Diffusive Scheme

When diffusive scheme is applied to (4.3), the following equation is obtained (Cunge et al (1980) :

$$U_i^{n+1} = \alpha U_i^n + (1-\alpha)\frac{U_{i+1}^n + U_{i-1}^n}{2} - \frac{\tau}{2}(F_{i+2}^n - F_{i-1}^n) + \Delta t S_i^n \tag{4.14}$$

Where, $0 \leq \alpha \leq 1$ and $\tau = \frac{\Delta t}{\Delta x}$ (4.15)

Δt is computation time step; Δx is space difference in x-direction and α is weighting coefficient in diffusive scheme.

Macchione et al (2003) found in their numerical investigation that diffusive scheme gives increasingly accurate results as the value of coefficient α increases. For $\alpha = 0.75$, the results are only slightly less accurate than those obtained through Roe’s First-Order Upwind Scheme. Hence $\alpha = 0.75$ have been considered here.

4.3.1.2. Second Order Two Step schemes

A number of explicit second-order two-step schemes exist which have been collected by Lerat and Peyret under the following general structure (Peyret and Taylor 1990).The well known MacCormack scheme belongs to this same family.

Predictor: $\overline{U}_i = (1-\beta)U_i^n + \beta U_{i+1}^n - \theta\theta\tau({F^n}_{i+1} - F_i) + \Delta t {S_i}^n$ (4.16)

Corrector: $U_i^{n+1} = U_i^n - \tau(2\theta\theta)^{-1}[(\theta\theta-\beta)F_{i+1}^n + (2\beta-1)F_i^n + (1-\theta\theta-\beta)F_{i-1}^n + (\overline{F_i^n} - \overline{F_{i-1}^n})] + \Delta t\overline{S_i}$ (4.17)

β is the Weighting coefficient in Lerat and Peyret’s familyof Lax-Wendroff schemes and $\theta\,\theta$ is the Weighting coefficient in Lerat and Peyret’s family of Lax-Wendroff schemes

In MacCormack scheme (1969), $\theta\theta = 1$, $\beta = 0$

The MacCormack scheme is recommended by different investigators like Garcia et al. (1986), Garcia et al (1992), Fennema, (1990) et al, Rahman, et al (1998), and Aureli et al (2004)) for unsteady gradually varied flow computations.

In this study, following two-step second order schemes examined are: Modified Predictor Corrector i.e., the well-known MacCormak scheme in a slightly modified form, and Total Variation Diminishing (TVD) MacCormack scheme.

4.3.1.2.1. Modified Predictor Corrector

The well-known MacCormack scheme in a slightly modified (Sarma 1999) form has been used here. The set of governing equations used for modelling purpose has an inherent property of signal propagation i.e., in case of sub-critical flow the information comes both from upstream and downstream, while the information comes only from upstream if the flow is super-critical. Numerical investigations have shown that better results are produced if the direction of differencing in the predictor step is the same as that of the movement of wave front (Choudhury 1990).

Hence, in the sub-critical region backward F.D. approximation is used for predictor step and forward F.D. approximation is used in the corrector step. In the super-critical region as the control is always on the upstream side, use of forward F.D. approximation in the corrector step is omitted to eliminate erroneous influence of downstream flux on the computed values. This simple technique has made application of the scheme possible in the mixed flow regions also.

When it is applied to (4.3), the predictor step is given as:

Predictor step:

$$UP_{i} = U_i^n - \tau(F^n{}_i - F^n_{i-1}) + \Delta t S_i{}^n \tag{4.18}$$

Corrector step:

It is applied to each node on the basis of the following conditions

(i) If, $v \le \sqrt{gh}_{\,i}$, for **sub-critical** flow

$$UC_{i} = U_i{}^n - \tau\left[FP_{i+1} - FP_{i}\right] - \Delta t(SP_i) \tag{4.19}$$

(ii) If, $v \ge \sqrt{gh}_{\,i}$, for **supercritical** flow

$$UC_i = UP_i \tag{4.20}$$

Finally, the U vector containing value of primitive flow variables in the next time step is calculated as:

$$U_i^{n+1} = \frac{UP_i + UC_i}{2} \tag{4.21}$$

4.3.1.2.2. TVD MacCormack Scheme

Among the Second-order two-step family, TVD MacCormak with Van Leer's Limiter is reported (Macchione et al (2003)) to demonstrate excellent behavior. Here the value of U_i^{n+1} given by the corrector step (4.17) is corrected by adding the following TVD term:

$$\tau(T^n_{i+1/2} - T^n_{i-1/2}) \tag{4.22}$$

Where:

$$T^n_{i+1/2} = \sum_{k=1}^{2} Dd^{(k)}_{i+1/2}\,\delta w^{(k)}_{i+1/2}\, r^{(k)}_{i+1/2} \tag{4.23}$$

The term $Dd^{j}_{i+1/2}$ which can be considered an artificial dissipation term, has the following expression:

$$Dd^{j}_{i+1/2} = \frac{1}{2}[(1-\phi)|\overline{\lambda}|(1-\tau|\overline{\lambda}|)]^{j}_{i+1/2} \tag{4.24}$$

In which ϕ = limiter allowing the TVD condition to be satisfied. The limiter ϕ is calculated as a function of the ratio ρ of the characteristic variations

$$\rho_{i+1/2} = \frac{\delta w_{i+1/2-\mathrm{sgn}(\overline{\lambda_{i+1/2}})}}{\delta w_{i+1/2}} \tag{4.25}$$

In the present study the Van Leer limiter is considered:

$$\phi(\rho) = \frac{\rho + |\rho|}{1+\rho} \tag{4.26}$$

The variations $\delta\, w_{(1)(2)}$ at the point(i+1/2) are expressed as follows:

$$\delta w_{(1)(2)} = \pm\left[(Q_{i+1} - Q_{i+1}) + \left(-\frac{\overline{Q_{i+1/2}}}{\overline{A_{i+1/2}}} \pm \overline{c_{i+1/2}}\right)(A_{i+1} - A_i)\right] \tag{4.27}$$

$\overline{r}^{(j)}$ are the approximate Jacobian matrix eigenvectors. For the construction of such a matrix for the case of system (4.3) the following averaged variables should be considered for each cell (i,$i+1$) (Garcıa Navarro[11] et al. 1992): $\overline{A_{i+1/2}} = \sqrt{A_i A_{i+1}}$ (4.28)

$$\overline{Q}_{i+1/2} = \frac{\sqrt{A_i}\,Q_{i+1} + \sqrt{A_{i+1}}\,Q_i}{\sqrt{A_i} + \sqrt{A_{i+1}}} \tag{4.29}$$

The averaged celerity is computed as follows:

$$\overline{c} = \sqrt{g\frac{I_{1_{i+1}} - I_{1_i}}{A_{i+1} - A_i}} \quad \text{When } A_{i+1} \neq A_i \tag{4.30}$$

$$\overline{c} = \sqrt{\frac{\frac{1}{2}g(A_{i+1} - A_i)}{\frac{1}{2}(b_{i+1} - b_i)}} \quad \text{When } A_{i+1}=A_i \text{ or } (I_{1_{i+1}} - I_{1_i})(A_{i+1} - A_i) < 0 \tag{4.31}$$

So that the approximate Jacobian matrix is characterized by the following eigen values and eigenvectors:

Eigenvalues:

$$\bar{\lambda}_1 = \frac{\bar{Q}}{\bar{\bar{A}}} + \bar{c} \tag{4.32a}$$

$$\bar{\lambda}_2 = \frac{\bar{Q}}{\bar{\bar{A}}} - \bar{c} \tag{4.32b}$$

Eigenvectors:

$$\bar{r}^{(1)} = \frac{1}{2\bar{\bar{c}}}[1, \bar{\lambda}_1]^T \tag{4.33a}$$

$$\bar{r}^{(2)} = \frac{1}{2\bar{\bar{c}}}[1, \bar{\lambda}_2]^T \tag{4.33b}$$

4.3.2. Non-Conservative Formulation

Here the first order Diffusive and second order modified Predictor Corrector schemes are formulated in non conservative forms to examine whether the performance of the schemes differs depending on the form of numerical formulation of the unsteady flow.

4.3.2.1. Diffusive Scheme

Rearranging Equations (4.1) and (4.2) at any point 'i' in known time step n,

$$A_i^{n+1} = \alpha A_i^n + (1-\alpha)\frac{A_{i+1}^n + A_{i-1}^n}{2} - \frac{\tau}{2}\left[V_i^n (A_{i+1}^n - A_{i-1}^n) + A_i^n (V_{i+1}^n - V_{i-1}^n)\right] \tag{4.34}$$

$$V_i^{n+1} = \alpha V_i^n + (1-\alpha)\frac{V_{i+1}^n + V_{i-1}^n}{2} - \frac{\tau}{2}\left[\frac{g}{b_m}(A_{i+1}^n - A_{i-1}^n) + V_i^n (V_{i+1}^n - V_{i-1}^n) - g\Delta x(S_{0i} - S_{f,i})\right] \tag{4.35}$$

4.3.2.2. Modified Predictor Corrector

Rearranging Equations (4.1) and (4.2) at any point '*i*' in the domain, the values of the flow parameters are obtained by the following predictor and corrector equations:

Predictor Step:

$$VP_i^{n+1} = V_i^n - \tau\left[\frac{g}{b_{m,i}}(A_i^n - A_{i-1}^n) + V_i^n(V_i^n - V_{i-1}^n) - g\Delta x(S_{o,i} - S_{f,i})\right] \quad (4.36)$$

$$AP_i^{n+1} = A_i^n - \tau\left[V_i^n(A_i^n - A_{i-1}^n) + A_i^n(V_i^n - V_{i-1}^n)\right] \quad (4.37)$$

Corrector step:

It is applied to each node on the basis of the following conditions

(i) If, $v \le \sqrt{gh}_i$, **for sub-critical flow**

$$VC_i^{n+1} = V_i^n - \tau\left[\frac{g}{b_{m,i}}(A_{i+1}^n - A_i^n) + V_i^n(V_{i+1}^n - V_i^n) - g\Delta x(S_{0,i} - S_{f,i})\right] \quad (4.38)$$

$$AC_i^{n+1} = A_i^n - \tau\left[V_i^n(A_{i+1}^n - A_i^n) + A_i^n(V_{i+1}^n - V_i^n)\right] \quad (4.39)$$

(ii) If, $v \ge \sqrt{gh}_i$, **for supercritical flow**

$$VC_i^{n+1} = VP_i^{n+1} \quad (4.40)$$

$$AC_i^{n+1} = AP_i^{n+1} \quad 4.41$$

Finally, velocity and flow area at unknown time step "n+1" are :

$$V_i^{n+1} = \frac{VP_i^{n+1} + VC_i^{n+1}}{2} \quad (4.42)$$

$$A_i^{n+1} = \frac{AP_i^{n+1} + AC_i^{n+1}}{2} \quad (4.43)$$

Stability is assured by the Courant–Friedrichs–Lewy condition:

$$C_r = \frac{\max(|v| + c)}{\Delta x/\Delta t} \le 1 \quad (4.44)$$

Where C_r is the Courant number, v is the velocity; c_x is celerity $(= \sqrt{gh})$ and $\max(|v| + c)$ stands for the maximum value over the whole range of grid points

To have low value of Courant number "Cr" the time step Δt is kept low here as Δx has to be made small to take into account the highly non prismatic structure of the natural river channel. In case of natural river channel like River Dibang, it is highly non- prismatic and the breath of the computational channel changes significantly within small distance.

4.4. Numerical Scheme Formulation of 2-D Model

The 2-D model for Dibang dam break flood simulation has been developed using two-step modified predictor corrector scheme

4.4.1. Second Order Two Step Modified Predictor Corrector Scheme

For the modeling purpose, two dimensional shallow water equations of gradually varied flow have been used. Theses equations have an inherent directional property of signal propagation. For sub critical flow, information comes from both upstream and downstream, while information comes only from upstream in case of super critical flow. Direction of propagation can be taken into account, as already explained in section 4.3.1.2.1. Such technique will make the scheme applicable in the flow region where both sub critical and super critical flow may be present simultaneously.

In the sub critical region, the backward F.D approximation has been used for the predictor step and forward F.D approximation has been adopted for the corrector step. In super critical region, as the control is always on the upstream side, use of forward F.D approximation in the corrector step has been omitted to eliminate influence of downstream flux on the computed value.

Predictor step: $UP_{i\,j} = U^n_{i,j} - \tau(E^n{}_{i,j} - E^n_{i-1,j}) - \tau(F^2{}_{i,j} - F_{i,j-1}{}^2) - \Delta t S_i{}^n$

Corrector step: It is applied to each node on the basis of the following conditions

(iii) If, $\sqrt{Vx^2{}_{i,j} + Vy_{i,j}{}^2} \le \sqrt{gh}_{\,i}$, for sub-critical flow

Corrector: $UC_{i\,j} = U^n_{i,j} - \tau(E^n{}_{i+1,j} - E^n_{i,j}) - \tau(F^2{}_{i,j+1} - F_{i,j}{}^2) - \Delta t(SP_{i,j}{}^n)$ (4.45)

(iv) If $\sqrt{Vx^2{}_{i,j} + Vy_{i,j}{}^2} \ge \sqrt{gh}_{\,i}$, for super-critical flow

Corrector: $UC_{i,j}=UP_{i,j}$ (4.46)

Finally, the U vector containing value of primitive flow variables in the next time step is calculated as: $U^{n+1}_{i,j} = \frac{UP_{i,j} + UC_{i,j}}{2}$ (4.47)

4.5. Programming Tool and Deveopment of the GUI Software for Dam Break Flood

4.5.1. Requirement of the Software

Numerical schemes are required to apply to those problems where equations representing the physical situations are so complex that an exact mathematical solution is virtually impossible. Dam break flood in real floodplain topography is also a problem of such kind. Implementation of any scheme for solution of this vexed problem, calls for repetitive solution of quite a large number of grid points. For example, the case taken here for simulation has been adopted for 43km up stream and 63 km downstream of the dam with a total longitudinal length of 106 km. In lateral direction the length of the flow domain is 63 km. The total number of computational grid points used in the1D model and 2D model are 1001 and 601601 respectively. Therefore, for obtaining the required flood parameters at different important downstream places due to failure of dam, development of an efficient computer model is essential.

4.5.2. Description of the Software

On the basis of the Governing equations as described above, graphical user interface (GUI) software has been developed in Visual Basic. The visual basic has been used as programming language as it has enabled the author to get graphical output in desired form without any additional graphical package.

All computations have been done elegantly by using matrix operation, with the help of three-dimensional array in the 2D model, two-dimensional array in the 1D model. As computations need considerable time, computed result after every time step is stored in data file and it is read as initial data for the next time step. Subprograms GROUND and INITIAL have been used with the main program. Subprograms GROUND provides all information of real field data. The subprogram INITIAL computes the initial profile with analytical solution of Ritter.

4.5.3. Software Outputs

It provides the outputs in 2 forms:

4.5.3.1 The GUI Outputs

The programs have been developed in such way that one can feed the necessary data in an interactive way. User can also specify in which form he/she wants to get the graphical output. Field engineers or planners can use the developed software without much knowledge of computer programming.

Necessary data are fed in an interactive way and the graphical outputs in desired forms have been obtained as presented below.

The Graphical Outputs of the Developed Software:

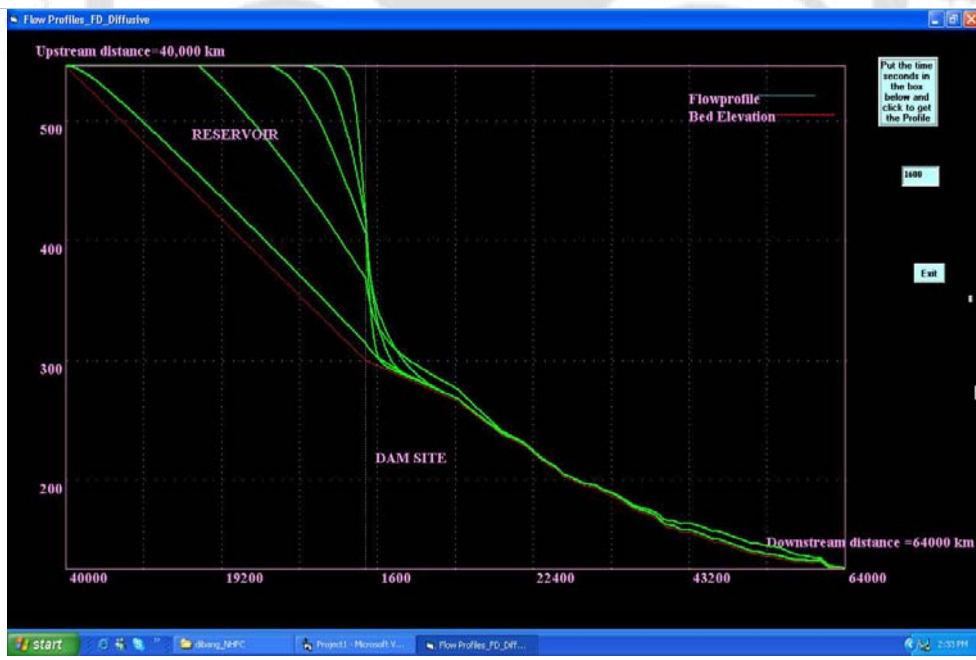

Figure4.1: Flow Profiles in River Dibang after 10, 40,100,300 and 1000 seconds of the Dam Failure

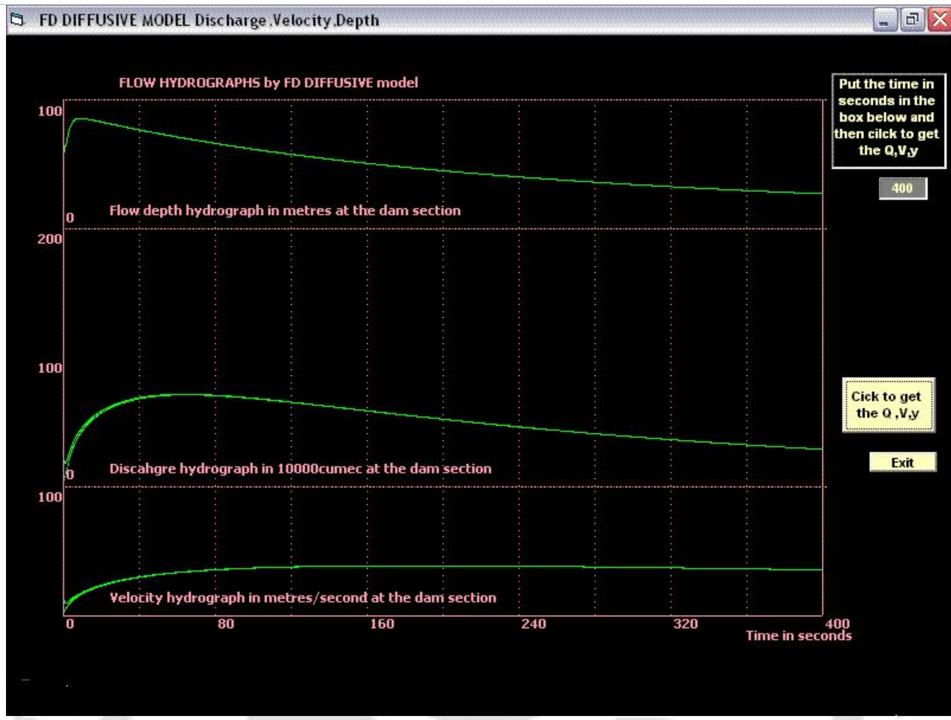

Figure4.2: Depth, Discharge, Velocity Hydrographs at Dam Section

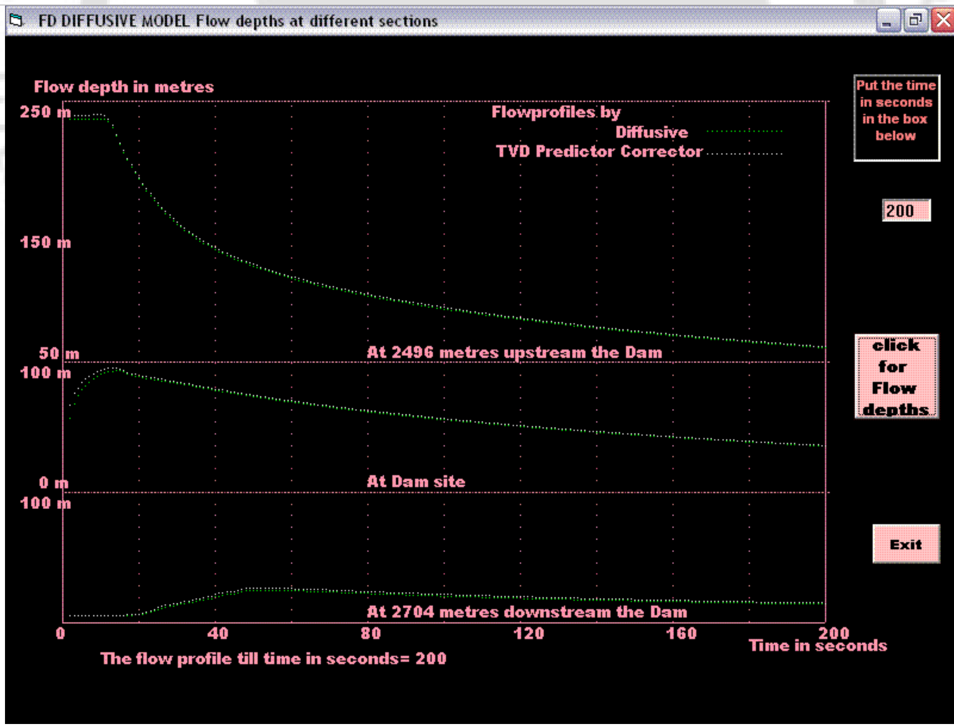

Figure4.3: Depth Hydrographs at Different Sections

4.5.3.2 The Data Stored in Text Files

The values of the important computed flood parameters are also stored in text files using "WRITE OUTPUT as file "txt" statements. These output data file can be imported to other graphical presentation packages. For example, author has imported output data files and made some graphical representations using MATLAB and Excels sheets as presented below:

Graphical Representations using MATLAB Importing the Results Stored in Text Files:

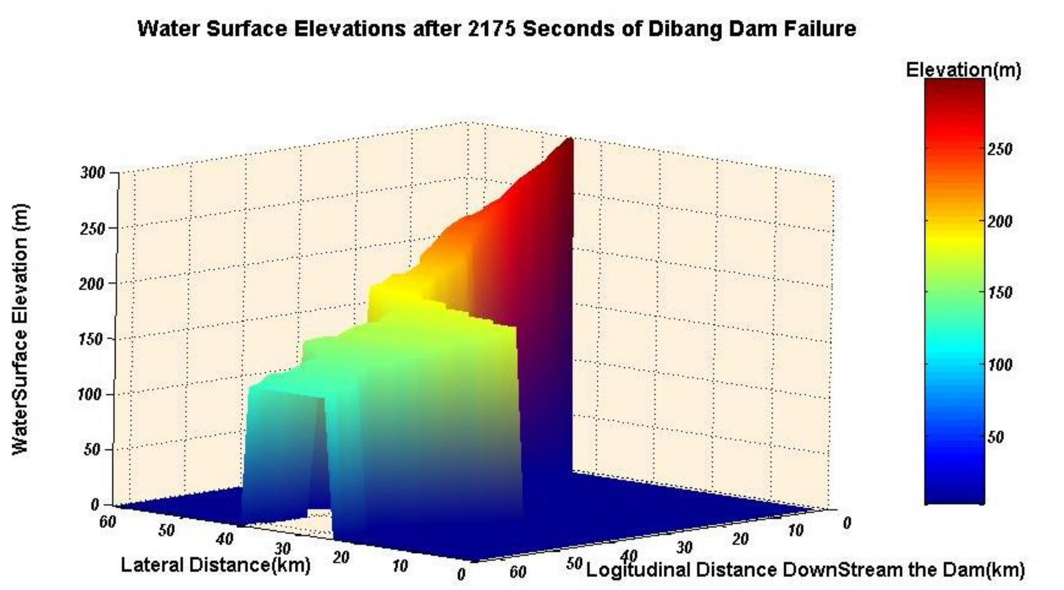

Figure4.4: Computed Flow Surface at 2175 seconds by the 2D Model

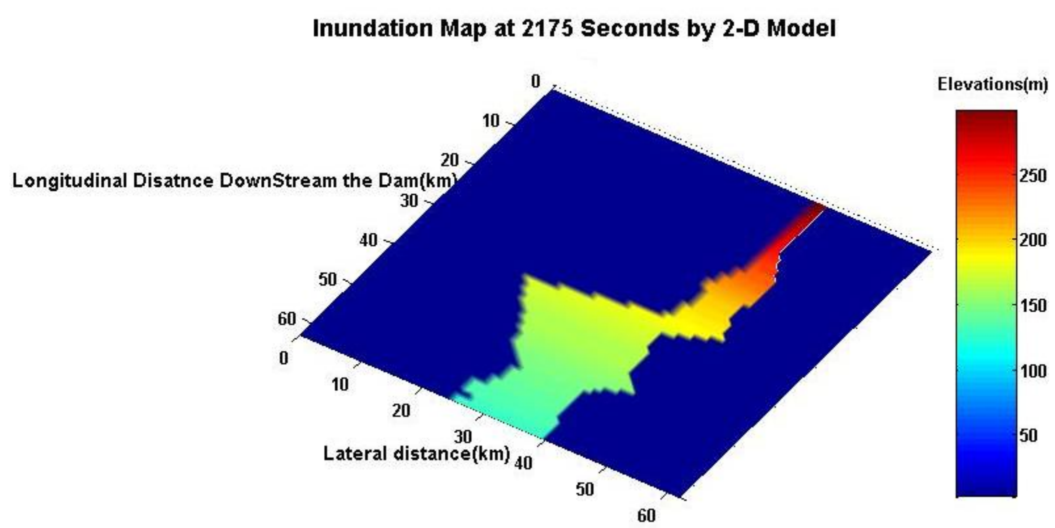

Figure4.5: Inundation Map at 2175 seconds showing the Water Surface Elevations at various Downstream Points

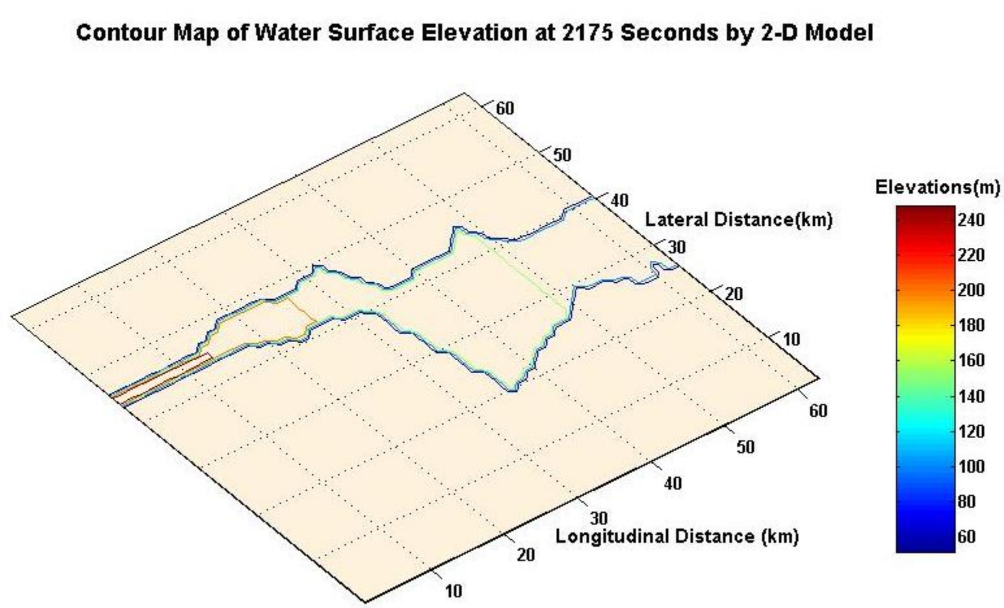

Figure4.6: Water Surface Elevation Contours at 2175seconds by 2-D Model

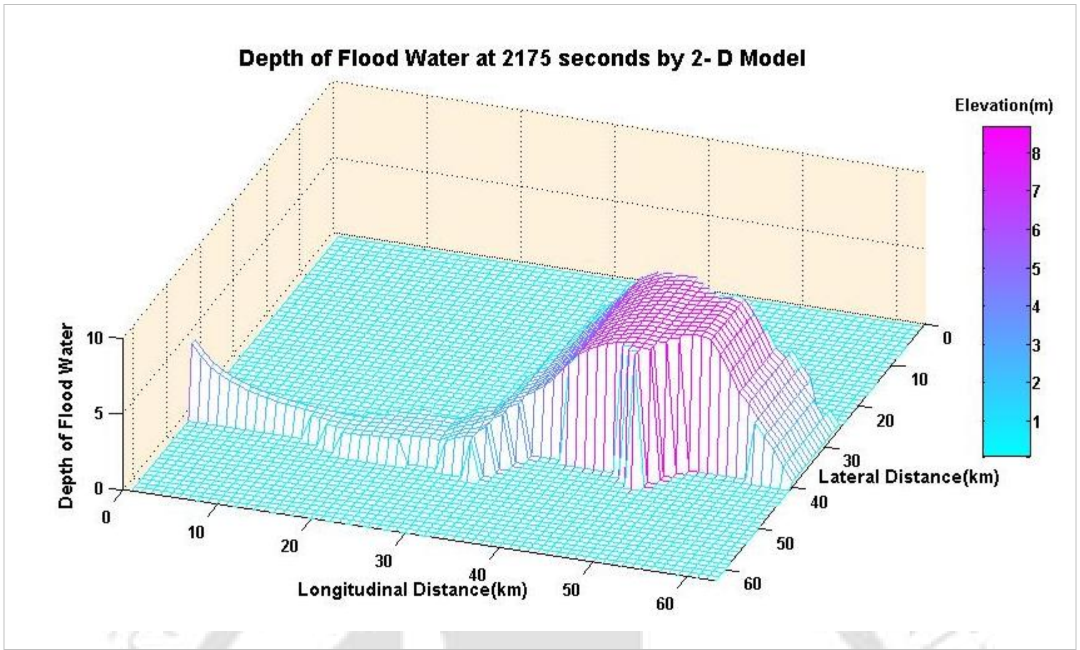

Figure4.7: Predicted Flood Depths Downstream after 2175 seconds of Failure of Dibang dam

Graphical Representations using Excel Sheets Importing the Results Stored in Text Files:

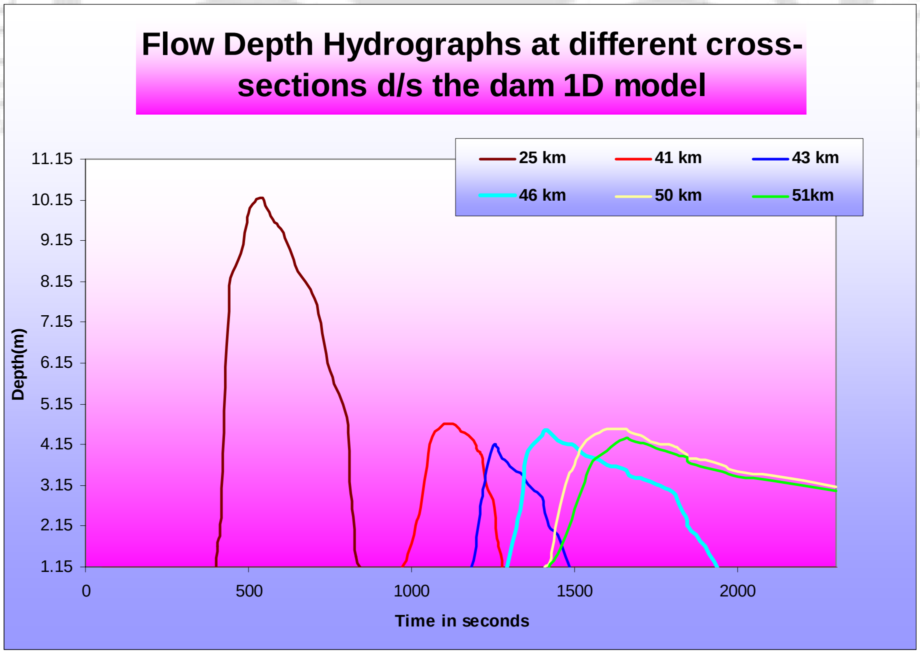

Figure4.8: Predicted Depths Downstream after 2175 seconds of Failure of Dibang Dam

4.6. Application of the Numerical Models to the Dam-Break Flood in River Dibang

The numerical models as described in the previous section 4.4 with conservative and non –conservative formulations have been applied to the river channel of Dibang to select the most suitable one for simulating flood due to failure of a large dam in a river with complex topography.

4.6.1. Analysis of the Results with respect to Practical Aspects and Formulations of the Governing Equations

The numerical simulations with the above mentioned explicit FD schemes are done for both conservative and non-conservative formulations of unsteady flow.

The conservative formulation of all the FD methods used in this study appears to produce much better flow profiles. From practical point of view considering factors such as wave propagation time, depth of flow; peak arrival time, simulated flood profiles by these FD schemes are quite comparable to one another(figure 4.9). On the other hand in the non-conservative formulations of these FD schemes, the MacCormack scheme although not failed, provides an oscillatory flow profile (figure 4.11). The diffusion scheme in non-conservative form is not only unstable but also fails to simulate the flood after 22 second(figure 4.10) The difference of the numerical solutions obtained here is mainly due to the dependable variable terms considered in the governing equations. In non-conservative form of the flow equations, i.e., in equation (4.1) and (4.2), the flow area and the velocity of flow are taken separately as dependable variable where as in conservative form, in equation (4.3) the variation of discharge, i.e. the variation of product of sectional area and velocity with respect to spatial distance is considered.

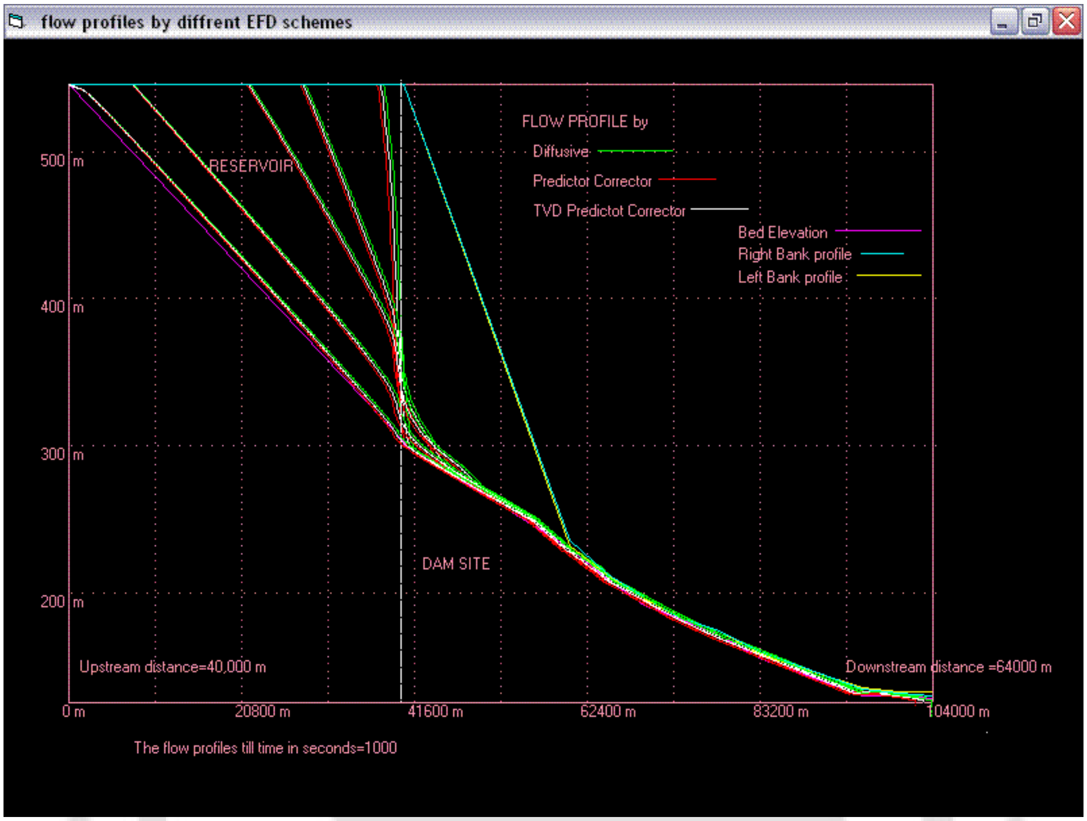

Figure 4. 9: Flow Profiles after Different Time Steps of the Instantaneous Failure of the Dam by the different Explicit Finite Difference Schemes

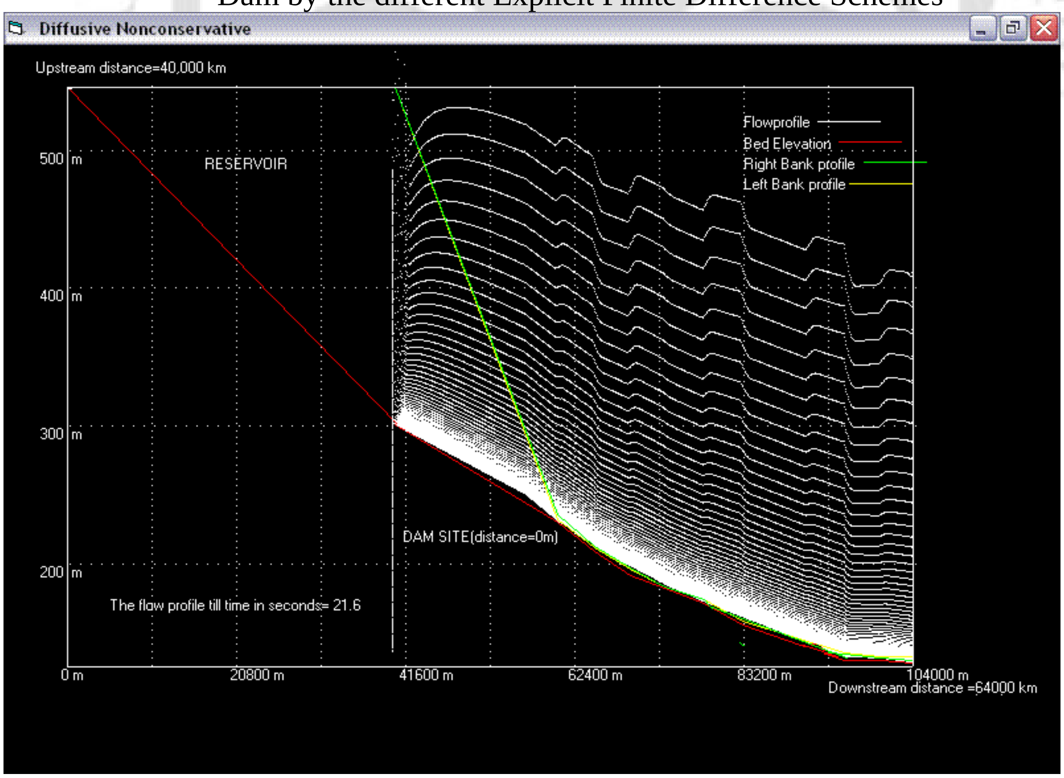

Figure 4. 10 Flow Profiles by Non-conservative Formulation of Diffusive Scheme.

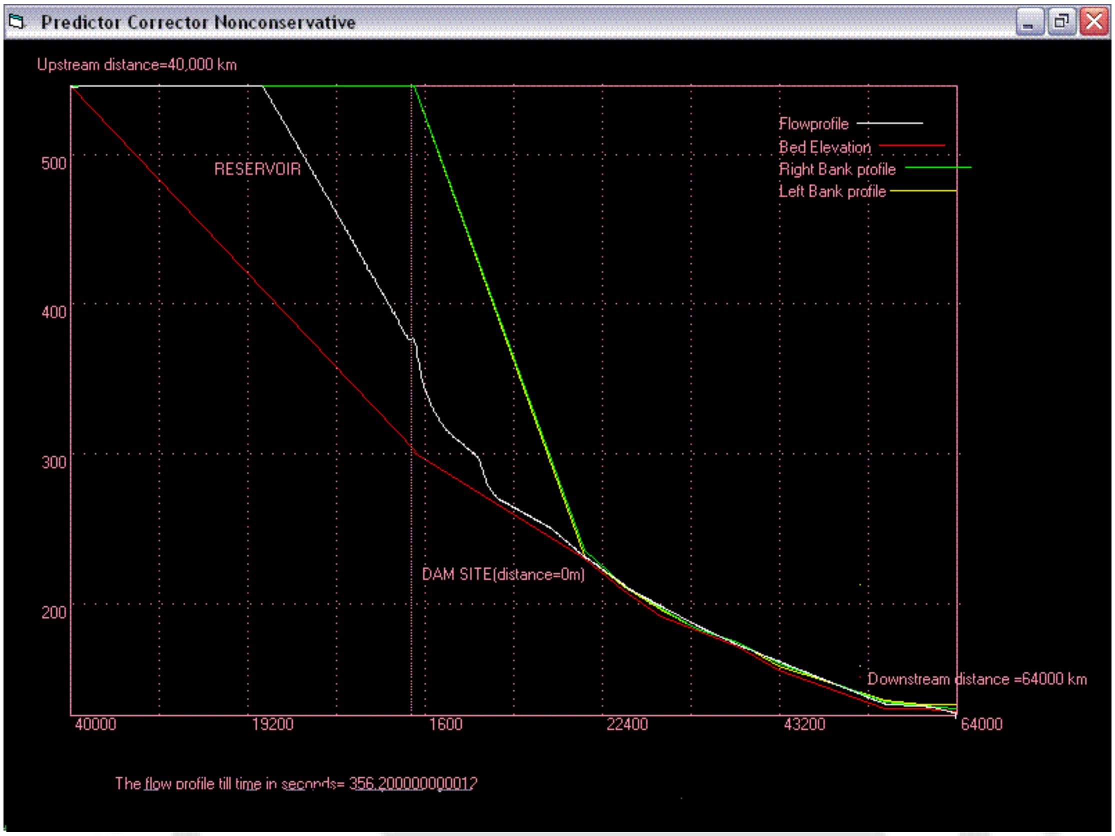

Figure 4. 11: Flow Profile by Non-conservative Formulation of Predictor Corrector Scheme

In figure 4.12 the plot of flow variables up to 200 second has been shown for dam failure at three different cross-sections i.e. one at dam site, one at 2496m upstream of the dam and another at 2706m downstream of the dam. It is clear from this plot that the changes in discharge and velocity with respect to spatial distance and time are quite less compared to the flow area. The shapes of the flow area hydrographs at the three different sections are observed to be completely contrasting. At upstream the flow area decreases with time with a reducing rate while at downstream it increases gradually with time. At dam site first it rises sharply with time and then falls down gradually to more or less constant value. But in case of discharge and velocity hydrographs, similar pattern is maintained in all the three sections. Therefore, numerical model with non-conservative formulation, where the variation of cross sectional area with space is computed separately, becomes unstable for dam break flow in such non-prismatic natural channel, as the error associated with the calculation of area gets compounded with time. It shows that the numerical formulations of such flow problem should be in conservative form. In this study it is found that the form of the governing equations is more important than the numerical scheme adopted.

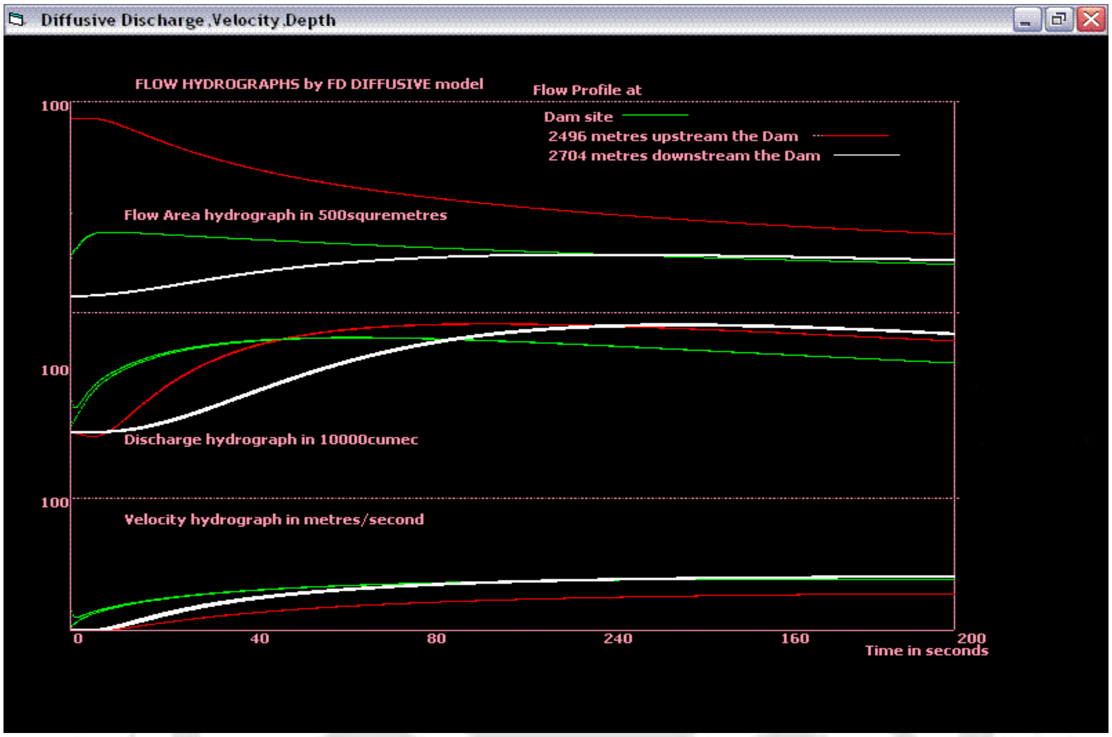

Figure 4. 12: Comparison of the flow hydrographs at different sections by Diffusive Scheme.

4.6.2. Comparisons of Computational Aspects

Time taken by different Schemes for simulation of the flow up to 1hour after dam failure, in a processor of (Pentium 4, 2.00 GHz, 512 MB RAM) specification are:

Diffusive scheme	39 minutes, 14.65 seconds
Modified predictor corrector	54 minutes, 56.51 seconds
TVD MacCormack	2 hours, 29 minutes, 7seconds

The flood in the real cases has to be simulated for a quite long period of time over river stretches of several kilometers long for proper flood management. Therefore, the runtime of the numerical formulation is one of the most important parameters to be considered. Among the above three FD methods the diffusion scheme is the easiest to implement and most advantageous even from runtime point of view. But there is no significant difference among the flow depths and time of wave propagations when schemes are formulated in conservative form.

4.6.3. Flow Conditions

It has been reported (Choudhury 1990, Jin 1997 et al) that applicability of the numerical schemes is difficult when sub critical and super critical flows are present either simultaneously in different parts of the channel or if they occur in the same section in sequence at different times. Models developed in this study using conservative form have shown their capability of handling mixed flow region. Figure 4.13 illustrates the flow conditions in the entire flow domain considered in this study. Here mixed flow condition has been observed, where sub critical flow changes to high supercritical, with a moving sub/super critical interface. This of course is obviously an expected case for a real large dam break case. The Diffusive scheme which is simplest among FD schemes also provides much better results even in complex flow situations, provided it is formulated in conservative form with small spatial grid points to maintain the high non prismatic nature of the complex real river channel.

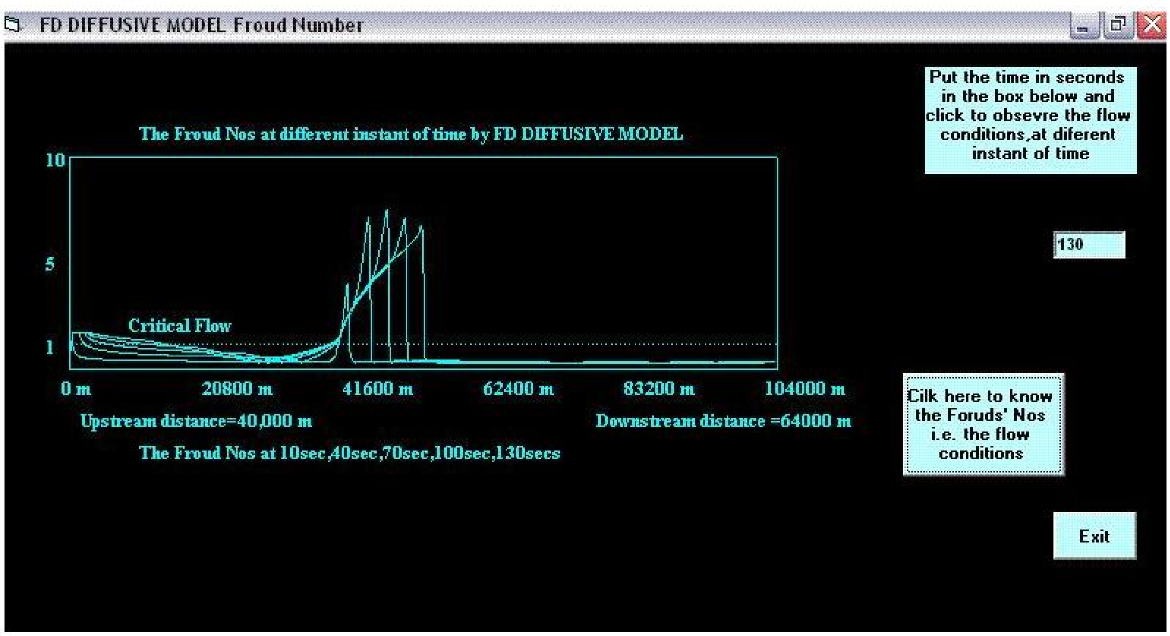

Figure 4. 13: Flow Conditions at Different Time, i, e., Froud Number of the Flow

4.7. Conclusion

It has been observed that the FD diffusive scheme formulated with non-conservative form of the governing equations fails after elapse of a few seconds (22 sec in the case of Dibang Dam). The two-step modified predictor corrector scheme with non-conservative formulation has produced oscillatory profiles. All the three numerical schemes considered in this study have performed satisfactorily when governing equations are in conservative form. Therefore, this study has clearly demonstrated the merit of using conservative form for computing dam-break flow in a highly non-prismatic natural channel.

The significance and need of using numerical refinement by applying corrections such as “TVD” limiters in real field situations are investigated. The use of TVD to the second order scheme has made the implementation complex. Run time is also burdensome even in one-dimensional calculation for the real field situation. Moreover, the refinement achieved in the flow profile by applying TVD correction may not be quite significant in a natural channel, as there always remains an obvious error in the input terrain data. Average values for different input parameters such as bed slope, channel width, and channel roughness are generally taken between two successive grid points. The application of the presented simple FD schemes i.e., first order diffusive and second order modified predictor-corrector, is quite justified as observed from the computed profiles in real riverbed topography. Another interesting observation of the investigation is that the numerical computation of unsteady flow profile depends to a great extent on the formulation of the governing equations. Use of conservative formulation of simple explicit FD schemes for computing dam break flood in complex real river topography is therefore, advantageous from practical and other points of view when compared to the higher order explicit schemes which make the run time longer and implementation complex.

Chapter-6

1-D and 2-D Simulation of Dam Break Flood Representing River Valley with Different Computational Channels

6.1. Introduction

Running of a one-dimensional model requires more personal interpretation of the data to get more reliable solutions (Soares et al 1999). The simulated output depends a lot on the valley or channel assumptions. A simplified channel is generally considered for the computation of flood parameters, in a natural channel. Such simplification may lead to erroneous estimation of the important parameters such as maximum depth, time of peak arrival, maximum velocity and inundated area. Therefore, due emphasis should be given in the selection of an appropriate computational channel while simulating a real dam break flood.

Development of a 2-D model for a dam break flood in real flood plain topography needs detailed ground data. Generally extensive field data for ground elevations are difficult to get in many cases. In those cases, there is no other alternative than to do the computations using 1-D model. Hence the comparison of flood prediction using 1-D model and 2-D model is a requirement for appropriate computations of complex real field Dam break floods. Extensive ground data are available for Dibang dam. Therefore, both 1-D and 2-D numerical models have been developed with an aim to evaluate the relative computing advantages or disadvantages, effort required for preparing input data and the last but most important one the comparisons of the prediction of the important flood parameters.

In this chapter three 1-D models and one 2-D numerical model have been developed for simulating the flood, due to the hypothetical failure of the proposed large gravity dam, in the natural floodplain topography of Dibang.

The analysis and comparisons of the results obtained by the models considering the different representation of the floodplain topography are presented with respect to the following important points:

a) Parameters of practical importance:
   - (i) Time of first arrival of flood water
   - (ii) Peak water surface elevation
   - (iii) Time of arrival of peak water level
   - (iv) Depth of flood water
   - (v) Extent of inundation

b) Important computational aspects:
   - (i) The ease of implementation
   - (ii) Runtime to simulate
   - (iii)Input data preparations

6.2.Formulation of 1-D Models with Natural Channel consideration of Dibang

Three 1D models have been developed for simulating the dam break flood in the natural floodplain topography of Dibang.

In one approach, the predictions are made by adopting a computational channel, which considers the whole floodplain downstream of the dam when the river Dibang enters the plain. In another approach the original simplified river channel of Dibang has been considered. To get insight into the effect of the channel characteristic on the computed results another model considering the floodplain with composite cross-sections has also been developed. Composite channel has been taken in the computation as soon as river Dibang changes its course from hills to the plains (11km downstream the dam site). The predictions of dam break flood by all the models have been compared and analyzed to assess their performance in the complex natural floodplain topography of Dibang.

6.2.1 Input Data

The input data in the programs are given in the form of regularly spaced grids. To accomplish it, the actual data acquired at convenient chainage points on the riverbed is linearly interpolated. The total channel reach is represented with a total number of 1000 grid points. The inputs for Dibang dam project can be summarized as follows:

(i) Elevation of River Bed: varies from values 545.00m -127.65m

(ii) Down stream boundary: 63km (up to confluence of the Brahmaputra).

(iii) Extension of the reservoir (upstream boundary): 43km

(iv) Channel roughness: (Manning's coefficients) varies from values 0.03- 0.035

(v) Floodplain roughness: (Manning's coefficients) varies up to 0.8

6.2.2 Initial and Boundary Conditions

At upstream of the dam the initial profile has been considered as the surface of the still water stored in the reservoir at maximum storage level. For computational advantage, a water depth of 0·1m is assumed in the dry downstream portion. Thus the formulation considers Drichlet conditions at u/s and d/s reaches of the computational domain. To start the numerical computation an initial profile is computed by Ritter's equation up to 1sec since failure of the dam has been introduced. Boundary values are obtained with the help of method of characteristics. Positive characteristic equation is solved simultaneously with the condition imposed by the boundary for downstream end condition and the negative characteristic equation is solved simultaneously with upstream end condition for the upstream boundary. Mathematically, it is represented as:

In upstream,

(i) $CU_{(t+\Delta t)}(i) = C1(i) + (dxl/\Delta x)(C1(i) - C1(i+1))$

(ii) $VU_{(t+\Delta t)}(i) = V1(i) + (dxl/dx)(V1(i) - V1(i+1)) - 2(C1(i-1) - C1(i)) + g(S_b(i) - S_f(i))\Delta t$

In downstream,

(i) $CD_{(t+\Delta t)}(i) = C1(i) + (dxr/\Delta x)(C1(i) - C1(i-1))$

(ii) $VU_{(t+\Delta t)}(i) = V1(i) + (dxr/\Delta x)(V1(i) - V1(i-1)) - 2(C1(i) - C1(i-1)) + g(S_b(i) - S_f(i))\Delta t$

where, $dxl = (V1(i) - C1(i))\Delta t$. $dxr = (V1(i) - C1(i))\Delta t$ .CU and CD are the celerity in upstream and downstream boundary respectively. VU and VD are the velocity in upstream and downstream boundary respectively. C1 and V1 are the celerity and velocity in the previous time step

6.2.3 Channel Roughness

To choose a correct roughness coefficient is quite important in unsteady flow simulation in a natural channel. Standard values of Manning's roughness coefficient "n_m" for natural streams have been given by Chow (1959) and Choudhury (1993) as given below.

(i) As compiled from Chow (1959) "Open Channel Hydraulics", McGraw-Hill Book Company, New York

Table 6.1(a): Channel roughness values

Type of the channel	"n_m" value
Clean and Straight	0.030
Bottom gravels, cobbles and boulders	0.040
Bottom cobbles with large boulders	0.050

(ii) As given by Choudhury (1993) "Open-channel flow". Prentice-Hal. India Pvt. Ltd. New-Delhi.

Table 6.1(b): Channel roughness values

Type of Channel bottom	Left bank	Right bank	"n_m" value
Slime covered cobble and gravel	Cemented cobbles	Cobble set in gravel	0.024
Sand and clay	Smooth and free vegetation	Smooth and free vegetation	0.030
Smooth Cobble"d_{10}=0.15m"	Smooth Cobble"d_{10}=0.15m"	Smooth Cobble"d_{10}=0.15m"	0.032
Gravel and boulders"d_{10}=1.72 m"	Overhanging bushes	Trees	0.036

Boulders"d_{10}=1.4m"	Gravel, boulders and trees	Gravel, boulders and trees	0.041
Angular Boulders"d_{10}=0.70 m"	Angular Boulders"d_{10}=0.70m"	Angular Boulders"d_{10}=0.70m"	0.050
Boulders"d_{10}=2.10 m"	Boulders"d_{10}=2.10m"	Boulders"d_{10}=2.10m"	0.060
Fine sand	Sand, silt with heavy growth of trees	Sand, silt with heavy growth of trees	0.070
Boulders"d_{10}=2.20 m"	Boulders, bushes, trees	Boulders, bushes, trees	0.075

Based on these tables, it has been found that value ranging from 0.03 to 0.035 can be considered for different reaches of the entire channel under consideration. In floodplain region it varies up to 0.8.

6.2.4 River Channel 1-D Model

Non-prismatic parabolic channel has been taken for the computations. To get the parabolic cross-section at a distance downstream the dam, the available terrain data of original river channel section have been taken and the parabolic least square curve for those data is fitted. The ground elevation for a cross-section of that parabolic channel is taken as the elevation of the centre point of the channel-section. For example, the equation for the parabolic least square curve to fit the available terrain data, to get the simplified river channel section, 52km downstream the dam is $y = 6E - 07x^{-2} - 0.0371x + 462.85$. The bed elevation considered for this section is 125.50 (m). The cross-section considering the computational channel by this approach, at 52km downstream the dam is represented in figure 6.1.

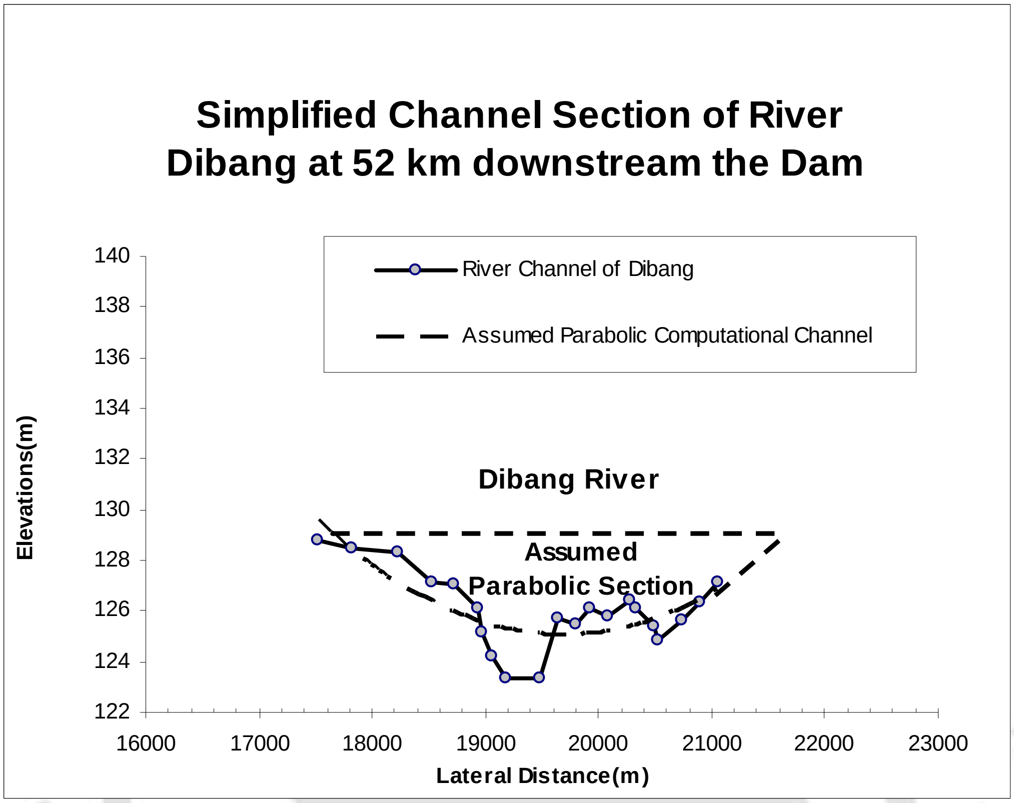

Figure 6.1: Simplified Computational Channel Cross-section of the river Dibang

6.2.5 Simplified Floodplain 1-D model (Computational channel considering the floodplain downstream of the dam)

The non-prismatic channel taken in the computations is a parabolic channel. After analyzing the terrain data for the downstream of the dam, it has been observed that when instantaneous failure of the dam will take place, the flow of the huge quantity of water will not be confined only to the original river channel of Dibang but it will spread out to the nearby land areas and also to the different streams flowing parallel to Dibang. Hence the whole terrain downstream is considered for computational channel. To get the parabolic channel cross section at a point downstream of the dam, the available terrain profile data have been taken and the parabolic least square curve for those data is fitted. The ground elevation for that particular section is taken as the elevation of the centre point of the channel. For example, the channel section at 52 km down stream of the dam has been taken for computations. The equation for the parabolic least square curve to fit available terrain data in the sections is $y = 3E\text{-}08x^{-2} - 0.0014x + 145.61$. The bed elevation considered for this section is 127.000 (m).

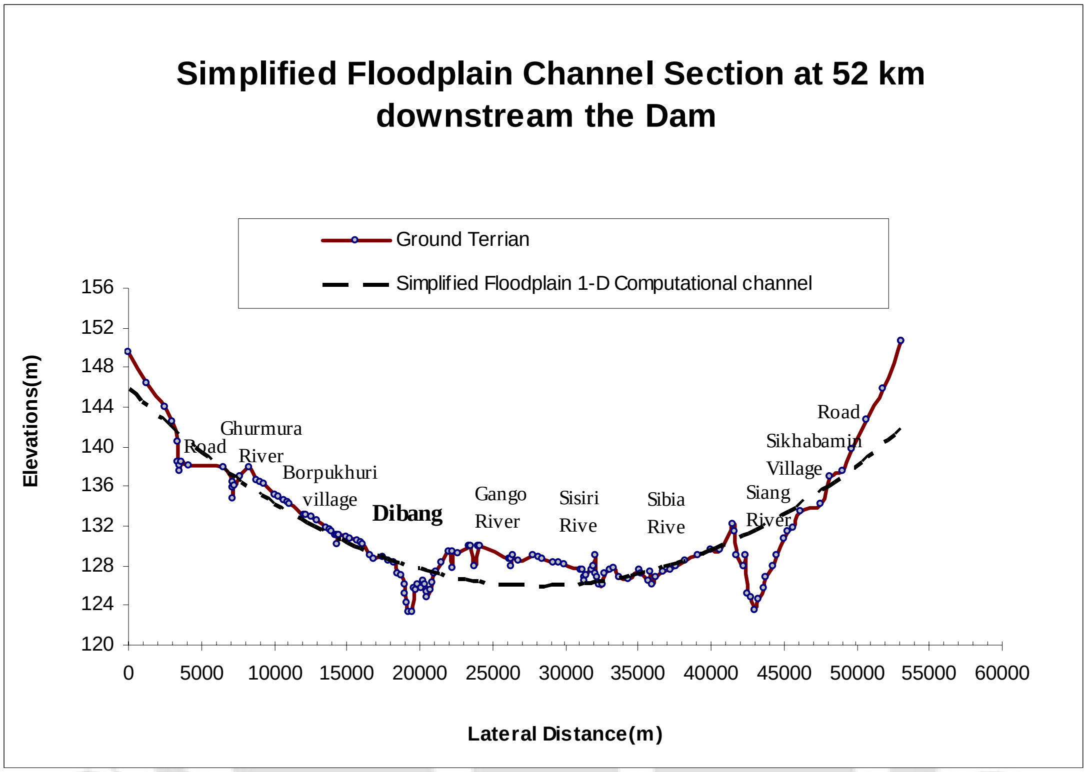

Figure 6.2: Simplified Computational Channel Cross-section in Simplified Floodplain 1-D Model

6.2.5.1 Computational Sections through some Important Places

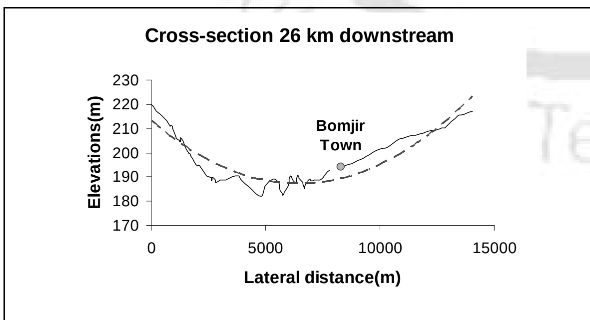

Figure 6.3 Computational cross-section through Bomjir Town

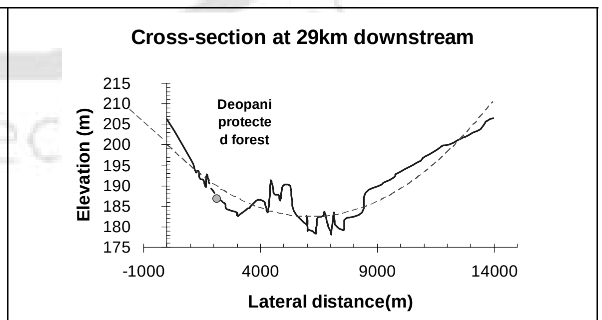

Figure 6.4 Computational cross-section through Deopani Protected Forest

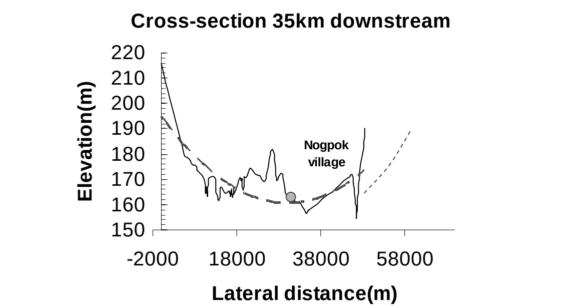	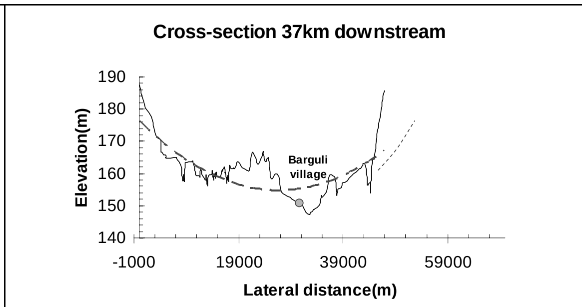
Figure 6.5 Computational cross-section through Nogpok Village	Figure 6.6 Computational cross-section through Barguli village
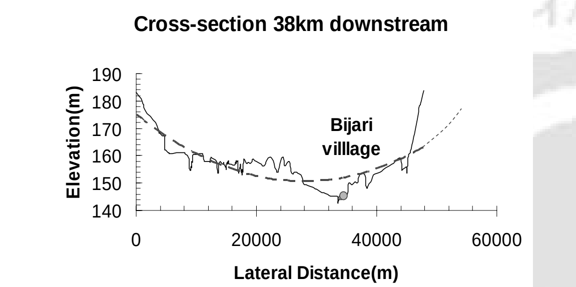	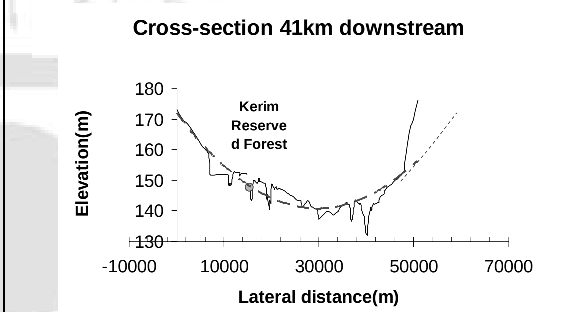
Figure 6.7 Computational cross-section through Bijari Village	Figure 6.8 Computational cross-section through Kerim Reserved Forest
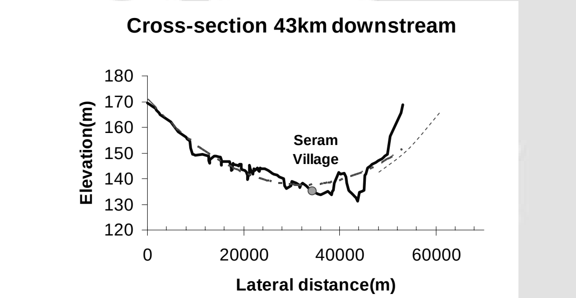	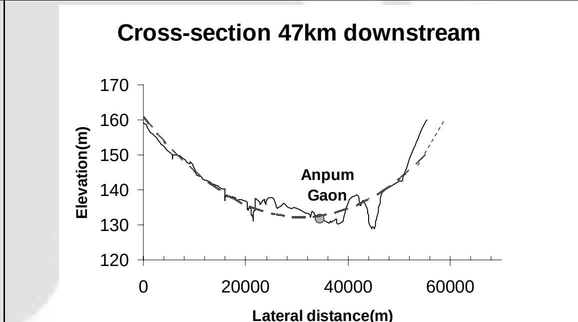
Figure6.9 Computational cross-section through Seram Village	Figure6.10 Computational cross-section through Anpum Gaon

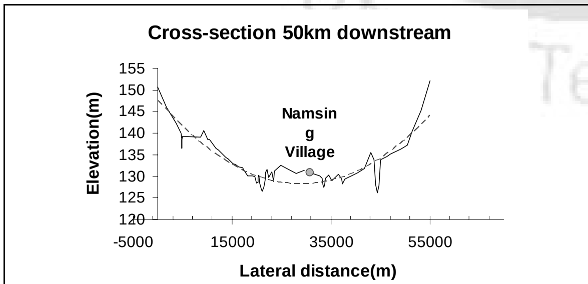	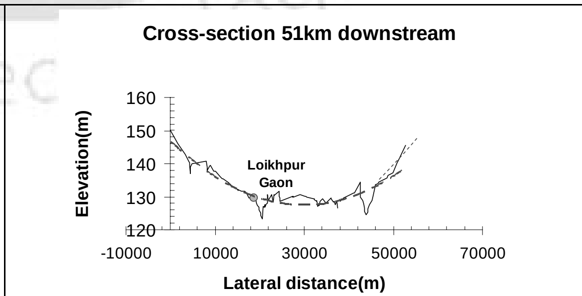
Figure6.11 Computational cross-section through Namsign Village	Figure6.12 Computational cross-section through Loikhpur Village

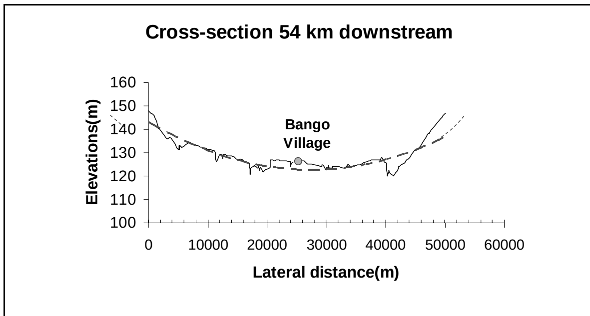

Figure6.13 Computational cross-section through Bango Village

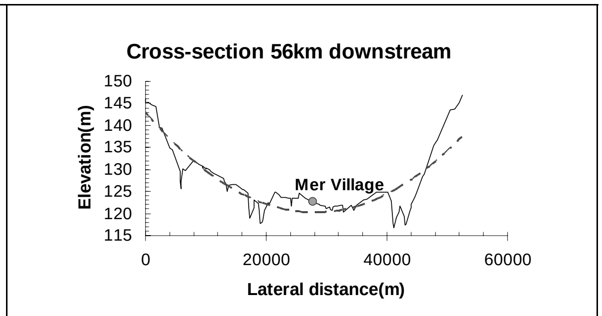

Figure6.14 Computational cross-section through Mer Village

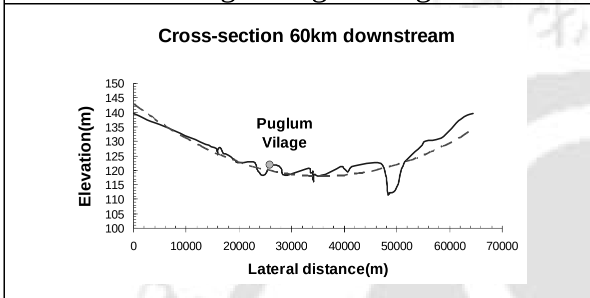

Figure6.15 Computational cross-section through Puglum Village

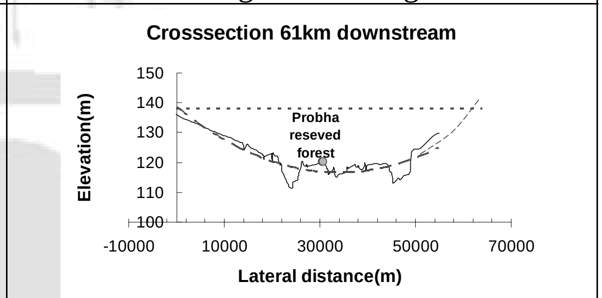

Figure6.16 Computational cross-section through Puglum Village

6.2.6 Composite Channel 1-D Model (Computational channel with compound channel sections)

6.2.6.1 Assumptions

(i) Main Channel and Floodplain Interaction:

Both channel roughness and the depth of the flow, and hence flow conditions are different in the main channel and in the flood plains. In general, floodplain flows are of relatively shallow depth with slow-moving flow adjacent to faster-moving flow in the main channel and hence transfer of momentum between the main-channel and the floodplain is essential. Although different investigators carried out study to shed light on this interaction of the flow conditions, most of their studies are confined to simple straight uniform compound channels in laboratories e.g. (Prooijen et al 2005). They are yet to be confirmed in real field cases where channel may comprise of curvature, roughened floodplains, and steep slopes at the interfaces.

As there is no proper literature available regarding the mechanisms that dominate the momentum exchange in complex natural channels, the total flow in the compound channel is calculated as the sum of the flows in the main channel and in the floodplain assuming no diffusion of lateral momentum in between the river and floodplain as assumed by Lotter (1933), Einstein & Banks (1950) and as in DCM (divided channel method) used in software HEC-RAS (2001).

(ii) Manning "n_m":

As an empirical parameter, the roughness coefficient depends on several factors including surface roughness, unsteadiness characteristics, land use around the section and channel irregularity, and the exact values are often uncertain. The unsteady characteristics i.e. its change with flood depth and time have been studied in details by Nguyen. (2004) and. Fenton (2005). Their study indicates that the main channel roughness values are almost constant while the floodplain roughness values change rather more with flood depth. The values of roughness values in main channel and floodplains of the river change with time. But the variations are found to be rather consistent within reasonable ranges. Their study also found that when roughness coefficients are given as functions of depths and as the number of variable parameters increases, the computation time also increases considerably. In their case study, the computation time was about four times more in comparison with the case where the roughness coefficients are considered constant with depth.Hence the unsteady variations of Manning "n_m" are not considered in this model.

6.2.6.2 Channel Consideration Process

The channel taken for computation has been considered such that once the elevation of flood level is greater than the elevation of the bank of the river channel of Dibang at that section; the floodplain area is gradually introduced there. The real flood plain area with the ground irregularities such as the differences in elevation and in roughness considering the type of land use in the lateral direction of the flow has been considered. It is appropriate to refer Weisbach's friction factor for point resistance while Manning's roughness for cross sectional and reach resistance (Yen 2002). The downstream of the proposed Dibang dam after 32km comprises of multiple stream channels, villages, roads, dense forests and corresponding Manning's roughness has been considered from Chow's (1956) table. Seventeen numbers of formulae have been listed for compound composite channels based on different assumptions about the relationships of the discharges, velocities, forces, or shear stresses between the component subsections and total cross section and different methods have been suggested to divide the cross section into subsections for applications of these formulae (Yen 2002). Motayed and Krishanmurthy (1980) compared four of the 17 equations by using the data available for 36 natural cross sections by U.S. Geological Survey. They observed that the equation by Horton (1933) and Einstein (1934) gives least error. Hence it is used here for computation of equivalent Manning's roughness. In computation of Equivalent Manning's "n_e" by Horton (1933) and Einstein (1934), it is assumed that mean flow velocity in each of sub areas is equal to the mean flow velocity. The equivalent Manning's "n_e" for a composite channel section subdivided into N sub areas having wetted perimeter P_i and Manning constant n_i (i =1,2,..,N) is given as : $n_e = \left(\frac{\sum P_i n_i^{\frac{3}{2}}}{\sum P_i} \right)^{\frac{2}{3}}$

6.3. 2D Numerical Model:

In this model, the flow domain for computations is taken as 106 km x 63 km. The length is (43km+63 km=) 106km along the longitudinal direction and in the lateral direction it is 63 km.

6.3.1 Initial and Boundary Conditions

The upstream boundary coincides with the upstream extremity of the reservoir at 43 km and the downstream one with its confluence with River Brahmaputra, at 63 km from the dam site. At upstream of the dam the initial profile has been considered as the surface of the still water stored in the reservoir at maximum storage level. For computational advantage, a water depth of 0·1m is assumed in the dry downstream portion. To start the numerical computation an initial profile computed by Ritter's equation after elapse of 1sec since failure of the dam has been introduced. Boundary values are obtained with the help of method of characteristics. The reflection boundaries are assumed in lateral direction.

6.3.2 Channel Roughness

Standard values of Manning's roughness coefficient "n" for natural valley has been considered as presented in section6.2.3 given by Chow (1956) and Choudhary. (1993).

6.3.3 Input data

(i) Fixed grid has been incorporated with small spatial interval

(ii) Total 1000 spatial interval in longitudinal direction

(iii) Total 600 spatial interval in lateral direction

(iv) dx=106 m and dy= 106m

(v) Total 6,01,601 grid points

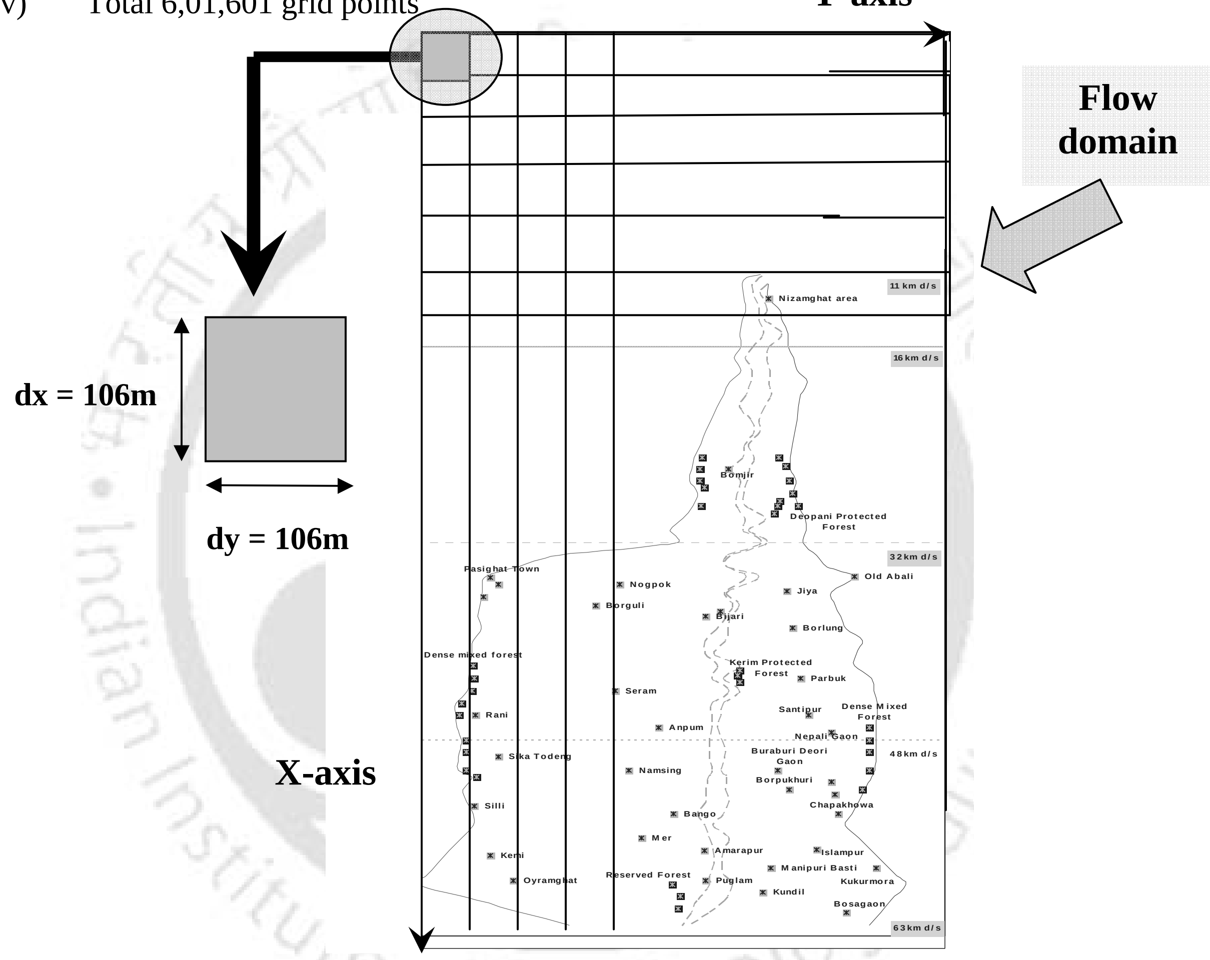

Figure 6.17: Flow Domain in 2D Model

6.4. Comparisons and Analysis of Computed Flood Parameters

6.4.1 Outputs of River Channel 1-D model and the Simplified Floodplain 1-D model

The maximum predicted depths, their time of occurrence and maximum predicted velocities obtained from both the simulation models have been plotted in figure 6.18, figure 6.19 and 6.20 respectively. It is intended to predict the flood at any downstream cross-section. For example, when the simplified river channel approach is considered the water level at 52km downstream is shown in Figure 6.21. The maximum predicted water level is 138m, which is 10m higher than the top elevation of the bank of the river Dibang. Hence it is obvious that flow will follow its' own path where the channel will comprise of the original channel of Dibang as well as the nearby area to some lateral distance at that section depending on terrain elevation. On the other hand if the predicted maximum depth by the simplified river channel for a particular section downstream of the dam is considered for that entire lateral terrain in that channel cross-section then the inundated area will also be over predicted. The prediction of maximum water level and the lateral inundation, at 52km down stream of the dam by simplified floodplain channel is presented in figure 6.22.

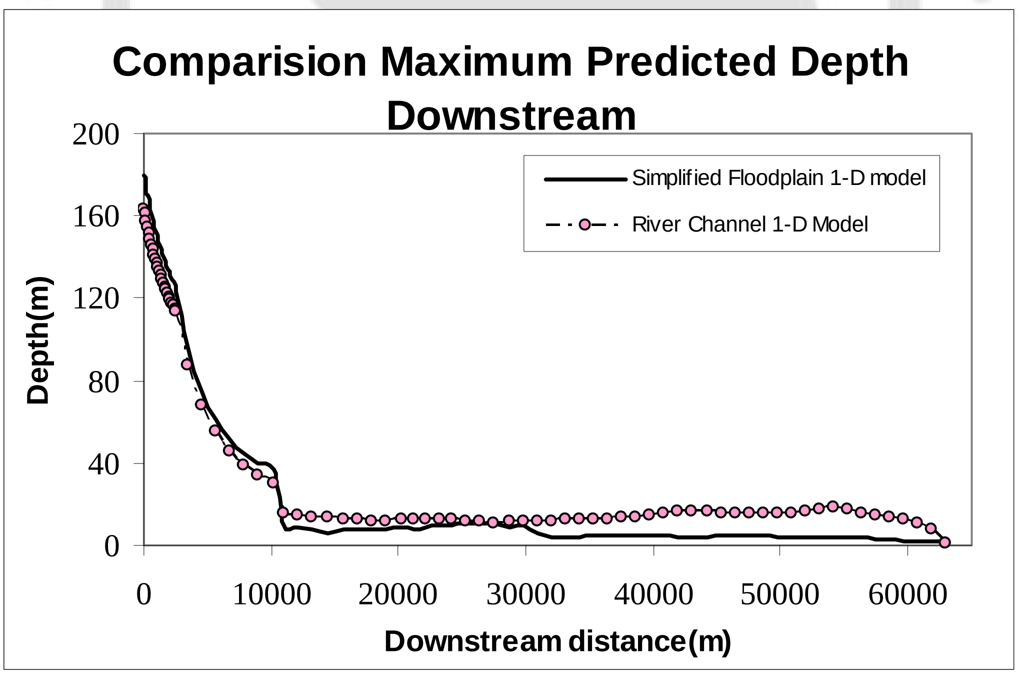

Figure 6.18: Comparison of Maximum Predicted Flood Depths -I

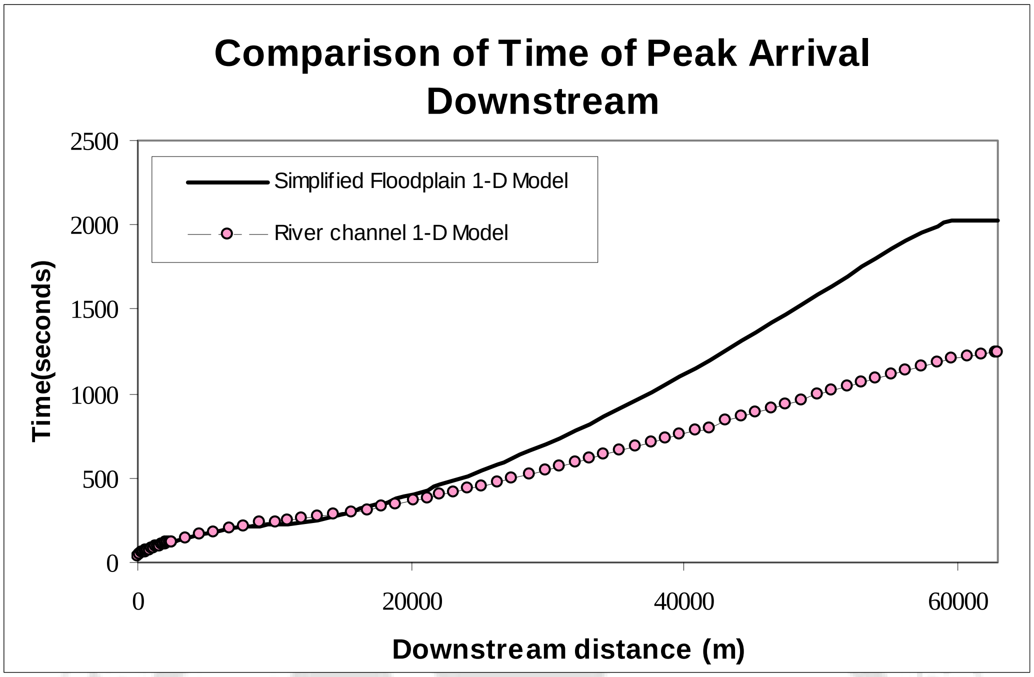

Figure 6.19: Comparisons of Time of Peak Arrivals-I

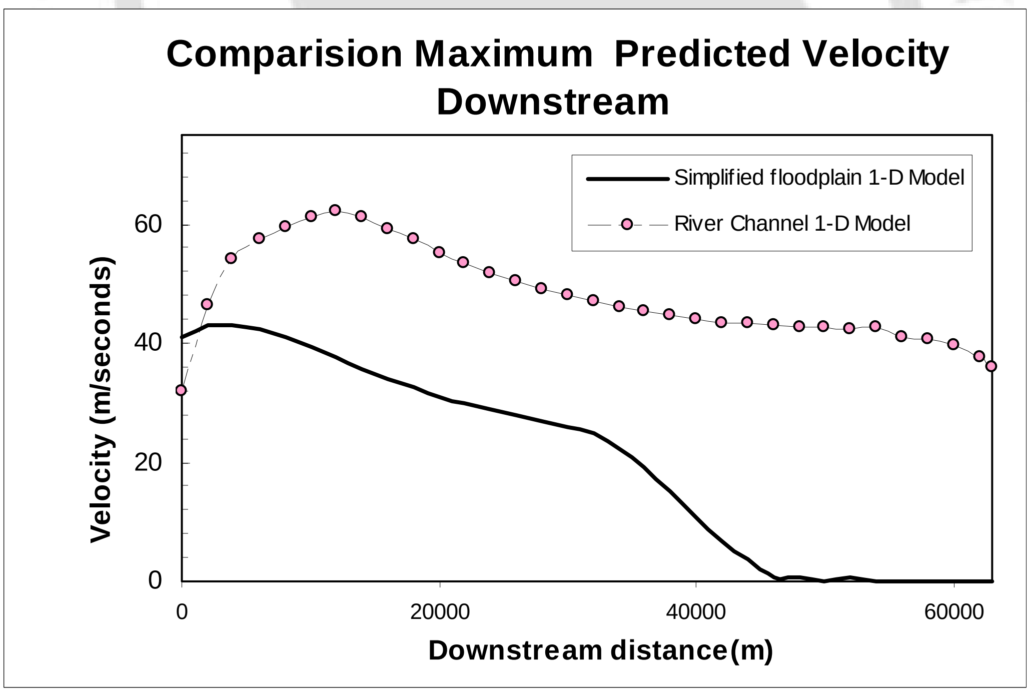

Figure 6.20: Comparisons of Maximum Predicted Velocity-I

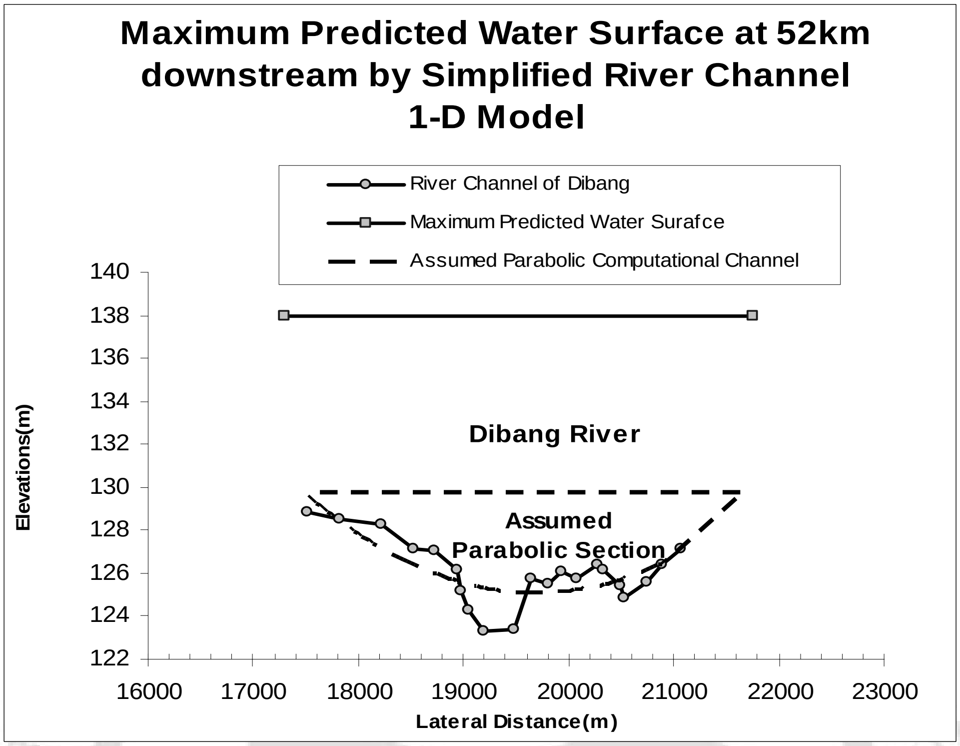

Figure 6.21: Cross-section at 52 km downstream the Dam - Simplified River Channel 1-D model

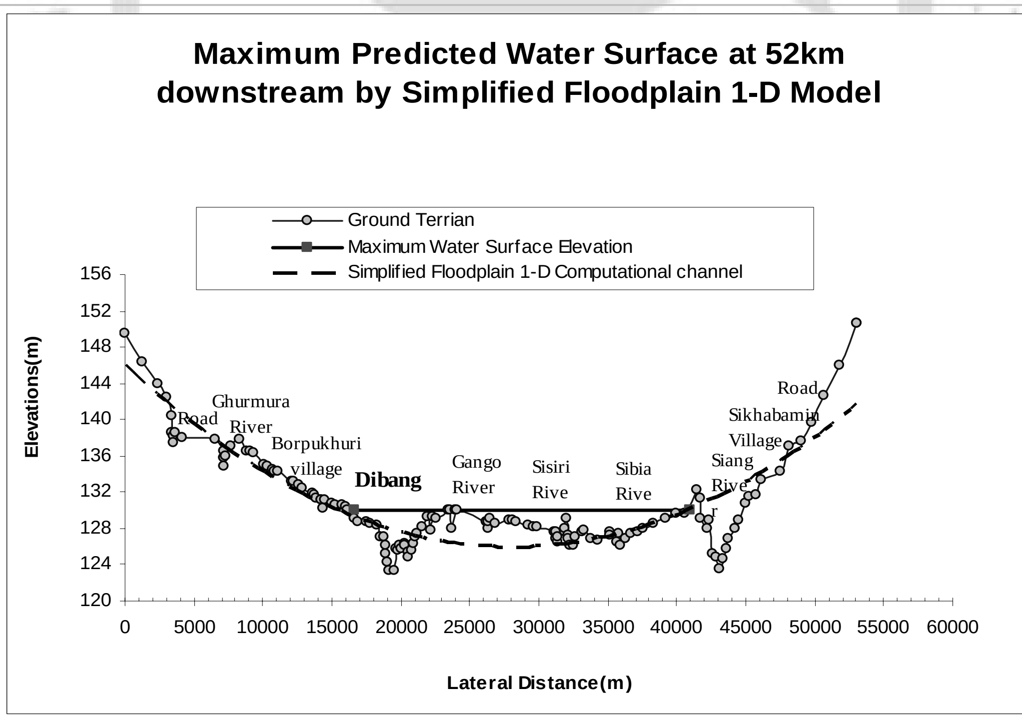

Figure 6.22: Cross-section at 52 km downstream the Dam - Simplified Floodplain 1-D model

Comparison of the maximum predicted flood depths shows that the computed depth is significantly reduced as the river Dibang enters the plain. As observed from Figure 6.18, reduction starts from 11km downstream of the dam. Significant high predictions have been observed from 32 km downstream, when the flood is simulated in the channel, where the simplified channel of river Dibang is only taken into account. For example, depths are over predicted as, 220.125%, 306.818 % and 248.185 % at 32km, 52km and 62 km downstream of the dam respectively by this approach compared to the computed maximum depths by simplified 1-D model.

While analyzing the time of peak arrival, it has been observed that flood wave arrives earlier in river channel model. As soon as the river Dibang enters the plain the time of peak arrival starts increasing remarkably, when the floodplain is considered in computational channel compared to the simplified river channel. For example, the peak arrival time increases 23.701% at 32 km downstream and the increment rises to as high as 63.802 % in 62 km downstream.

The comparison of the maximum predicted velocity shows that the predicted value is highly overestimated in the simplified river channel. When the river enters in plain region, at 12 km downstream the velocity is predicted as 65.616% higher when the computational channel is considered as the simplified river channel of Dibang. Another observation made from the plot of maximum velocities shows the significant difference in the prediction of the highest maximum value for velocity at any section cross-section downstream. The simulation results with simplified river channel predict the maximum possible velocity downstream of the dam as 62.342 m/second at 12000.00m downstream whereas on the other hand the simulation of the flood in the simplified floodplain 1-D model predicts it as 42.376 m/s at 6035.26m downstream .The over prediction of the maximum velocity increases continuously towards the downstream after 32km downstream the dam.

6.4.2 Outputs of the Simplified Floodplain 1-D model and the Composite Channel Model

The maximum flood depths, maximum velocities, and times of arrival of peak depth predicted by of both the models are shown in the following figures 6.23, 6.24 and 6.25 respectively.

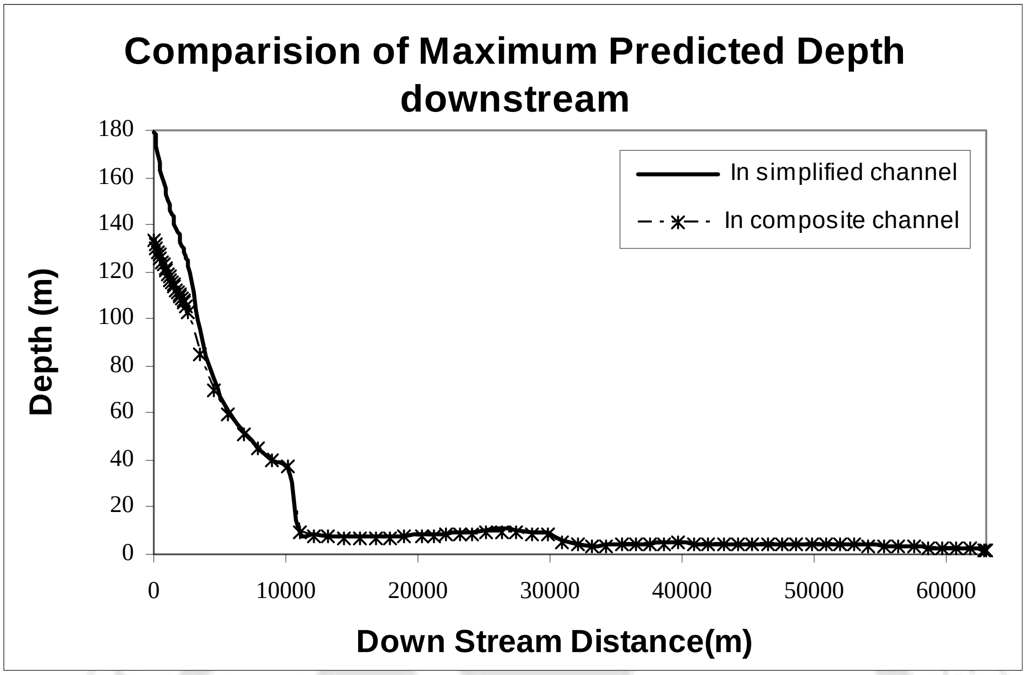

Figure 6.23: Comparison of Maximum Predicted Flood Depths -II

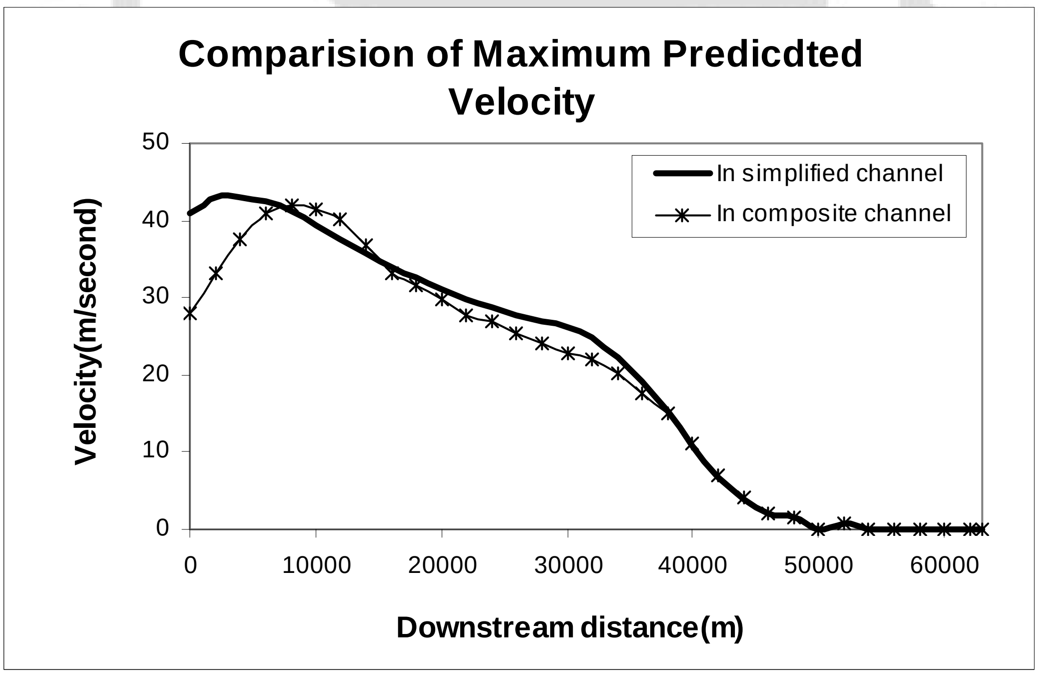

Figure 6.24: Comparisons of Maximum Predicted Velocity-II

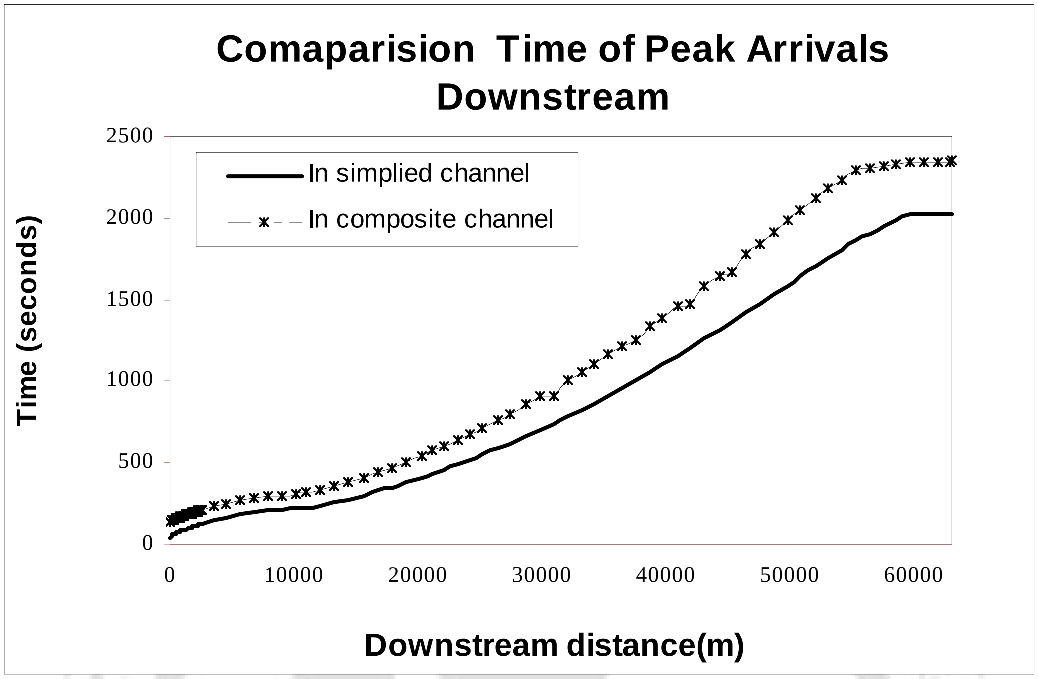

Figure 6.25: Comparisons of Time of Peak Arrivals-II

The comparisons of the maximum predicted values by the simplified model in terms of difference in percentage with respect to the values computed considering the composite channel are presented in the figures 6.26, 6.27 and 6.28.

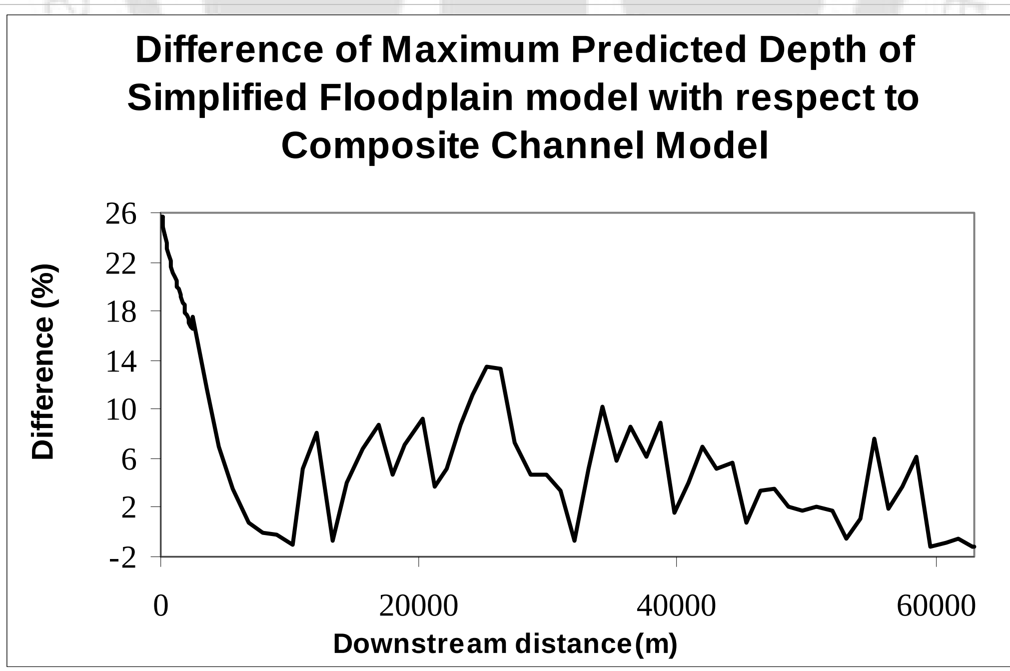

Figure 6.26: Comparison of Maximum Predicted Flood Depths –III

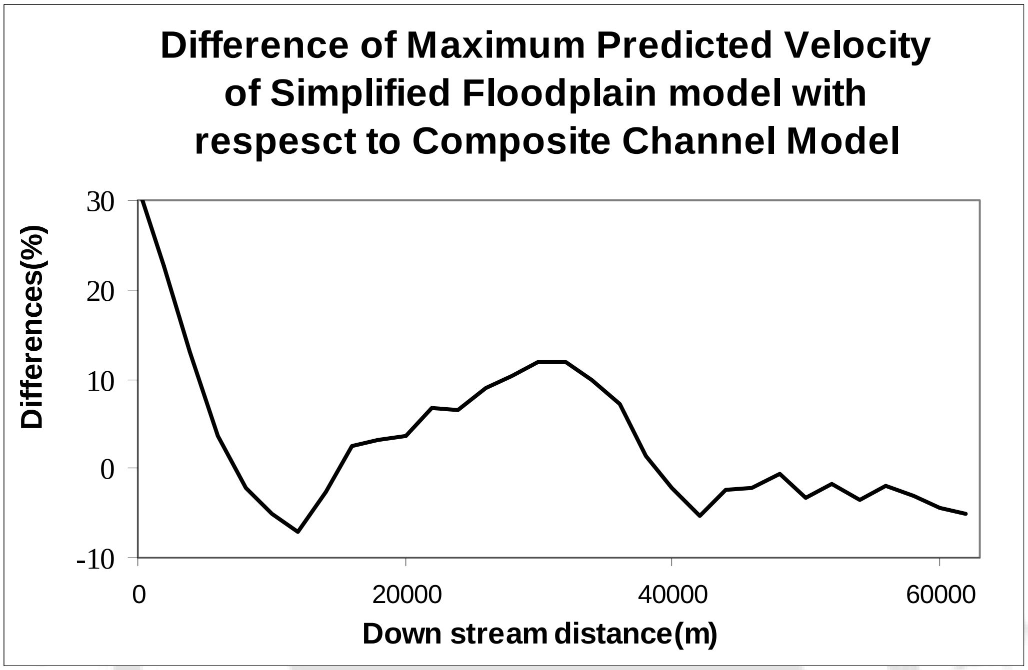

Figure 6.27: Comparisons of Maximum Predicted Velocity III

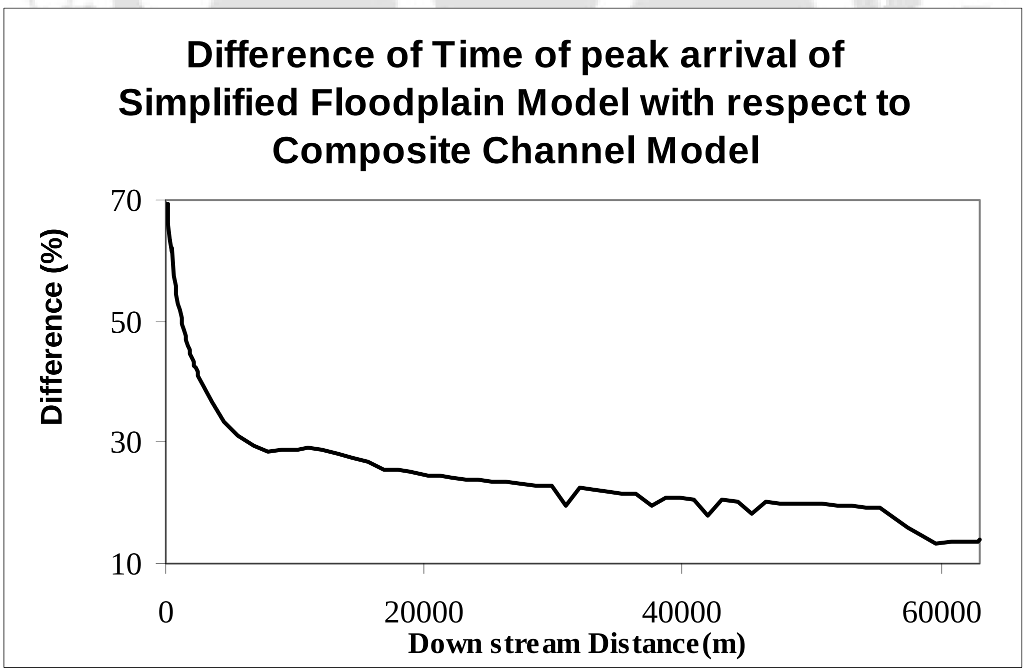

Figure 6.28: Comparisons of Time of Peak Arrivals-III

The maximum depths are estimated slightly more in case of the simplified floodplain model. The times of peak arrivals are more in case of the composite channel as equivalent Manning's "n" is taken as surface resistance considering the details of downstream land use. It is interesting to note that the estimations of the above two parameters given by the model with simple parabolic section are in safer side from flood prediction, early warning and managerial point of view. The velocity in the composite channel is reduced due to the high surface resistance of the flood plain. The reductions of depth at further downstream of the flow in case of composite channel remain confined within the main channel and thus the flood plain resistance does not influence the flow computation. Therefore, the maximum velocities in the simple channel sections are found to be less after 40 km downstream than that in the composite sections. This in turn increases the flow velocity of the composite channel marginally as compared to that of the simple parabolic section. However, the maximum variation of velocity beyond 40km is only 5.22%, which may be considered not that significant from practical point of view.

6.4.3 Outputs of the Simplified Floodplain 1-D model and the 2- D model

The maximum water surface elevations and the times of peak arrival at important points downstream of the dam are compared with the values obtained by both the 1-D and 2-D numerical models and they are presented in the table 6.1 and table 6.2 respectively. The important places downstream of the dam have been shown in Figure 6.29.

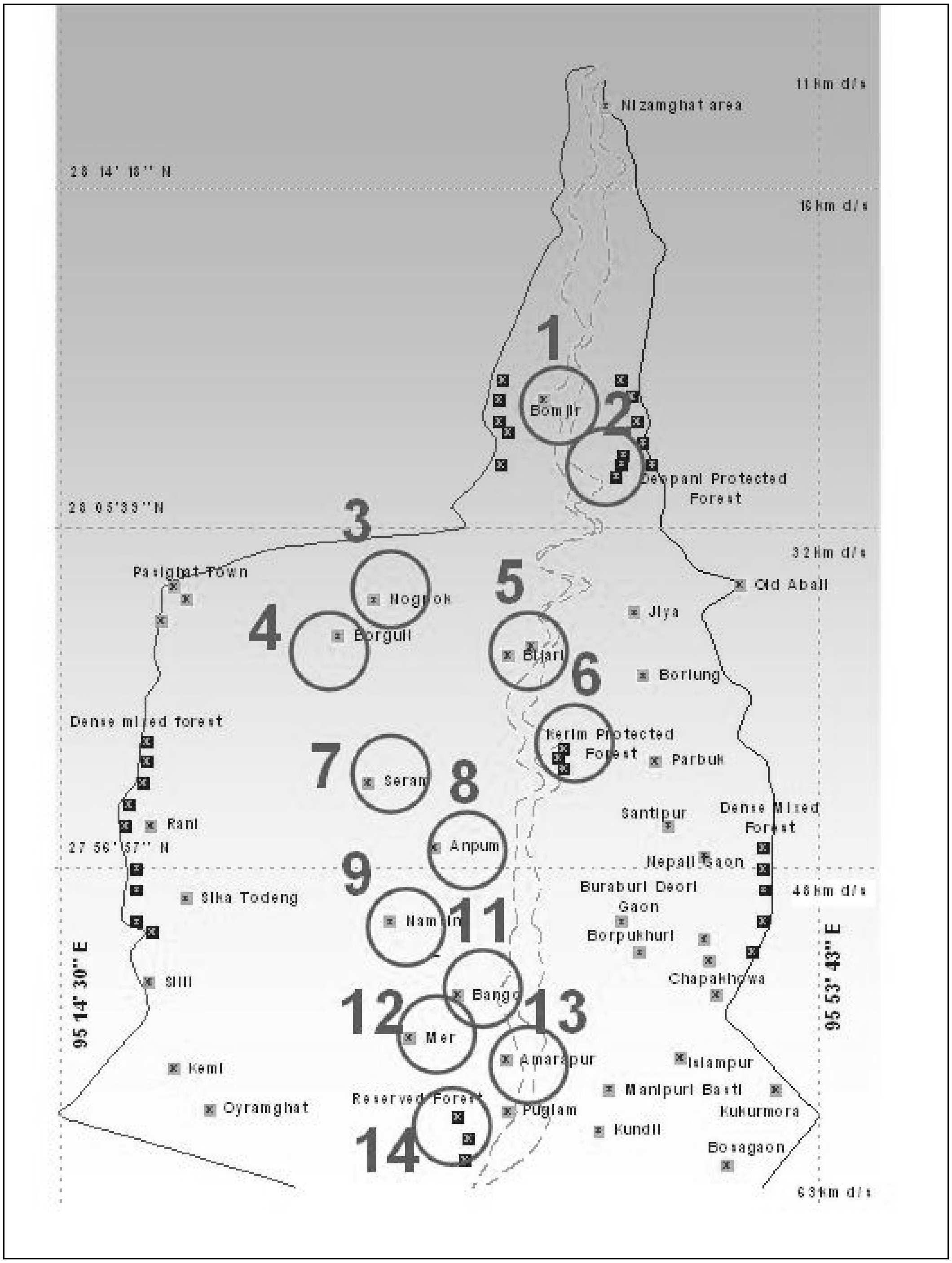

Figure 6.29 Some Important Places downstream of the Dam

Table 6.2: Comparisons of Maximum Water Depths at Important Places Downstream the Dam between 1-D and 2-D Numerical Models.

	Places	Downstream distance (km)	Computational Nodes	Water surface elevations by 1-D Model (m)	Water surface elevations by 2-D Model (m)
1	**Bomjir**	26	616,382	208.266	206.5
2	**Deopani potected forest**	26	656,446	201.34	166.76
3	**Nogpok**	35	716,248	180.252	176.3
4	**Borguli**	37	736,217	173.502	172.51
5	**Bijari**	38	746,368	171.432	171.03
6	**Kerim r forest**	41	776,367	151.065	148.86
7	**Seram**	43	766,242	147.563	145.06
8	**Anpum**	47	826,243	140.473	141
6	**Namsing**	50	866,276	132.875	133
10	**Loikhpur gaon**	51	876,366	130.555	128.81
11	**Bango**	54	606,266	127.01	124.1
12	**Mer**	56	626,283	123.866	122.62
13	**Amapur**	57	636,320	122.368	121.45
14	**Puglam**	60	666,356	121.4607	120.2
15	**Probha r.forest**	61	676,300	116.261241	115.06

Table 6.3: Comparisons of Times of Peak Arrivals at Important Places Downstream the Dam between 1-D and 2-D Numerical Models.

	Places	Downstream distance (km)	Computational Nodes	Time of arrival of peak depth by 1D Model (Seconds)	Time of arrival of peak depth by 2D Model (Seconds)
1	**Bomjir**	26	616,382	543.801	760.101
2	**Deopani protected forest**	26	656,446	677.301	664.5
3	**Nogpok**	35	716,248	610.701	1165.22
4	**Borguli**	37	736,217	1005.801	1256.3
5	**Bijari**	38	746,368	1052.41	1341.05
6	**Kerim Reserved Forest**	41	776,367	1152.601	1735
7	**Seram**	43	766,242	1257.401	1775
8	**Anpum**	47	826,243	1406.301	1666
6	**Namsing**	50	866,276	1602.8	2234.2
10	**Loikhpur Gaon**	51	876,366	1661.601	2275
11	**Bango**	54	606,266	1768.501	2467.4
12	**Mer**	56	626,283	1860.2	2583.55
13	**Amapur**	57	636,320	1634.2	2564.1
14	**Puglam**	60	666,356	2012.788	2630
15	**Probha Reserved Forest**	61	676,300	2022.801	2630

The comparisons presented in the above tables are shown graphically in the figure 6.30 and figure 6.31 respectively.

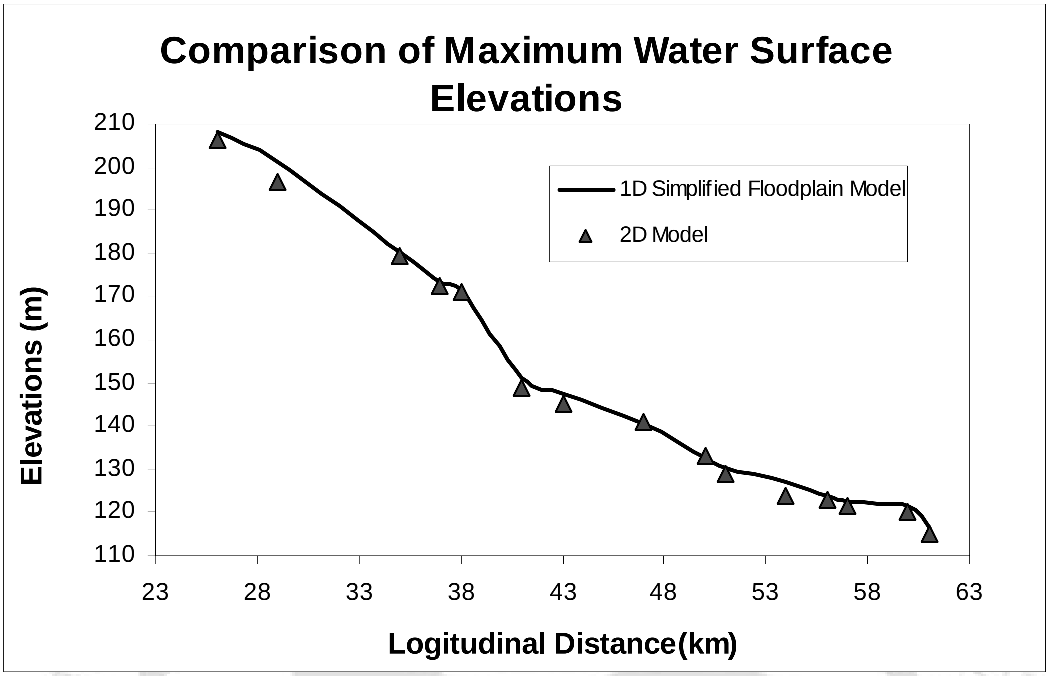

Figure 6.30: Comparison of Maximum Water Surface Elevations

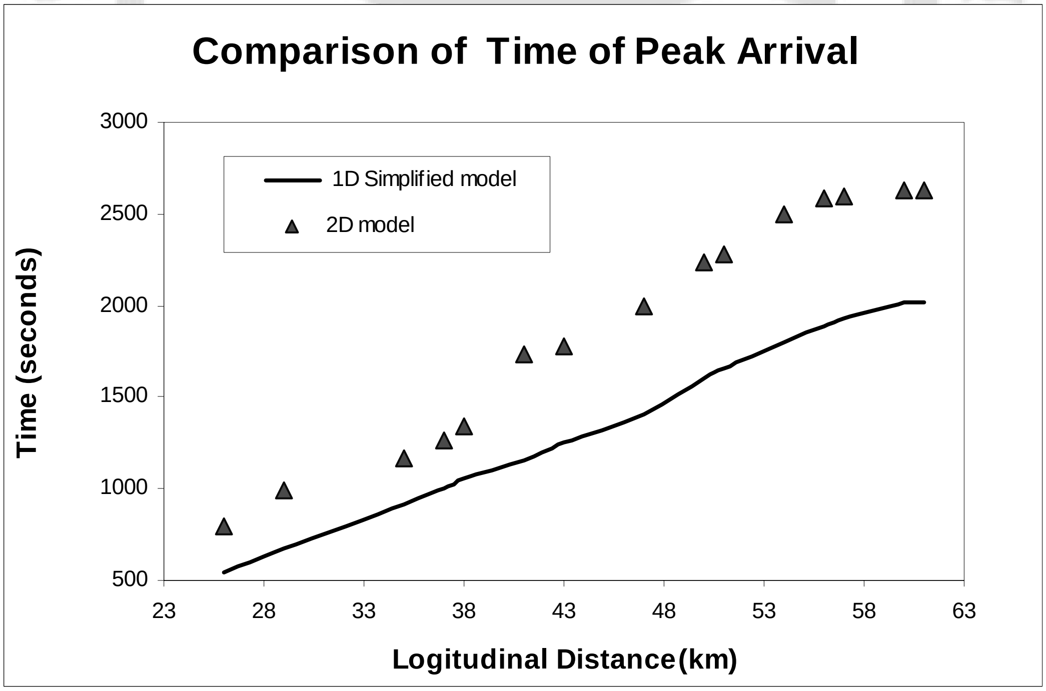

Figure 6.31: Comparison of Time Peak Arrival

Comparisons of the flood depth hydrographs at some important points downstream are presented in figures 6.32, 6.33, 6.34, 6.35, 6.36 and 6.37. Water surface elevations and inundations downstream at a particular instant of time i.e. at 600 seconds after dam failure by both simplified floodplain 1-D model and 2-D model are presented in figures 6.38 and 6.39 respectively. The wave arrival in the 2- D model is slow. The maximum predicted flood depths in most of the downstream places are estimated higher except in the computational point at Anpum village.

The ultimate inundations downstream due to the failure of Dibang dam by both simplified floodplain 1-D model and 2-D model are presented in figure 6.40.

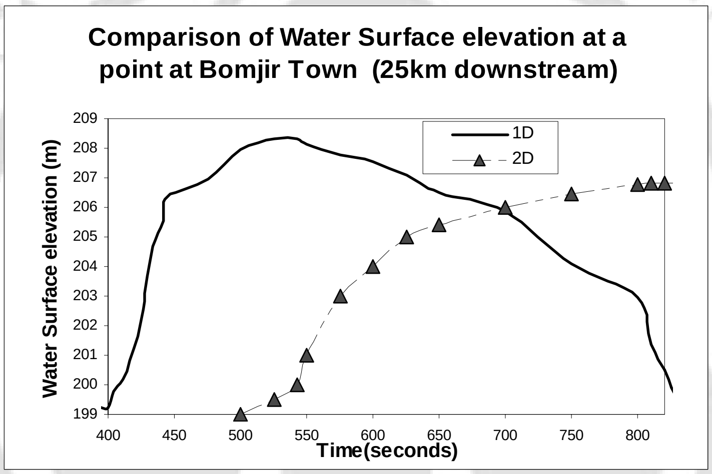

Figure 6. 32: Comparison of Water Surface Elevations by 1-D and 2-D Models at various Times at Bomjir town (node 616,382)

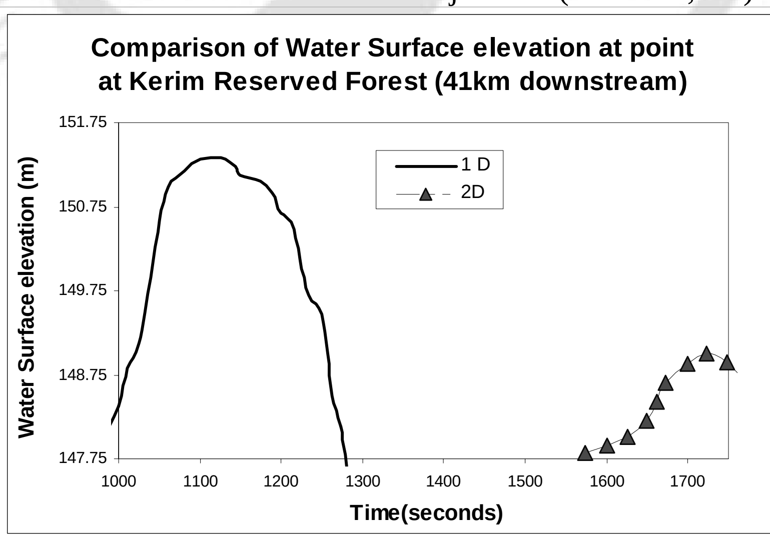

Figure 6. 33: Comparison of Water Surface Elevations by 1-D and 2-D Models at various Times at Kerim Reserved Forest (node 776,367)

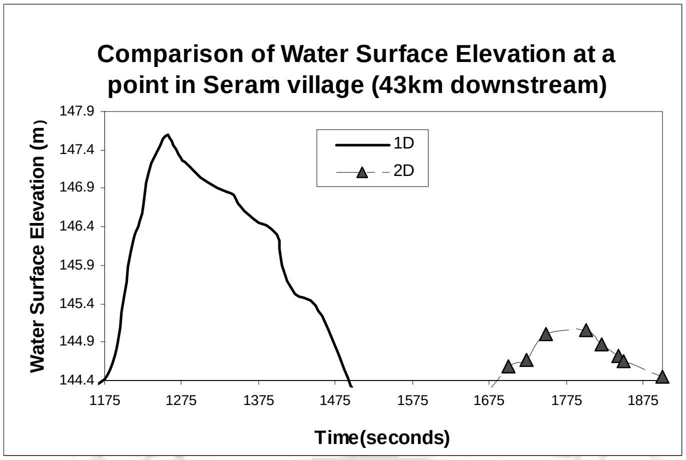

Figure 6. 34: Comparison of Water Surface Elevations by 1-D and 2-D Models at various Times at Seram Village (node 766,242)

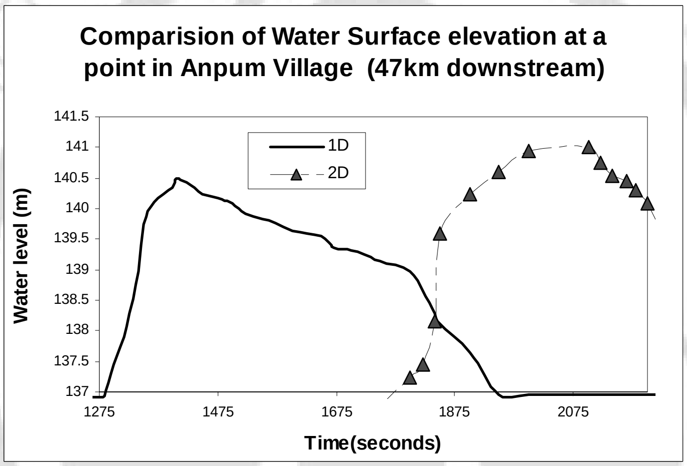

Figure 6. 35: Comparison of Water Surface Elevations by 1-D and 2-D Models at various Times at Anpum Village (node 826,243)

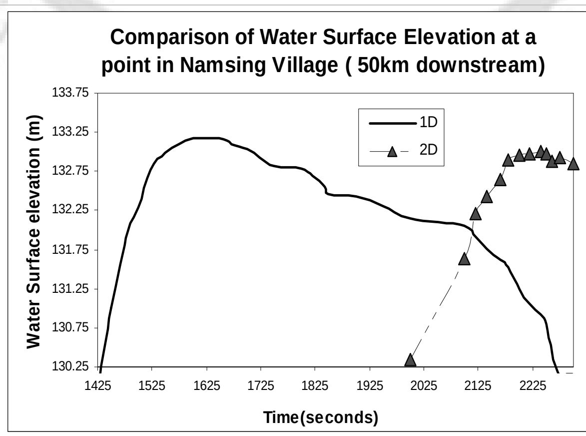

Figure 6. 36: Comparison of Water Surface Elevations by 1-D and 2-D Models at various Times at Namsign Village (node 866,276)

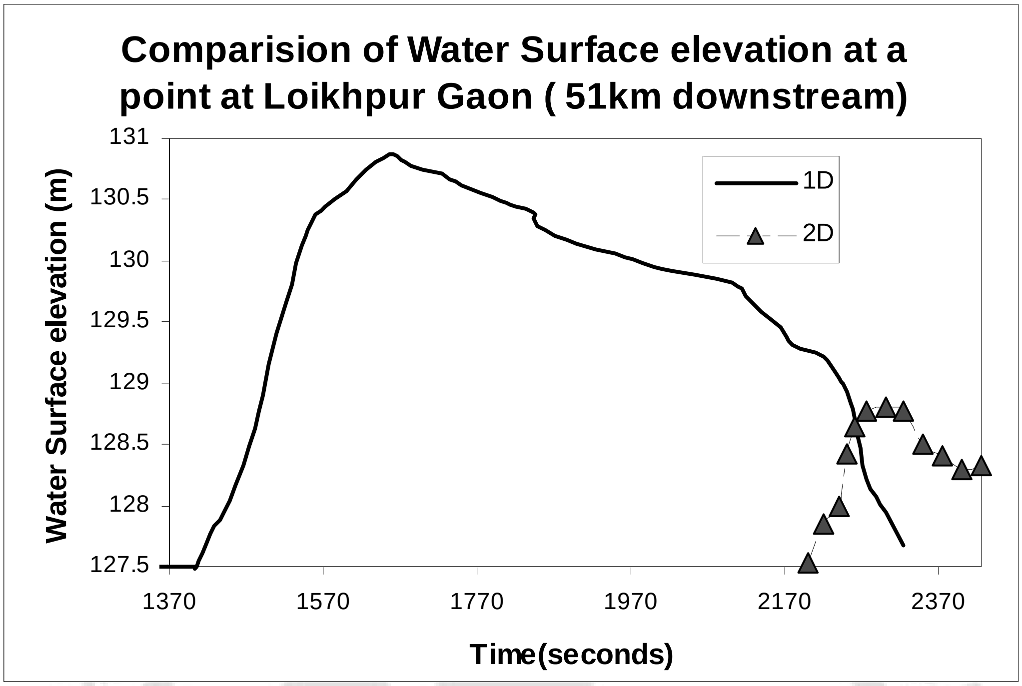

Figure 6. 37: Comparison of Water Surface Elevations by 1-D and 2-D Models at various Times at Loikhpur Village (node 876,366)

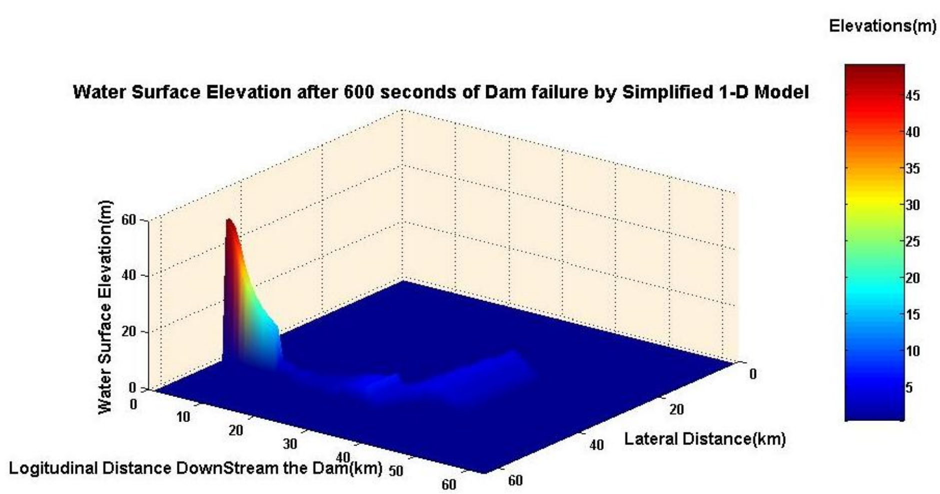

Figure 6. 38: Water Surface Elevations and Inundations by Simplified Floodplain 1-D model after 600 seconds of Dam Failure

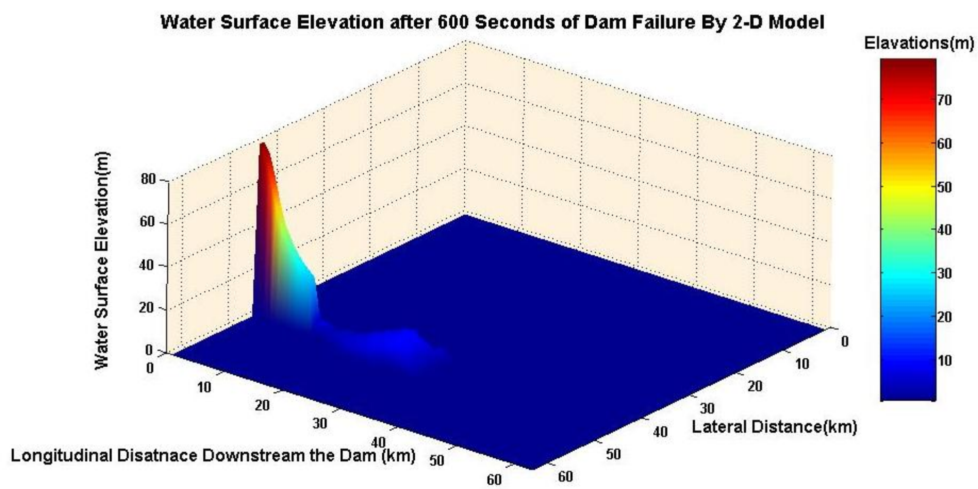

Figure 6.39: Water Surface Elevations and Inundations by 2-D model after 600 seconds of Dam Failure

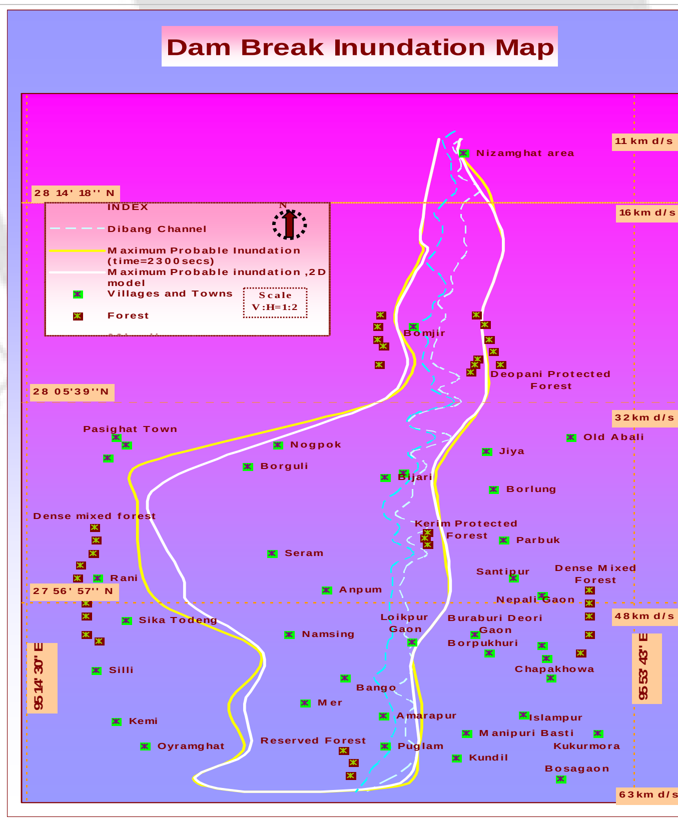

Figure 6.40: Comparison of Inundation with 1D and 2D model

6.4.4 Comparison of the Computed Volume of Water Released and Volume of Water Arrived Downstream at Different Time Intervals

. To analyze the accuracy of the predictions of the simplified floodplain 1-D model, volume of water released from the reservoir and the volume of water arrived downstream at various instant of time are compared .The comparison is presented in figure 6.41. It illustrates quite similar values for volume of water released and volume of water accumulated downstream; which in turn indicates better prediction of the flood parameters by the developed simulation model. .

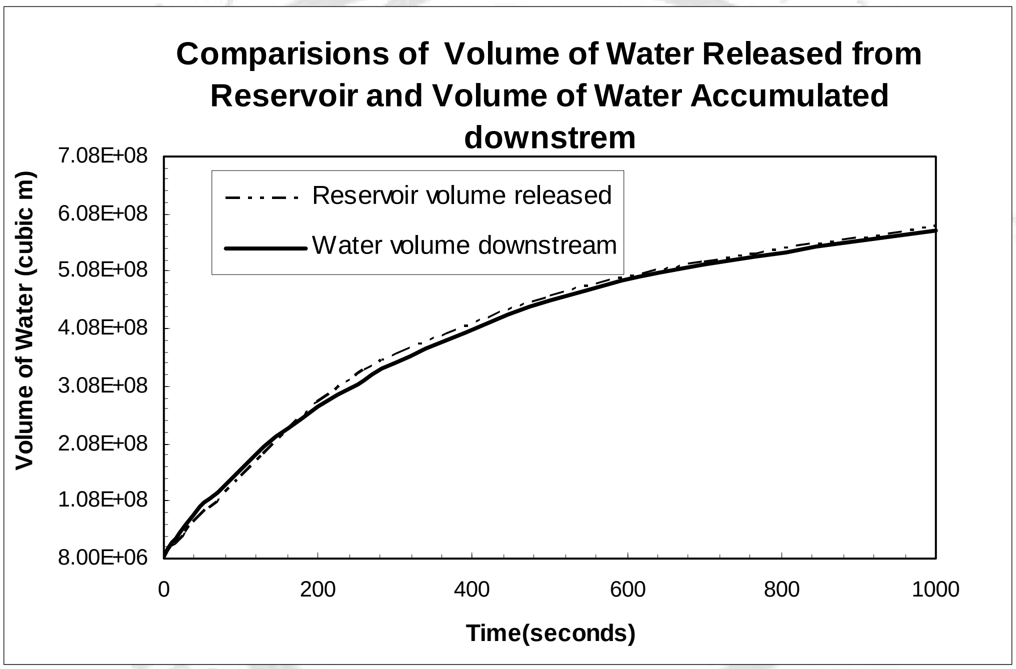

Figure 6. 41: Comparison of Volume of Water Released from the Reservoir and Volume of Water Arrived downstream by Simplified Floodplain 1-D model

6.5. Conclusions

6.5.1. Observations from the Analysis and Comparisons of River Channel 1-D Model and the Simplified Floodplain 1-D Model

It has been observed that, the simplified river channel 1-D model over predicts the maximum flood depth, velocity and under predicts the time of peak arrival at the downstream sections when the river enters the plain. The overestimation is quite significant after a distance 32km downstream the dam, with a maximum at 52 km downstream, which is 306.818% higher compared to the computed maximum depth by simplified floodplain 1-D model. The time of peak arrival starts increasing remarkably after a distance 14400m downstream, when the floodplain is considered in computations compared to the simplified river channel. The comparison of the maximum predicted velocity shows that the predicted value is highly overestimated in the simplified river channel for the entire downstream sections. In the simplified river channel the maximum possible velocity downstream of the dam is predicted as 62.342 m/s at 12000.00m downstream where the maximum predicted velocity is 42.376 m/s at 6035.26m downstream when the flood is simulated on the channel, which comprises of the floodplain.

6.5.2. Observations from the Analysis and Comparisons of Simplified Floodplain 1-D Model and the Composite Channel 1-D Model

The prediction and the analysis of the flood computed by the simplified floodplain model have been compared with the model developed considering the detailed downstream terrain as a composite section. The estimations of flood parameters are quite comparable with slight high estimation in case of simplified floodplain model, which is on the safer side so far flood prediction; early warning and mitigation plans are concerned. Therefore, the proposed simplified floodplain model is suitable to use for prediction of dam break flood in such complex floodplain topography. Moreover, the model is quite easy to implement as compared to the composite channel sections, which requires significant effort in preparing database of the terrain.

6.5.3. Observations from the Analysis and Comparisons of Simplified Floodplain 1-D Model and the 2-D Model

While comparing the computations with 2-D model with the 1-D model following conclusions can be made:

(i) Additional work is required to undertake 2-D modeling. i.e. the data preparations for the input. For example in case of Dibang project :

- 1-D model has 1001 computational points with 1000 intervals of 106 m each
- 2-D model has total 6,01,60 computational points with 1000 x 600 intervals of 106m each

(ii) The flood levels in the 1-D model are predicted higher than that in 2-D

(iii) 1-D model predicts wave speed faster than that for 2-D model.

For the present situation, it has been observed that, even with 1-D model, relatively satisfactory performance could be achieved. Therefore, in a situation where detail data are not available, one dimensional model serves the purpose for engineering design.

Chapter-7

Simulation of Dam Break Flow in Mobile Bed Channels

7.1. Introduction

Over the recent decades, there have been continuing efforts to understand dam-break hydraulics and the majority of the studies have been carried out in laboratories where channel bed is assumed to be fixed under such a highly transient flow. Dam-break hydraulics of natural rivers is indeed complicated, and involves not only water flow, but also morphological change in the riverbed, which in turn modifies the predicted flow profiles. Although recently very few numerical models are proposed for mobile beds also, applications were only in simple laboratory channels. In this chapter investigations have been made to test the following:

(i) The change in the original river channel bed profile under the highly transient flow of Dam break.

(ii) Effect of Change of the channel bed profile on free water surface profiles and hydrographs.

(iii) Again as the change in the topography is not confined only to a rise (or a lowering) of the riverbed but consequences a completely different cross section. Hence another important point is included here to get the ultimate cross-sections of the river channel after the dam break flood.

(iv) The role of the form of the governing equations for developing suitable numerical simulation models for real world complex conditions.

7.2. Mathematical Formulations

Two steps are proposed here to study dam break hydraulics in real field complex situations:

(i) **Step 1**. Computation of hydrodynamics and erosion or deposition of the river bed, by mass and momentum conservation for the water-sediment mixture and the mass conservation for sediments.

(ii) **Step 2**. To find out the ultimate shape of river channel cross-sections after dam break flood by distribution of eroded mass or deposited mass inside every cross section.

The details are as follows:

7.2.1. Governing Equations

The mass and momentum conservation equations for the water-sediment mixture and the mass conservation equation for sediment are the governing equations. Different investigators tried different solutions of dam break flow on movable bed, some coupled together the three equations and some tried it by solving the flow and sediment equations separately termed as decoupled solutions.

The mass and momentum conservation equations for the water-sediment mixture and the mass conservation equation for sediment in a non-prismatic channel can be expressed as:

$$\frac{\partial U}{\partial t} + \frac{\partial F}{\partial x} = S \tag{7.1}$$

Where

$$U = \begin{Bmatrix} A \\ AV \\ Ac \end{Bmatrix} \tag{7.2}$$

$$F = \begin{Bmatrix} AV \\ AV^2 + \frac{1}{2} g \frac{A^2}{b_m} \\ AVc \end{Bmatrix} \tag{7.3}$$

$$S = \begin{Bmatrix} (E-D)/(1-p) \\ -gA(\frac{\partial z}{\partial x} + S_f) - \frac{(\rho_s - \rho_f) g A^2}{2 \rho b_m} \frac{\partial c}{\partial x} - \frac{(\rho_o - \rho)(E-D)V}{\rho (1-p)} \\ E-D \end{Bmatrix} \tag{7.4}$$

t is the time; x is the direction parallel to the river; A is the cross-sectional flow area, V is the depth-averaged velocity; z is the bed elevation; c is the flux-averaged volumetric sediment concentration; b_m is the mean cross sectional width; g is the acceleration due to gravity; S_f is the friction slope; p is the bed sediment porosity; E, D is the sediment entrainment and deposition fluxes across the bottom boundary of flow, representing the sediment exchange between the water column and bed, $\rho = \rho_w(1\text{-}c) + \rho_s c$ is the density of water-sediment mixture; $\rho_o = \rho_w p + \rho_s(i - p)$ is the density of saturated bed; ρ_w, ρ_s are the densities of water and sediment, respectively.

Mass conservation equations for bed material is

$$\frac{\partial z}{\partial t} = \frac{D - E}{1 - p} \tag{7.5}$$

7.2.2. Empirical Functions

For the friction slope, the conventional empirical relation is used, which involves the Manning roughness "n"

$$S_f = \frac{n^2 |AV| AV}{A^2 R^{4/3}} \tag{7.6}$$

R is the hydraulic radius.

Sediment exchange between the flow and the erodible bed involves two distinct mechanisms, i.e., bed sediment entrainment due to turbulence, and sediment deposition due to gravitational action.

Sediment deposition and entrainment are represented in this study by the Equations (7.7) and (7.8) respectively, which were introduced by Cao (1999):

$$D = \alpha c \omega_o (1 - \alpha c)^m \tag{7.7}$$

Where ω_o is the settling velocity; α is the coefficient=min [2, (1-p)/c] and αc is the near bed concentration of sediment which is assumed to be proportional to the depth-averaged concentration.

$$E = \begin{cases} \varphi(\theta - \theta_c) V (A / b_m)^{-1} d^{-0.2} & \text{If } \theta > \theta_c \text{, else} \\ 0 & \end{cases} \tag{7.8}$$

Here following assumptions are made: The free surface velocity is approximated by depth average velocity=7V/6; s =ρ_s/ρ_w-1 =1.65; θ is the Shields parameter; θ_c is the critical Shields parameter=0.045; ν is the kinematic viscosity of water= 1.2E-6 m^2/s; ϕ=0.015[$m^{1.2}$]; m is the exponent=2.

The initial porosity of sediment deposits has been investigated by Hembree et al. (1952), Lane and Koelzer (1953), Colby (1963),Komura (1963), and Han et al (1981),Wu et al (2006) and they made a comparative study for the available literature of initial porosity considering the extensive laboratory data of Trask (1931) and Straub (1935) as well as various the field data from all over the world. After their analysis they proposed the Komura's (1963) expression in a slightly modified form as follows, which has been used in this study.

$$p = 0.13 + \frac{0.21}{(d_{50} + 0.002)^{0.21}} \tag{7.9}$$

Where *d* is the size of sediment in millimeters

Different investigators developed empirical or semi empirical relationships, curves for sediment particle and settling velocity. Some have considered only sediment particles as sphere (Brown and Lawler (2003)). But the particle shape and surface roughness affect the settling process and natural sediment particles deviate from that of spheres which have been also considered by some investigators e.g. Swamee and Ojha, (1991), Dietrich, W. E. (1982), Cheng (1997), Ahrens (2000). Wu et al (2006) replaced the original curves and tables by an explicit mathematical expression for the settling velocity that can be used more conveniently. They made a comparative study and it has been shown that the formula they proposed exhibits better performance than other nine existing formulae available in the literature, in which all consider the effect of particle shape on the sediment settling.

Wu et al (2006)'s formula:

$$\omega_o = \frac{M\nu}{Nd}[\sqrt{\frac{1}{4} + (\frac{4N}{3M^2} D_+^3)^{1/u}} - \frac{1}{2}]^u \tag{7.10}$$

Where

$$D_+ = d[(\rho_s / \rho - 1)g / \nu^2]^{1/3} \tag{7.11}$$

d= nominal diameter of the sediment particles

$$M = 53.5e^{-0.065S_c} \tag{7.12}$$

$$N = 5.65e^{-2.5S_c} \tag{7.13}$$

$$u=0.7+0.9S_c \tag{7.14}$$

Where S_c is the Corey shape factor $= cc/\sqrt{aabb}$ in which aa, bb, and cc are the lengths of the longest, intermediate, and shortest axes of the particle. Here it is assumed =1

7.2.3. Mathematical Model for the Ultimate Cross-Sections after Dam Break

The one-dimensional simulation of the evolution of riverbed appears to be not sufficiently complete if ultimate natural irregular channel-section is found out. If a 2D model is used computational runtime becomes very high and more precise terrain data are required which in most of the cases are difficult to have. Hence effort has been made here to find the ultimate channel section after the dam break flood by analyzing and distributing in that section laterally the mean channel surface evolution obtained by the 1D model

Diplas and Vigilar (1992) have studied the geometry for a threshold channel. They proposed a fifth-polynomial profile to represent the shape of threshold channels with threshold banks and mobile beds channel. In case of dam failure the consequential flow in the channel is highly transient. Hence, it is proposed here for further more detailed method to get the ultimate channel section.

Deformation of the cross-sections has been computed with the hypothesis that: scour and deposit are directly related to shear stress. The shear stress is computed by the Merged Perpendicular method proposed by Khodashenas and Paquier, (1999)

In one cross-section, change in bottom elevation **(**Khodashenas and Paquier, (1999), Khodashenas (2002)**)** in point *j* or segment *j*, change in bottom elevation in point j, $\Delta Z\ j$ initial , is proportional to the sediment discharge rate, q_s , computed in that point j .Considering Meyer-Peter and Muller's relation in point j:

$$\Delta Z_{j\ initial} = \frac{8\Delta t}{(1-p)\Delta x}\sqrt{gd^3(s_s-1)}.\left(\zeta\tau_j^* - \tau_{c_j}^*\right)^{\frac{1}{2}} \qquad (7.15)$$

Where , *d* is the mean sediment size, *g* is the acceleration of gravity, ζ= (*K s /K′ s)*2 is a roughness parameter , K_s is the total Manning-Strickler coefficient, K_s ′ = 21/ $d^{1/6}$ =grain Manning-Strickler coefficient, τ_j^* is the shear stress in point j obtained by M.P.M. (a geometrical method developed to compute the shear stress in an irregular cross section and called Merged Perpendicular Method by Khodashenas and Paquier, (1999) and τ_{jc}^* is the critical shear stress in point j which is computed by relation given in Ikeda, 1982.

Now, from the 1-D simulation model in the previous step the mean bed surface evolution (erosion or deposition) at that particular cross-section is obtained which is represented as ΔS_{1D}. Then the change in surface in the lateral direction in that cross-section is $\Delta S_{2D} = \sum \Delta Z_{jinitial}$.

Then the ultimate cross-section of the river channel after dam break flow is evaluated as ((Khodashenas and Paquier, (1999), (2002)) distributions.

The final evaluation of the channel surface (ΔZ *j* final) at any point j at a cross-section is given as follows:

(i) When $\Delta S_{2D} > \Delta S_{1D}$

$$\sum \Delta Z_{jinitial} > \Delta S_{1D} \Rightarrow C_c \sum_{\Delta S_{jinitial} \geq 0} \Delta Z + \sum_{\Delta Z_{jinitial<o}} \Delta Z_{jinitial} = \Delta S_{1D} \qquad (7.16)$$

$$\Rightarrow C_c = \frac{\Delta S_{1D} - \sum_{\Delta S_{jinitial<0}} \Delta Z_{jinitial}}{\sum_{\Delta S_{jinitial} \geq 0} \Delta Z_{jinitial}} \qquad (7.17)$$

If $\Delta Z_j \geq 0 \Rightarrow \Delta Z_{jfinal} = C_c \times \Delta Z_{jinitial}$

If $\Delta Z_j < 0 \Rightarrow \Delta Z_{jfinal} = \Delta Z_{jinitial}$ (17a)

(ii) When $\Delta S_{2D} < \Delta S_{1D}$

$$\sum \Delta Z_{jinitial} < \Delta S_{1D} \Rightarrow C_c \sum_{\Delta S_{jinitial} < 0} \Delta Z_{jinitial} + \sum_{\Delta Z_{jinitial \geq 0}} \Delta Z_{jinitial} = \Delta S_{1D}$$

$$\Rightarrow C_c = \frac{\Delta S_{1D} - \sum_{\Delta S_{jinitial \geq 0}} \Delta Z_{jinitial}}{\sum_{\Delta S_{jinitial} < 0} \Delta Z_{jinitial}} \tag{7.18}$$

$$\text{If } \Delta Z_j \geq 0 \Rightarrow \Delta Z_{jfinal} = \Delta Z_{jinitial} \tag{7.18a}$$

$$\text{If } \Delta Z_j < 0 \Rightarrow \Delta Z_{jfinal} = C_c \times \Delta Z_{jinitial}$$

7.2.4. Mathematical Formulations

7.2.4.1. Flow Sediment Coupled

Application of higher order schemes or total variation diminishing (TVD) corrections, although brings accuracy in finer level, makes the mathematical treatment complex. The computation time also increases significantly for a real river channel. Therefore, first order "Diffusive Scheme" (Sarma and Saikia (2005), Saikia and Sarma (2006) is used in the proposed model, which is very easy to implement.

By applying diffusive scheme (Chunge et al 1980) to equation (7.1), the flow parameters at any time step "n+1" from the known values at time step "n" is obtained as:

$$U_i^{n+1} = \beta U_i^n + (1-\beta)\frac{U_{i+1}^n + U_{i-1}^n}{2} - \frac{\tau}{2}(F_{i+2}^n - F_{i-1}^n) + \Delta t S_i^n \tag{7.19}$$

Where

β = Weighting coefficient in diffusive scheme and $0 \leq \beta \leq 1$;

$\tau = \frac{\Delta t}{\Delta x}$,

β =0.75 has been considered here.

The bed evolution is computed from equation (7.5)

$$Z_i^{n+1} = Z_i^n + \frac{\Delta t}{(21-p)}[(D-E)_i^n + (D-E)_i^{n+1}] \tag{7.20}$$

7.2.4.2. Flow Sediment Decoupled

As variation in riverbed is small in one time step compared to the flow, in analyzing the dam break flow different investigators (Paquire and Balayn (1998), Paquire (2002)) solved the third equation in governing equations (7.1) separately with similar numerical schemes. Suitable numerical solutions have been observed in literature comparable to real laboratory test values. Hence, decoupled model has been also applied here with same numerical scheme as in coupled solution to simulate the flow and riverbed profile under the failure of Dibang dam.

7.3. Analysis of the Simulation Results

Bed mobility can modify the hydrographs and free water surface profiles as illustrated in figure 7.1, 7.2 and 7.3. . The free surface profiles and riverbed profiles are plotted at time 20s, 60s, 200s and 300s after the dam failure. It has been observed that significant scouring is there in the initial stage at the dam site but afterwards, rate of scouring decreases. There is remarkable depression in bed from 20 sec to 200 sec, but the change in the bed level is very less from 200 sec to 300 sec. The flow wave propagates slowly over a mobile bed as compared to its propagation speed over a fixed bed. After the failure of dam, the mass picked up from the bottom, where it lays at rest and than it gains the momentum determined by the flow velocity. This momentum exchange, with no energy dissipation, determines not only an increase of the mass involved in the flow, but also a consequent reduction of the flow velocity. Because of that, the advancement of the dam-break wave becomes slower. The discharge hydrographs at three channel cross sections (one upstream the dam, one at dam site and the other downstream the dam) considering both fixed and mobile bed are plotted in figure 7.2. The changes in the riverbed elevations up to 200 seconds at four cross sections are presented in figure 7.4.

The most of the early 1-D sediment transport models were decoupled, which resulted in simpler computer codes as flow values were obtained with St. Venant equations and sediments are separately obtained. This strategy was justified because of the different time scales of flow and sediment transport. Most of the cases taken for dam break flow over mobile bed for applications were in simple laboratory channels. e.g., (Paquire and

Balayn (1999)) compare their solution with the experiments performed in 1991 (Gendreau and Séchet, 1992) in the laboratory of the IMFT (Institute of Fluid Mechanics of Toulouse, France). A Plexiglas rectangular channel (0.2 m wide) was fed by a reservoir through a gate, which was suddenly opened. The change in channel bed is comparatively quite less than in the change in flow parameters. Paquier (2002) used his model to the real dam break case of Lawn Lake dam located in Colorado (U. S. A.), which broke on July 15, 1982. The height of the dam was only about 8 m and reservoir contained 830,000 m^3, when failed. But the decoupled model cannot at all be recommended for large dam break with high transient flows and bed deformations as the flow becomes highly supercritical where high erosion is there near the dam site. Hence flow of sediment (river bed) interaction through coupling is clearly important in case of failure of large dams. In case of failure of Dibang dam the decupled model fails to provide solution after 20 seconds. The results of the decoupled model are presented in figure 7.5

The analysis clearly illustrates that the change in topography of channel modifies the predictions of flow parameters. However, in such cases another important point to note is that, the change in the topography is not limited to a rise (or a lowering) of the bottom but implies a completely different cross section. Hence change in cross-sections of natural river channel due to the dam break flow is also found out as per the mathematical formulation as described in section 7.2.3. Two cross-sections at 4 km and 26 km upstream of the dam and two other at 200m and 1500 m downstream of the dam are presented figures 7.6, 7.7, 7.8, 7.9 respectively.

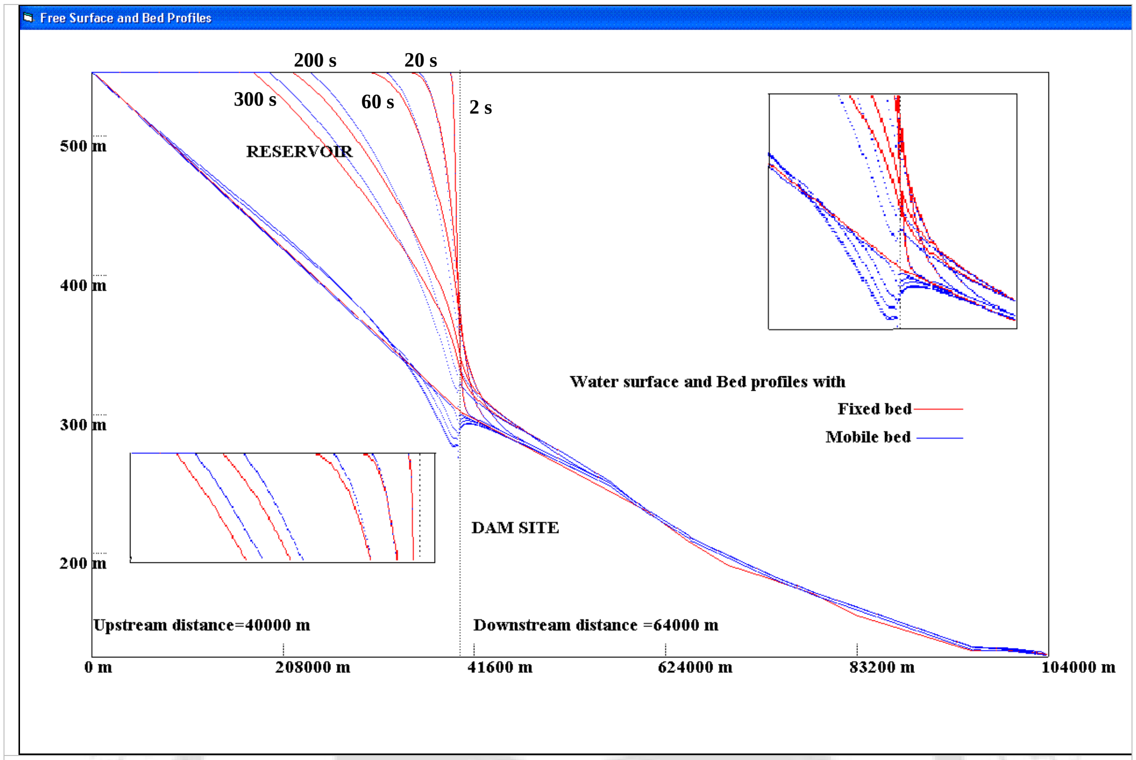

Figure 7.1: Free surface Flow Profiles and the Bed Profiles by the Proposed Model and it's Comparison with the Profiles Obtained when the Riverbed is Assumed Fixed

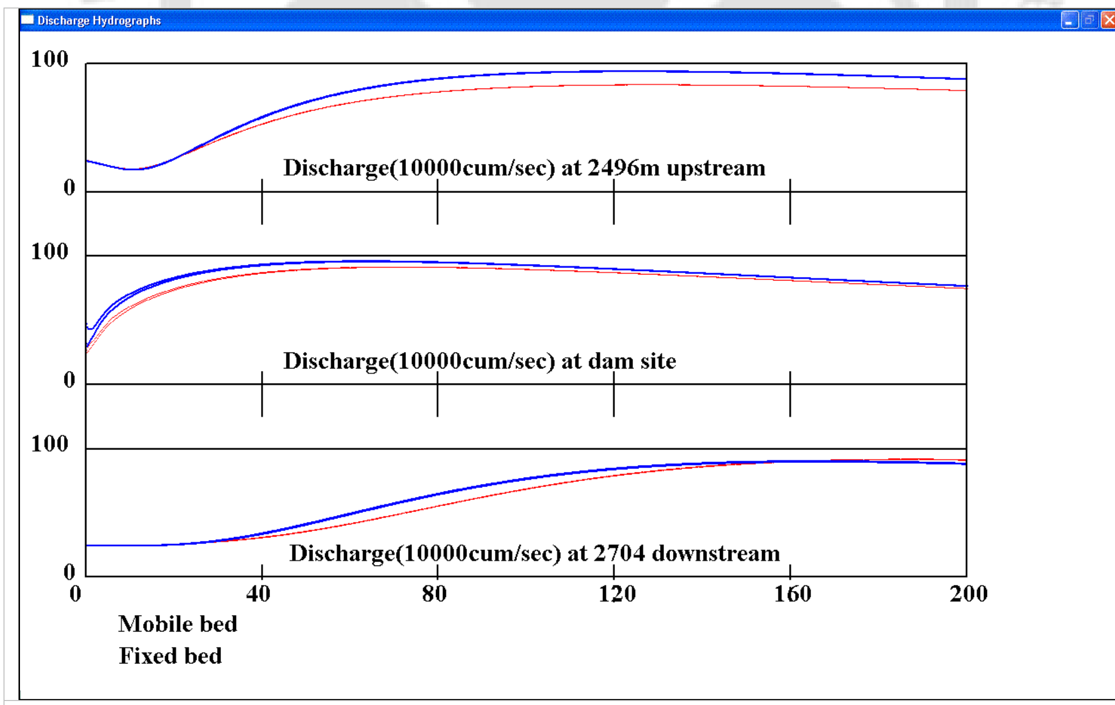

Figure 7.2 Comparison of the Discharge Hydrographs for Mobile and Fixed Riverbed

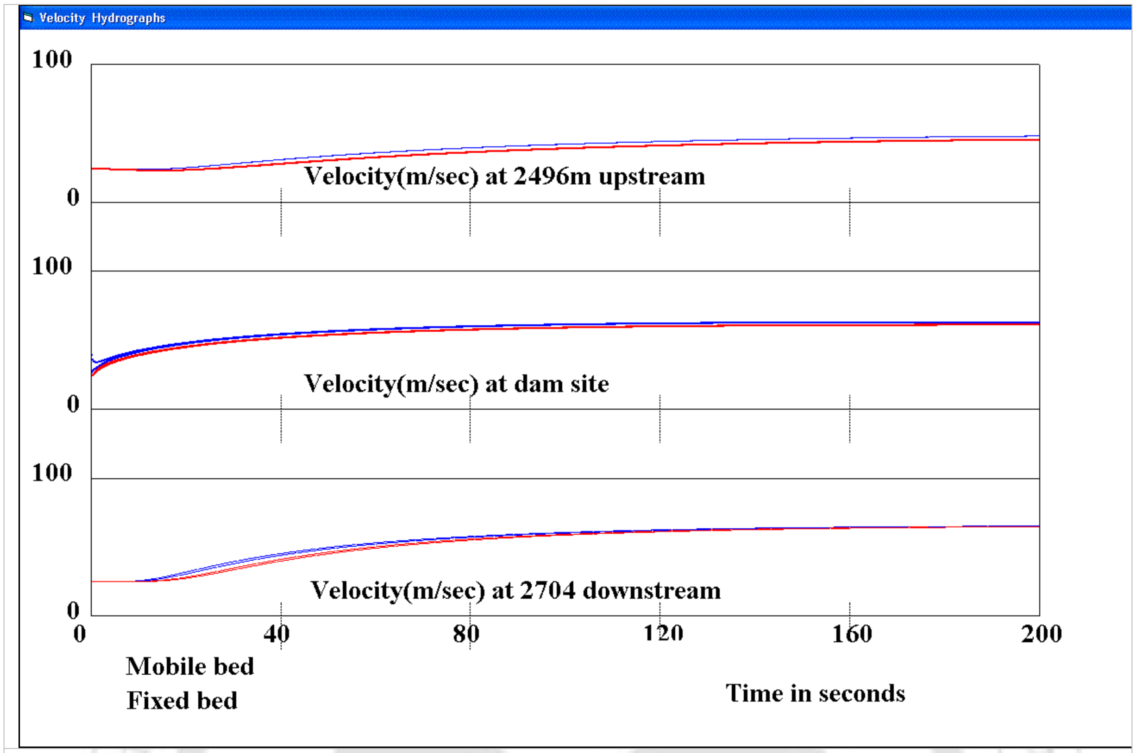

Figure 7.3 Comparison of the Velocity Hydrographs for Mobile and Fixed Riverbed

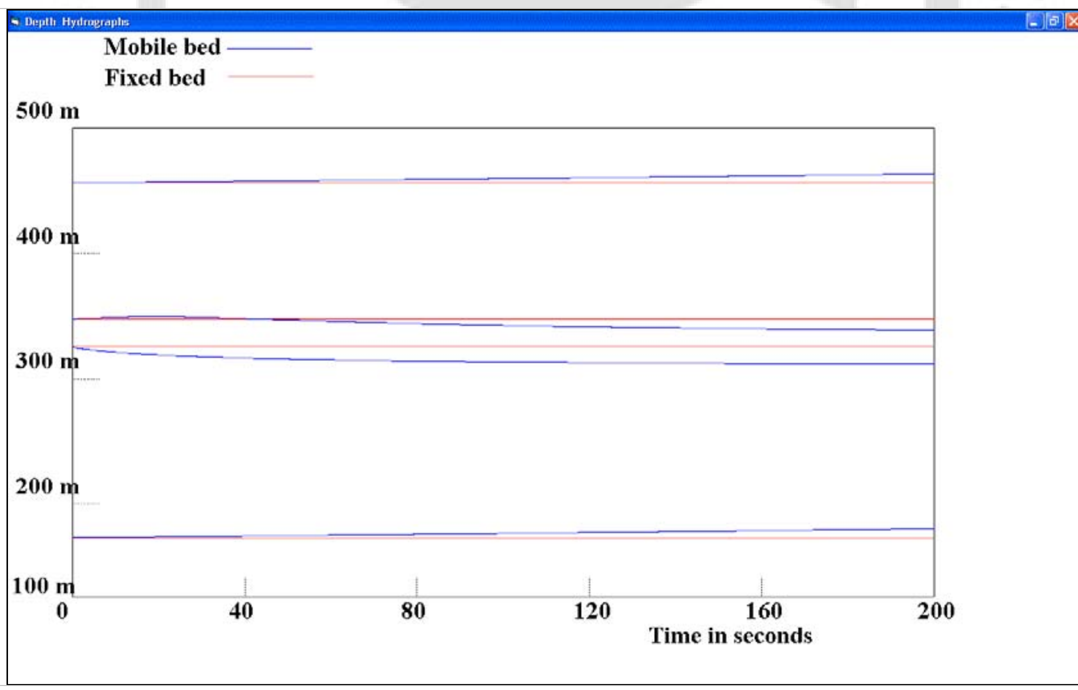

Figure 7.4: Changes in the Bed Elevations at 21 m, 280 m and 3, 600 m upstream the Dam, at Dam site, and at 51,520m downstream the dam respectively (from top to bottom)

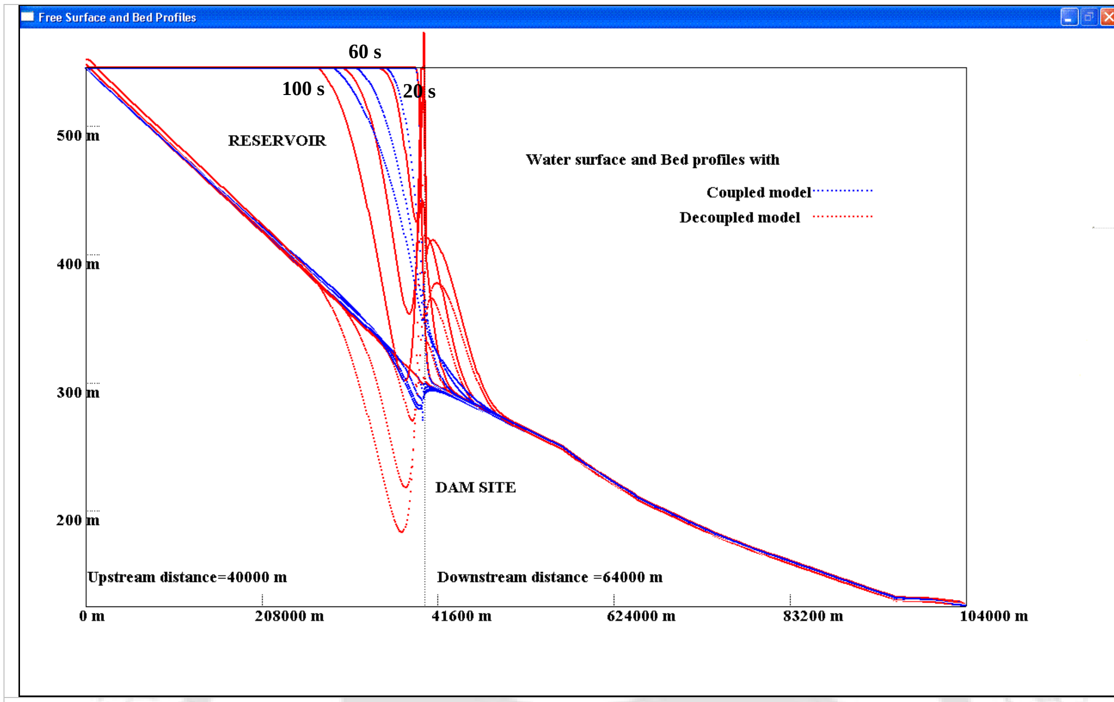

Figure 7.5 Comparison Free Surface Flow Profiles and the Bed Profiles by Coupled and Decoupled Models

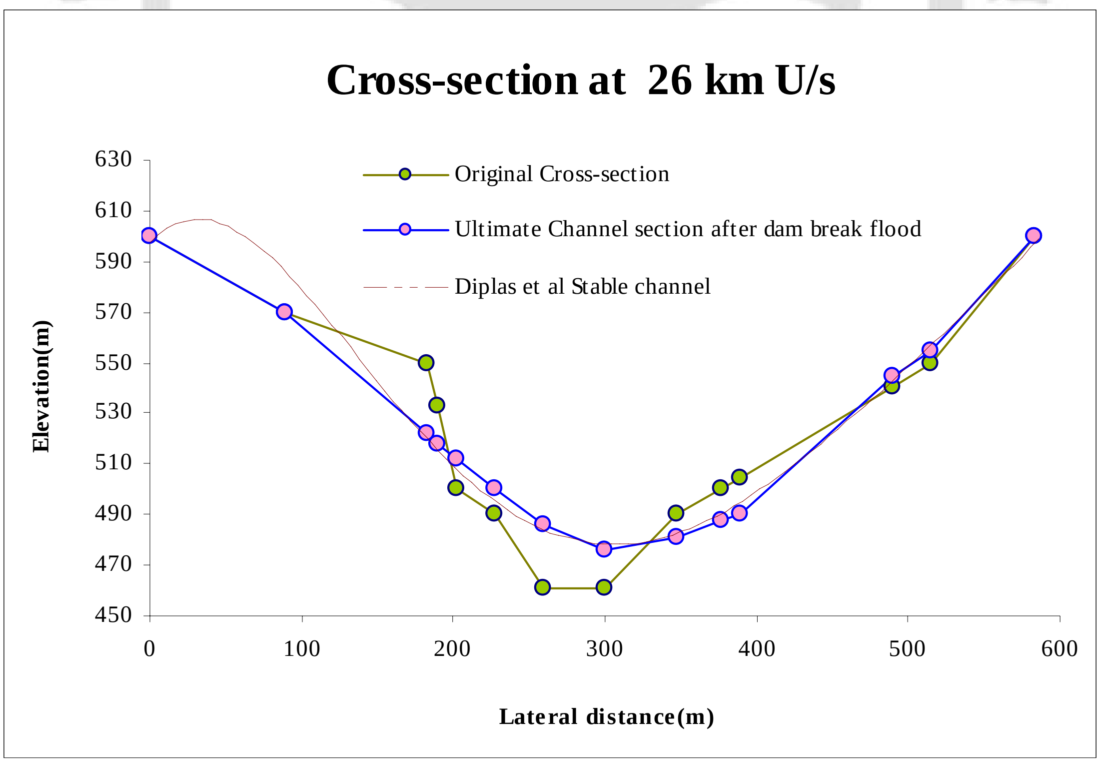

Figure 7.6 Change in the Cross-Section at 26 km upstream

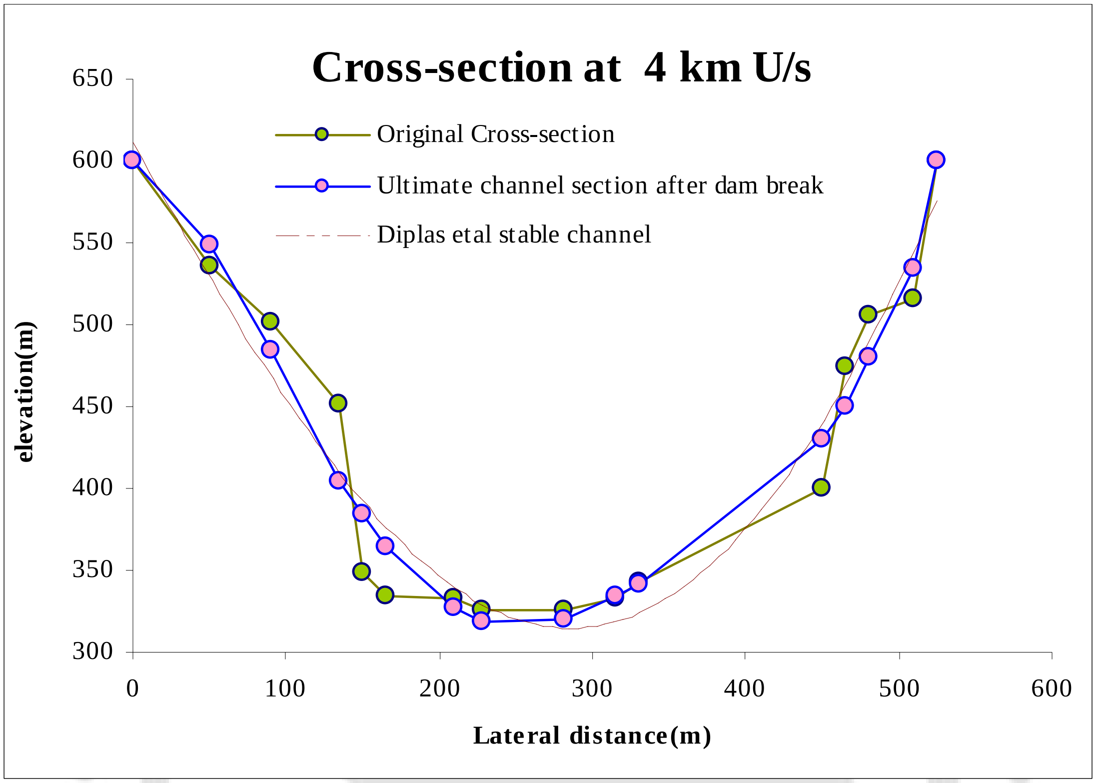

Figure 7.7 Change in the Cross-Section at 4 km upstream

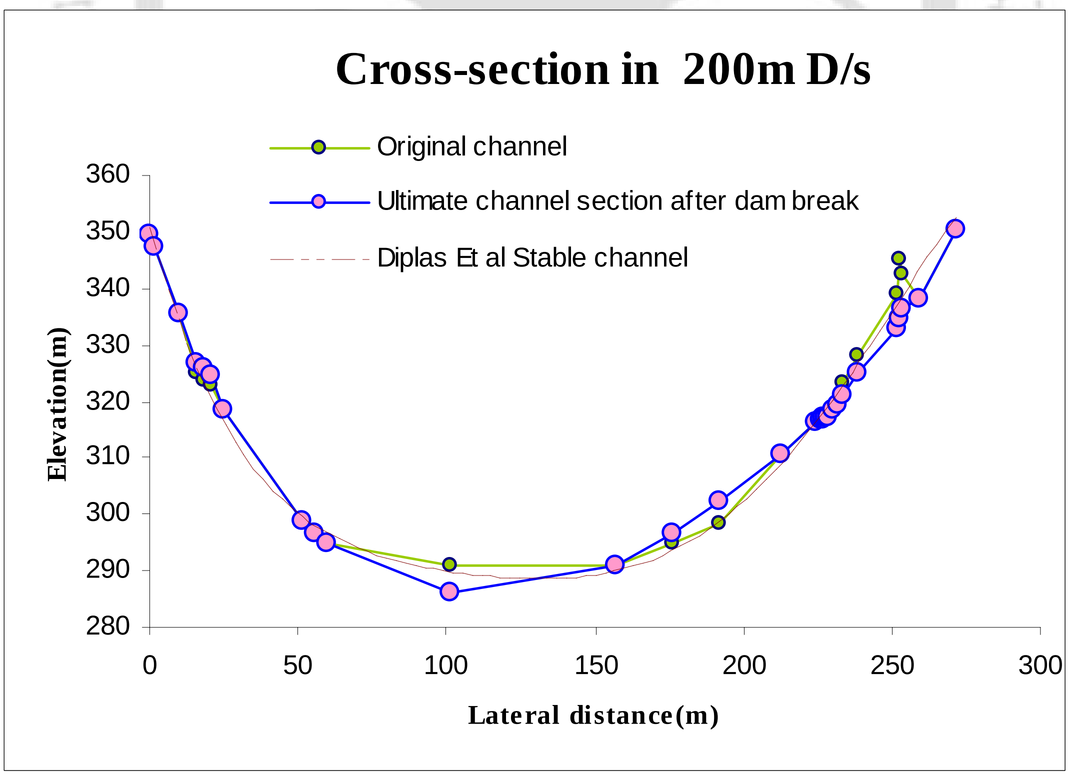

Figure 7.8 Change in the Cross-Section at 200m downstream

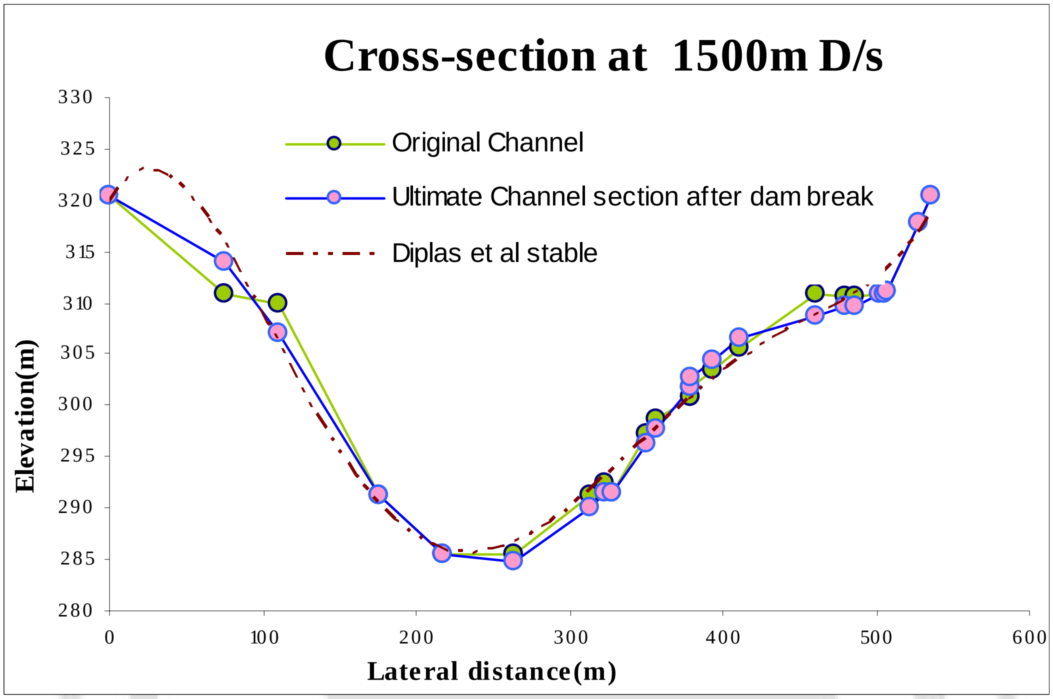

Figure 7.9 Change in the Cross-Section at 1500m downstream

7.4. Conclusion

Dam-break hydraulics in a real river channel over an erodible bed is studied. Simulated water-surface and riverbed profiles clearly indicate the following:

(i) There is a considerable change in the riverbed at the dam site under the highly transient flow condition occurring due to the failure of a large dam.

(ii) Consideration of bed mobility in the dam break flow model modifies the free surface flow profiles and the hydrographs, which is most important for predicting dam break flood by the model.

(iii) The mobile bed can be significantly scoured in the initial stage; the dimensions of the scour hole are of quite comparable magnitudes to those of the flow itself

(iv) The rate of bed deformation signifies the need for the coupled modeling of the strongly interacting flow-sediment-morphology system under dam break flow.

(v) The decoupled model which was illustrated earlier to provide reasonable solutions cannot at all be recommended for large dam break with high transient flows and bed deformations. In case of Dibang dam failure the decoupled model fails to provide solutions after 20 seconds.

Hence a simple FD model with coupled flow and sediment equations is formulated here for simulating the flow parameters and bed profiles in a natural river due to the failure of a large dam incorporating morphological changes of the river bed. However, minor numerical difference may be there in the simulated profiles with the real one, as it is difficult to pinpoint roughness parameter for natural rivers, especially when sediment transport is involved and formulations for the sediment entrainment and deposition fluxes are empirical. Moreover the different empirical formulae as available in literature have been used here to start a study for analysis of movement of the flow due the failure of a large dam in mobile bed and bank conditions. No modified from of theses empirical formulae are available to use it in dam break case. Modification of those formulae is an obvious requirement as flows are highly transient and further extensive study is required for these modifications. Hence it is proposed here to use the model developed with fixed bed in the previous chapter for prediction of Dibang dam failure flood.

Chapter-8

Model Assessment

8.1. Malpasset Dam Break(France)

The dam failed in 1959, at 21:14 hours on 2nd December due to an exceptionally heavy rain. A total of 433 casualties were reported. The Malpasset dam-break case was selected as **a benchmark test example for dam-break models** in the CADAM (1996-1999) project due to its complex topography and availability of measured data.

8.2. Topography

Dam was located in a narrow gorge of the Reyran river valley approximately 12 km upstream of Frejus on the French Riviera. Immediate downstream of the dam, the Reyran river valley is very narrow with two consecutive sharp bends. It widens as it goes downstream and eventually reaches the flat plain.

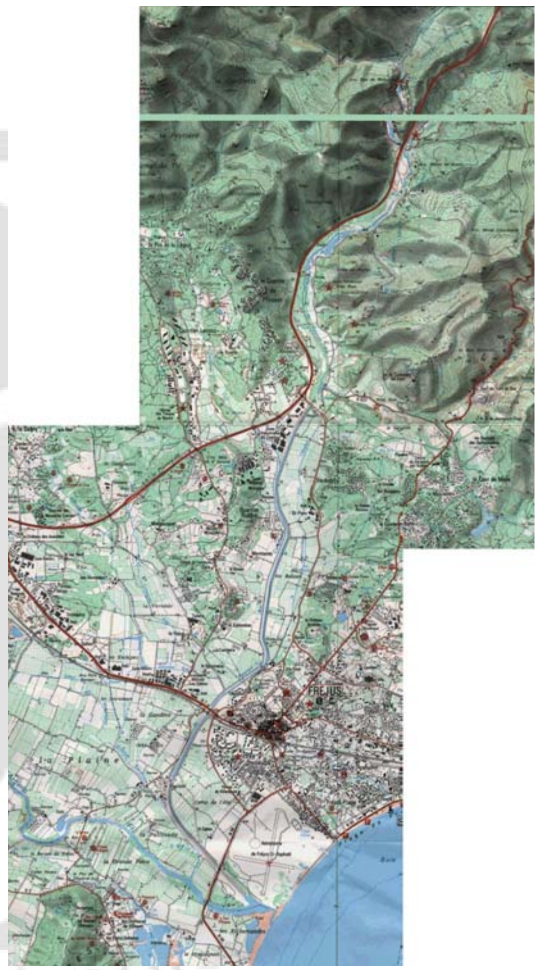

Figure 8.1: Map of the Valley

8.3. Description of the Dam

(i) **Type -** arch dam

(ii) **Height-**66.5 m

(iii) **Crest length -** 223 m

(iv) **Maximum reservoir capacity-** 55.106 m^3

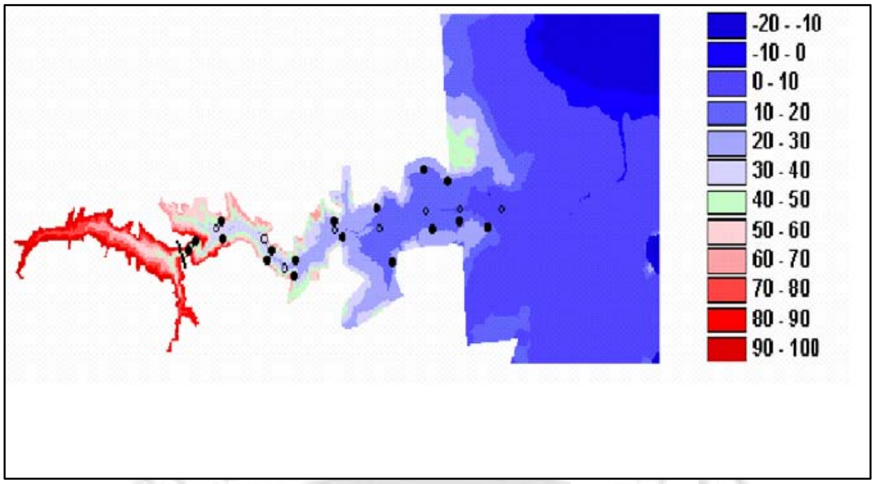

Figure 8.2: Ground Elevations in meters of Malpasset Dam Natural Valley…

8.4. Dam Break Data Available

8.4.1. Field Data Available

After the dam failure, a field survey was performed by Police. Total 17 points have been surveyed in both the banks to obtain the maximum water levels along the river valley. Times of shut down of 3 transformers were also noted down.

8.4.2. Physical Model Data

A physical model with a scale of 1/400 was built to study the dam-break flow in the laboratory of EDF, France 1964. At 9 points along the river valley following are measured:

(i) The maximum water level

(ii) flood wave arrival time

8.5. Initial and Boundary Conditions

(i) The reservoir level elevation may be taken as constant = 100 m.

(ii) The elevation of the sea is constant = 0 m.

(iii) Upstream of the reservoir = zero discharge.

8.6. Roughness Value

Schwanenberg and Harms (2004) have simulated Malpasset dam break flood with different roughness coefficients ranging form Mannings n=0.025 to n=0.035. While comparing the computed flood values with the real dam break data, they have found that, computations with Mannings' roughness value n=0.03333 have given best results. Hence in this study this value i.e., n=.003333 is used as the roughness value for flood computations.

8.7. Topography Data Available

Two data sets of data are available

(i) Ground Elevations of 13541gorund points

(ii) 70 numbers of the computational channel cross-sections

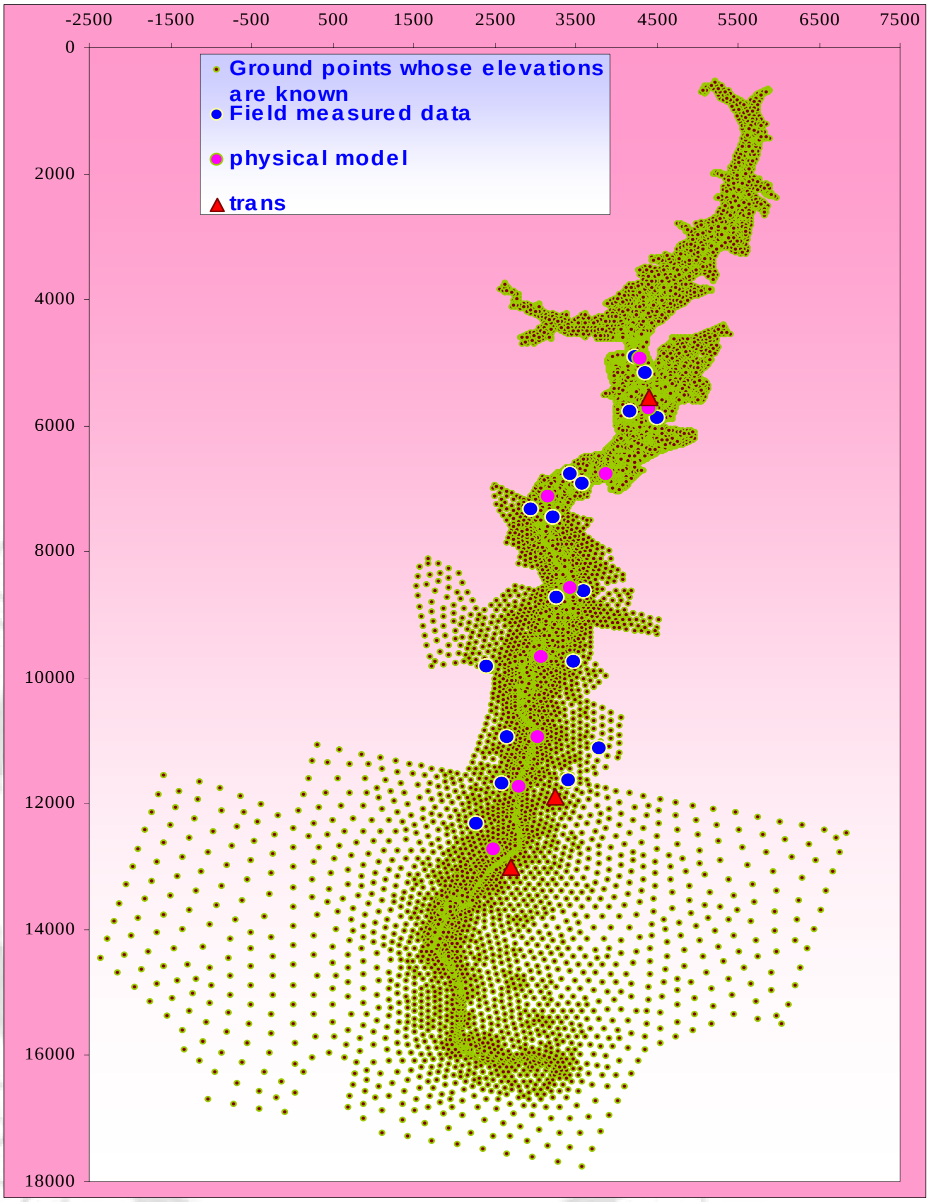

Figure 8.3: Ground points where elevations are available

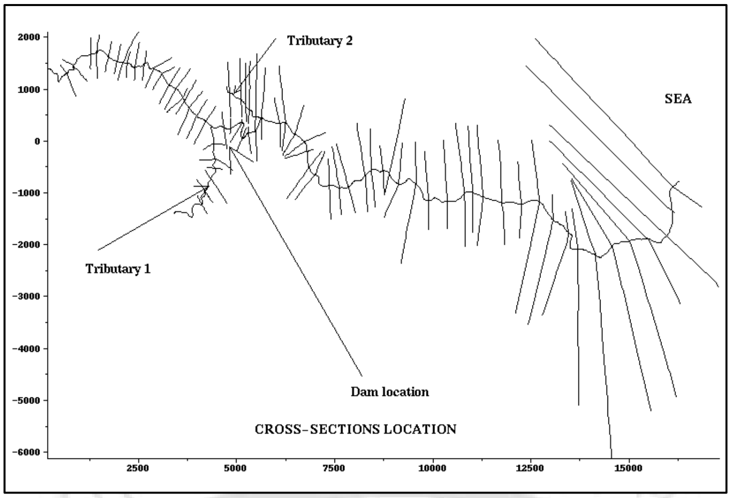

Figure 8.4: Cross-sections containing the whole floodplain:

8.8. Computational Cross sections

8.8.1. Simplified Floodplain 1D model

A parabolic computational channel has been considered here. To get the channel cross-section at a particular distance downstream, the ground elevation data available in the lateral direction are taken from the first topography data set of CACADM considering the whole floodplain area. Then and the parabolic least curve for those ground elevation data has been fitted. For example two computational channel section ground points P7 and P17 are shown in figures8.5 and 8.6 respectively.

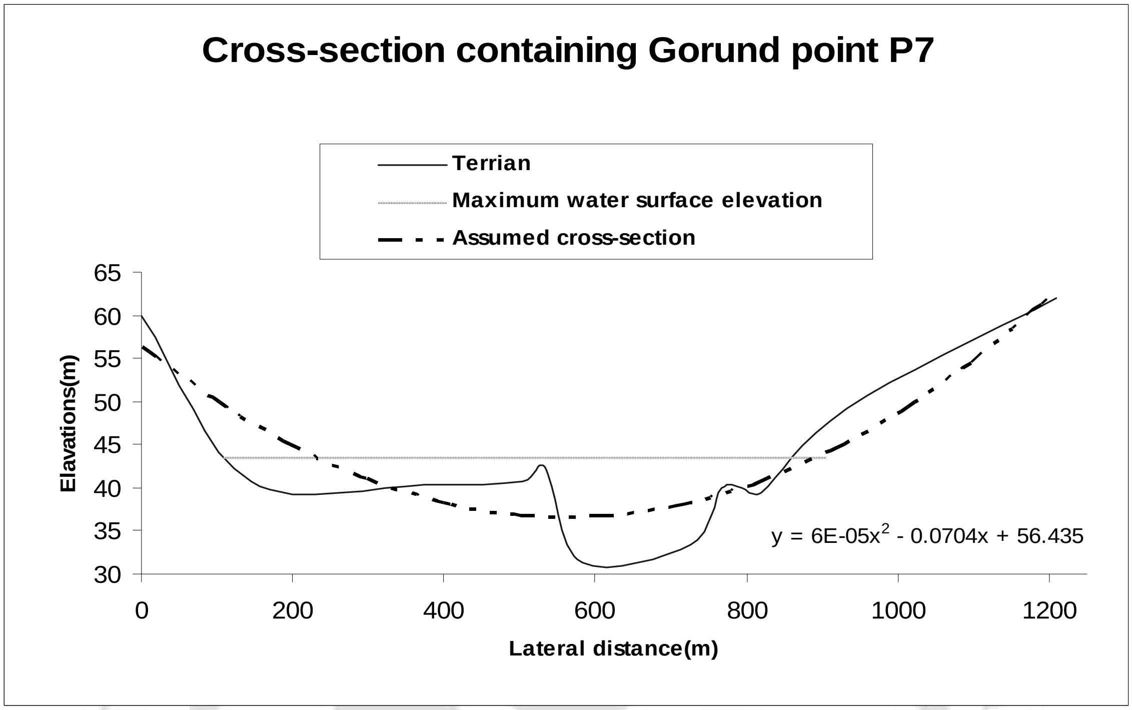

Figure 8.5: Channel Cross-Section in Simplified Floodplain 1-D Model through Ground Point P7

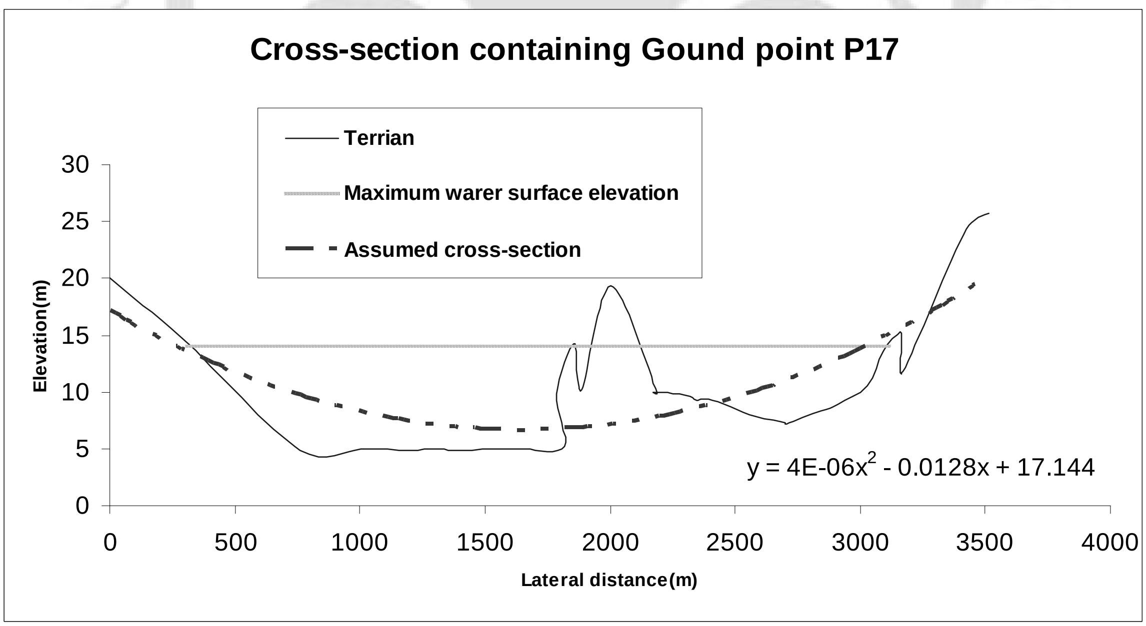

Figure 8.6: Channel Cross-Section in Simplified Floodplain 1-D Model through Ground Point P17

8.9. Analysis of the Computations

The dam break flood is of such a high magnitude that it will not confine only in the river channel. Hence the accuracy of flood prediction depends on the channel considered for computations. For analyzing the importance of it, the computations are done with a channel considering the second dataset available for Malpasset topography also. While considering the predictions with this computational channel, it has been found that the computed maximum water levels are estimated higher than the computations with the channel as considered by the author i.e., simplified floodplain channel. For example, in case of the computations with the channel as available in CADAM second data set, the maximum water surface elevation at a cross-section containing point S3 is obtained as 26.15m. But when author's simplified floodplain channel has been used for computations, maximum water surface elevation is predicted as 18.25m. The maximum water surface elevation obtained in that section by the physical model is 17.8 m.

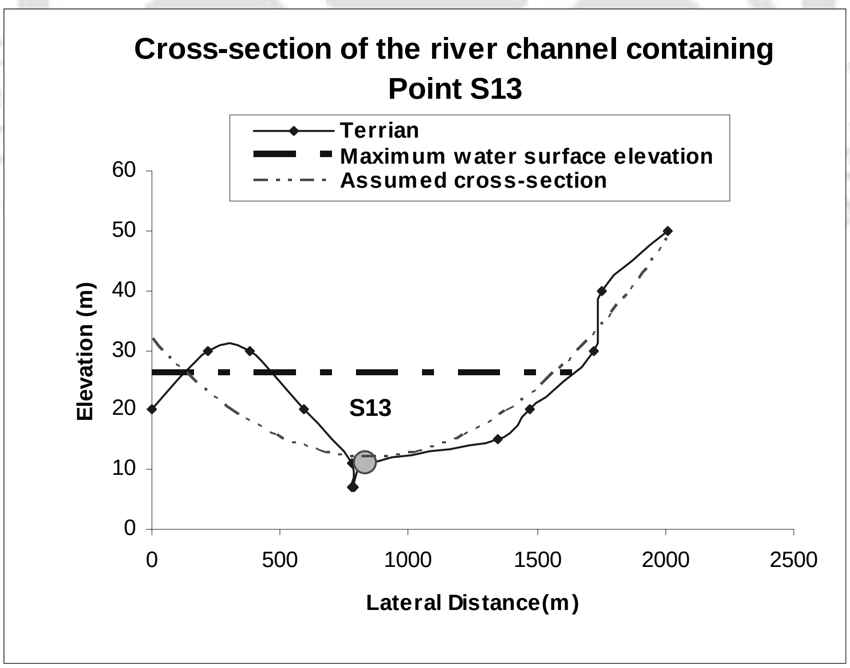

Figure 8.7: Maximum Water Surface Elevation=26.15m

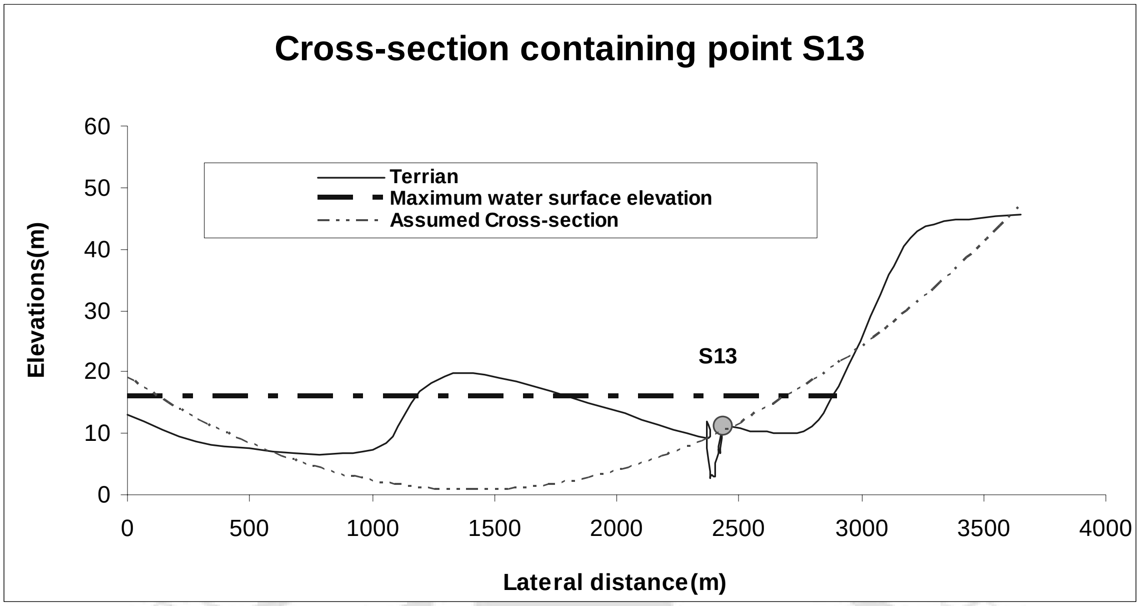

Figure 8.8: Maximum Water Surface Elevation=18.25m

8.10. Comparison of the. computed flood with Real Dam break Data

The comparisons of shutdown times of the three transformers downstream by simplified floodplain 1D model are presented in figure 8.9. To compare time for transformer shutdown, both first wave arrival time and peak depth arrival time are plotted. The transformer shut down times as recorded by Police in case of the real dam break case, are found in between these two times i.e. between the time of first wave arrival and the time peak depth arrival in those sections. Figure 8.10 represents the comparisons of the predicted water surface elevations by simplified floodplain 1D model to the real dam data obtained by Police at 17 ground points (P1 to P17) on both the sides of the original river channel.

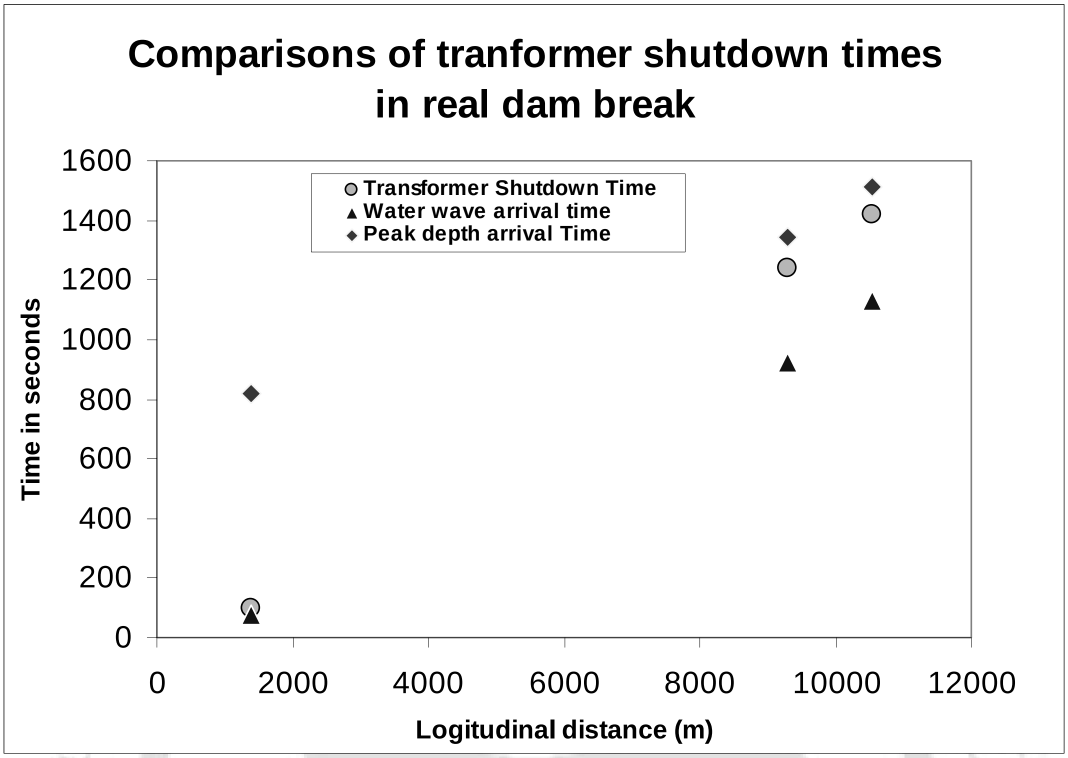

Figure 8.9: Comparison of Transformer Shutdown Times in Real Dam Break

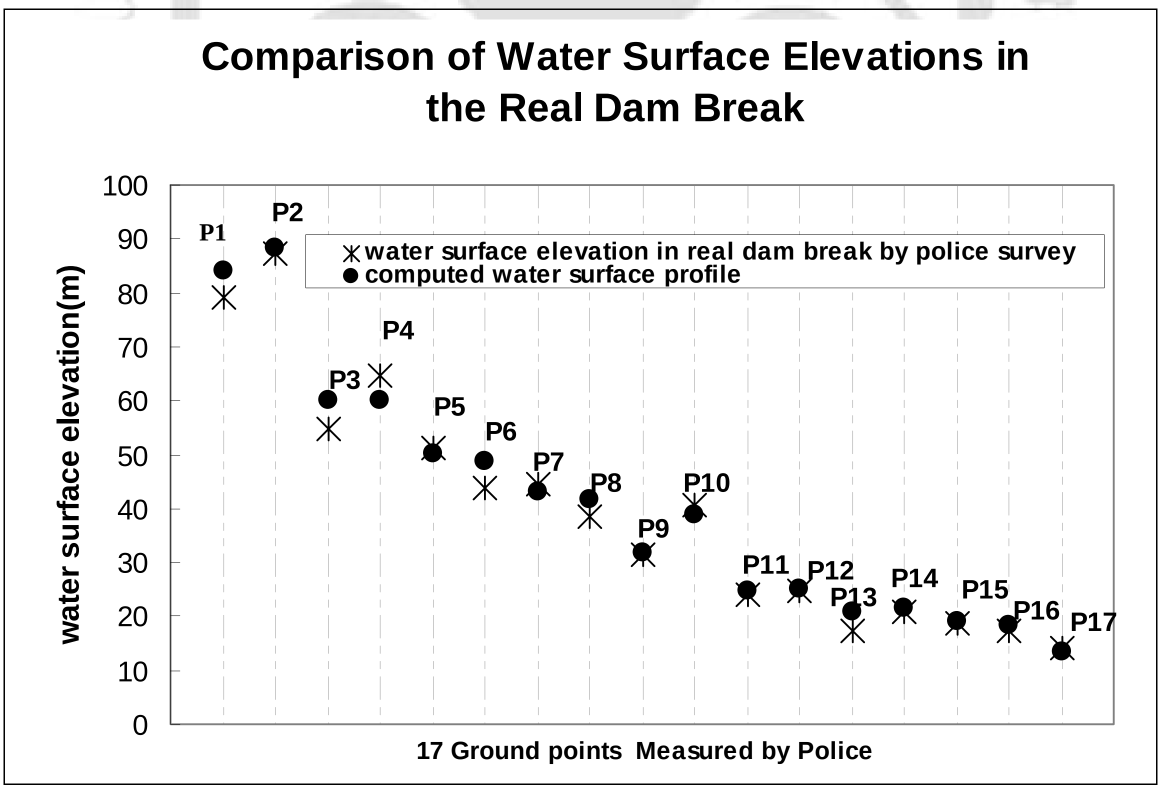

Figure 8.10: Comparison of Water Surface Elevations in Real Dam Break

8.11. Comparison of the. computed flood with Physical Model data

At 9 ground points (S6 to S14) along the river valley, the computed water surface elevations and wave arrival times by simplified floodplain 1D model are compared to the values obtained in the physical model and they are presented in figures 8.8and, 8.12 respectively:

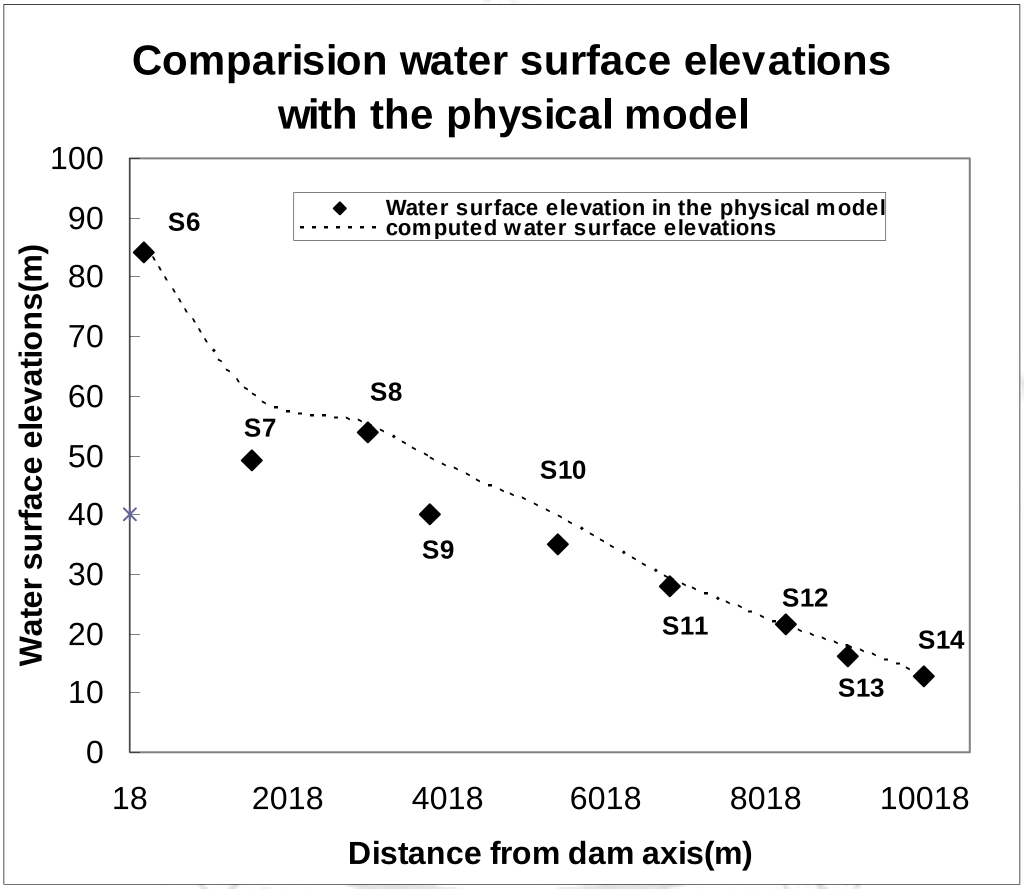

Figure 8.11: Comparison of Water Surface Elevations with the Physical Model

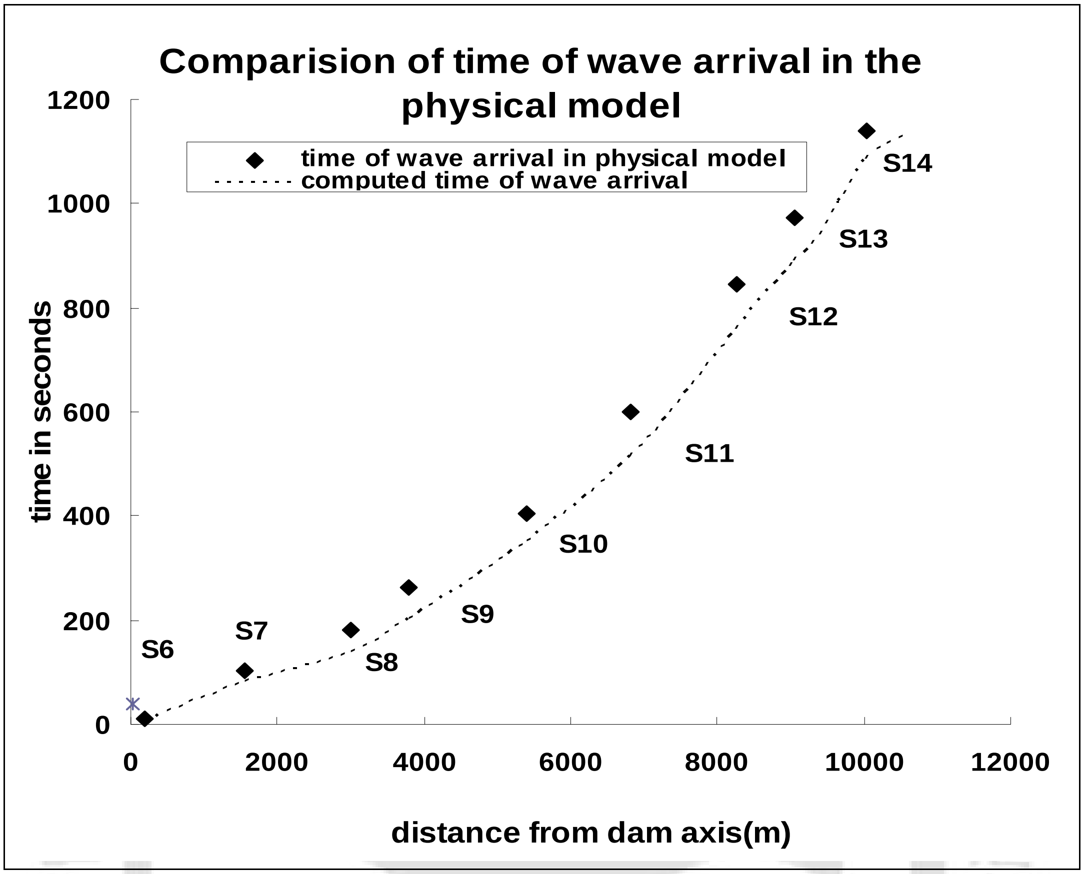

Figure 8.12: Comparison of Time of Wave Arrival in the Physical Model

8.12. Conclusions

(i) Computational channel assumptions have modified the predicted maximum flood level, e.g., at cross-section containing gauge S13, the maximum water level computed considering the channel as given in second data set of CADAM is 26.15 m, while with simplified floodplain 1D model it is 18.25m which is close to the actual value 17.8m as obtained in the physical model experiment.

(ii) The actual times of shutdown of the transformers as measured by police after the real dam break, are found in between the computed time of first wave arrival time and time of peak depth arrivals.

(iii) Maximum flood predicted and the actual dam break flood level shows that computed values are comparable with estimation on higher side except 3 (P4,P7,P10) points

(iv) In the ground points P7, P13, P14, P16, P17, no flood depths were computed by 2D models as observed in the literature earlier (Frazao and Zech). But police measured flood depths in the real dam break case in these points. In these points well comparable flood levels have been observed by author's simplified floodplain 1 D model.

(v) Incase of comparison of the computed values with the physical model test case the predictions of the flood depth are slightly higher and peak arrival times are lower in simplified floodplain 1D model.

Chapter-9

Flood Forecasting, Inundation Mapping, Damage Estimation and Mitigation Measures

9.1. Introduction

The four dam-break models considering the natural floodplain topography of Dibang have been developed, analyzed and compared in the chapter-6. Out of these four models, simplified floodplain 1-D model has been found to be suitable for flood forecasting, inundation mapping, damage estimation and mitigation measures in case of failure of the dam in a complex floodplain topography. The flood parameters obtained by this model are quite comparable with slightly higher estimations. Thus the predictions are on the safer side so far the flood prediction; early warning and mitigation plans are concerned. Moreover, the model is quite easy to implement as compared to the composite channel sections model or the 2-D model which require significant effort in preparing database of the channel cross-sections, ground elevations etc. Therefore, the proposed simplified floodplain 1-D model with parabolic section is suitable to use for prediction of dam break flood in such complex floodplain topography.

Hence the simplified floodplain 1-D model of the Dam-break flood has been used and the following important flood parameters have been forecasted for the Dibang dam failure:

- (i) Maximum flood depths at downstream channel /valley cross-sections
- (ii) Maximum flow discharges at downstream channel /valley cross-sections
- (iii) Maximum flow velocities at downstream channel /valley cross-sections
- (iv) Time of the maximum flood depth arrivals at various locations in the downstream valley
- (v) An inundation map depicting the aerial extend of flooding

A generalized method for damage estimation for that region has been made by method of curve fitting form the available historical flood data of Assam from 1953 to 2005 and the necessary disaster management plan for Dibang dam has been also proposed in this chapter.

9.2. Flood Forecasting at the Downstream Section due to Failure of Dibang Dam

The maximum probable flood depths, maximum peak and maximum probable discharges and velocities at the sections downstream the dam are presented in figures 9.1,9.2 and 9.3 respectively. Figure 9.4 represents the time of peak arrival at downstream sections.

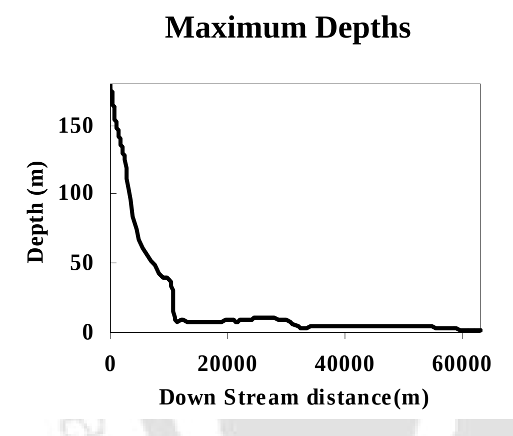

Figure 9.1: Maximum Predicted Flood Depths

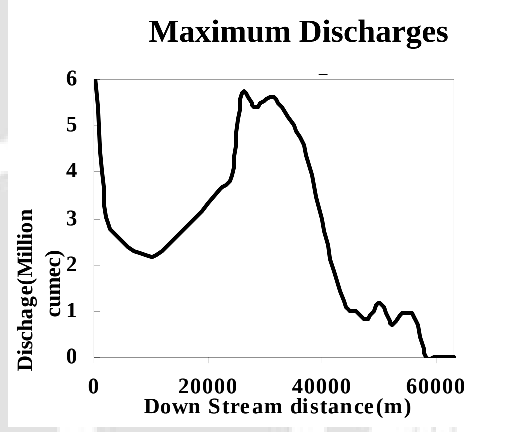

Figure 9.2: Maximum Predicted Flood Discharges

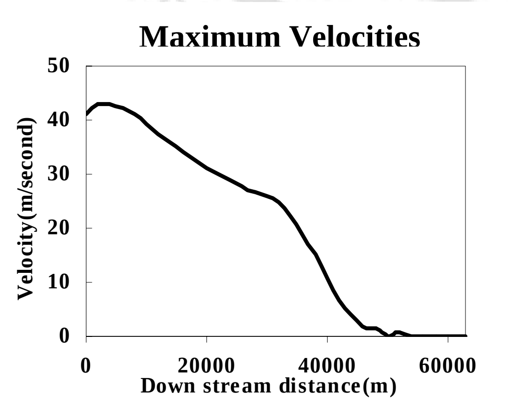

Figure 9.3: Maximum Predicted Velocities

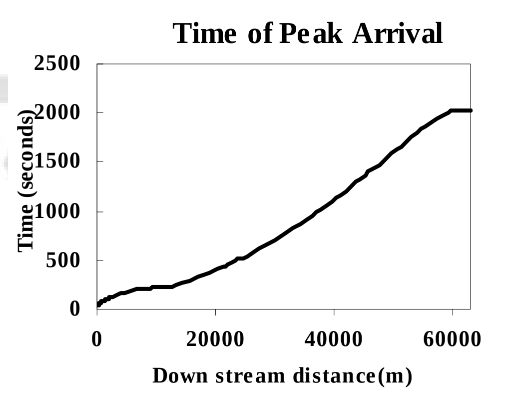

Figure 9.4: Time of Arrival of Peak Flood Depths

9.3. Inundation Mapping

9.3.1. Prediction of Maximum Inundation Depth at a Point Downstream the Dam by Simplified Floodplain 1D Model

The maximum flood depths predicted in different cross-sections downstream of Dibang dam are presented in figure 9.1 above. Now, if the maximum flood depth in a cross-section is known, one can predict the maximum inundation depths at different important ground points present in that computational channel cross-section.

For example, at 40 km down stream of the Dibang dam the maximum inundation depths at the different villages, roads are to be found out. The maximum flood depth as shown in figure 9.1 at 40 km downstream is 9.85m. The channel bed elevation at 40 km downstream is 142.52m. It is the elevation of the centre point the parabolic cross-section obtained by least square curve fitting of the available ground data for that downstream cross-section. Hence the maximum predicted flood surface elevation at this considered downstream section is (9.85+142.52=) 152.37m. Then the water surface profile is plotted and it touches the ground points laterally at15924.80 m and 46852.60m respectively. Hence the inundation will extend laterally from 15924.80 m to 46852.60m at section 40km downstream. Now, knowing the ground elevation of a point at this considered section the maximum depth of flood at that point can be computed as the difference in the elevation of the water surface to the elevation of that ground point. In the two villages "Bijari village" and "Barguli village" maximum depths of flood are 9.15 and 8.55 m respectively. Where, another village Bolung and the nearby Tinsukia -Roing road are not going to be effected by the flood. The Guwahati –Pasighat road passing nearby the Barguli village at 40 km downstream the dam will be inundated with a maximum probable depth of 2.50 m.

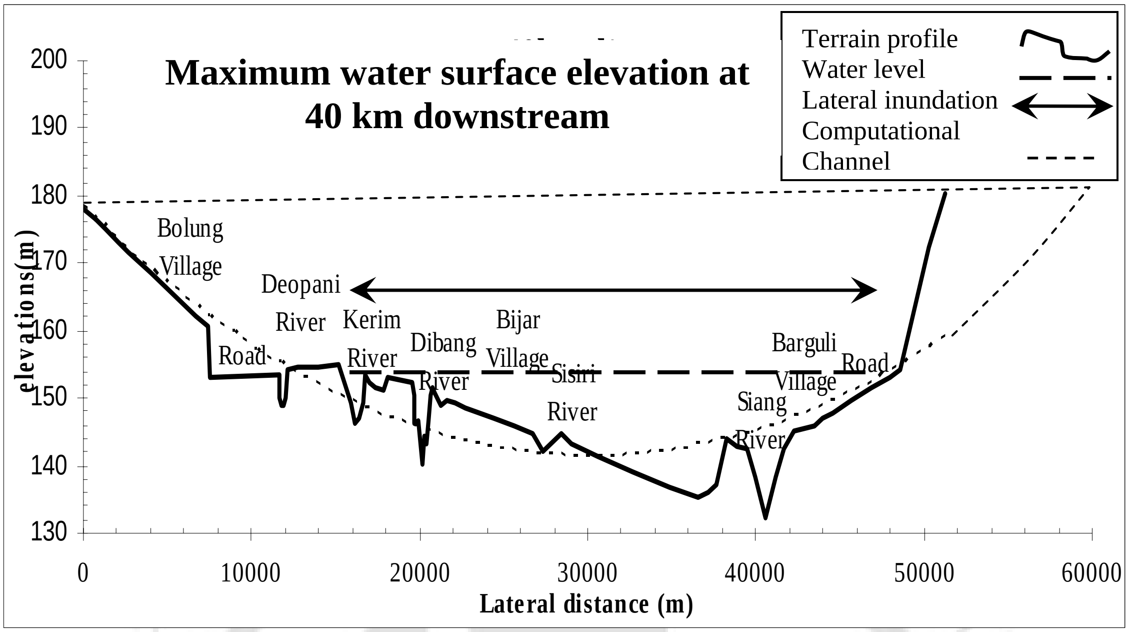

Figure 9.5: Maximum Probable Inundations at 40 km downstream

9.3.2. Dam Break Flood Inundation Map

To prepare dam break flood inundation map 2D models are generally used which are computationally costly in terms of run time and input data preparation. But the developed simplified floodplain 1D model unlike the other 1D dam break models, is capable of estimating extend of lateral inundation also at a cross-section downstream as described in the previous section i.e., in section 9.2.2. In figure 9.6 the predicted inundation by simplified 1D model at downstream of Dibang dam after 300seconds and at 1000 seconds of Dam failure are plotted. The maximum predicted inundation downstream due to failure of Dibang dam is also plotted in that figure (figure 9.2). It will occur after 2300 seconds of Dibang dam failure. The 3 D views of maximum flood depths and inundation downstream predicted by simplified floodplain 1D model are presented in figures 9.7, 9.8, 9.9 respectively.

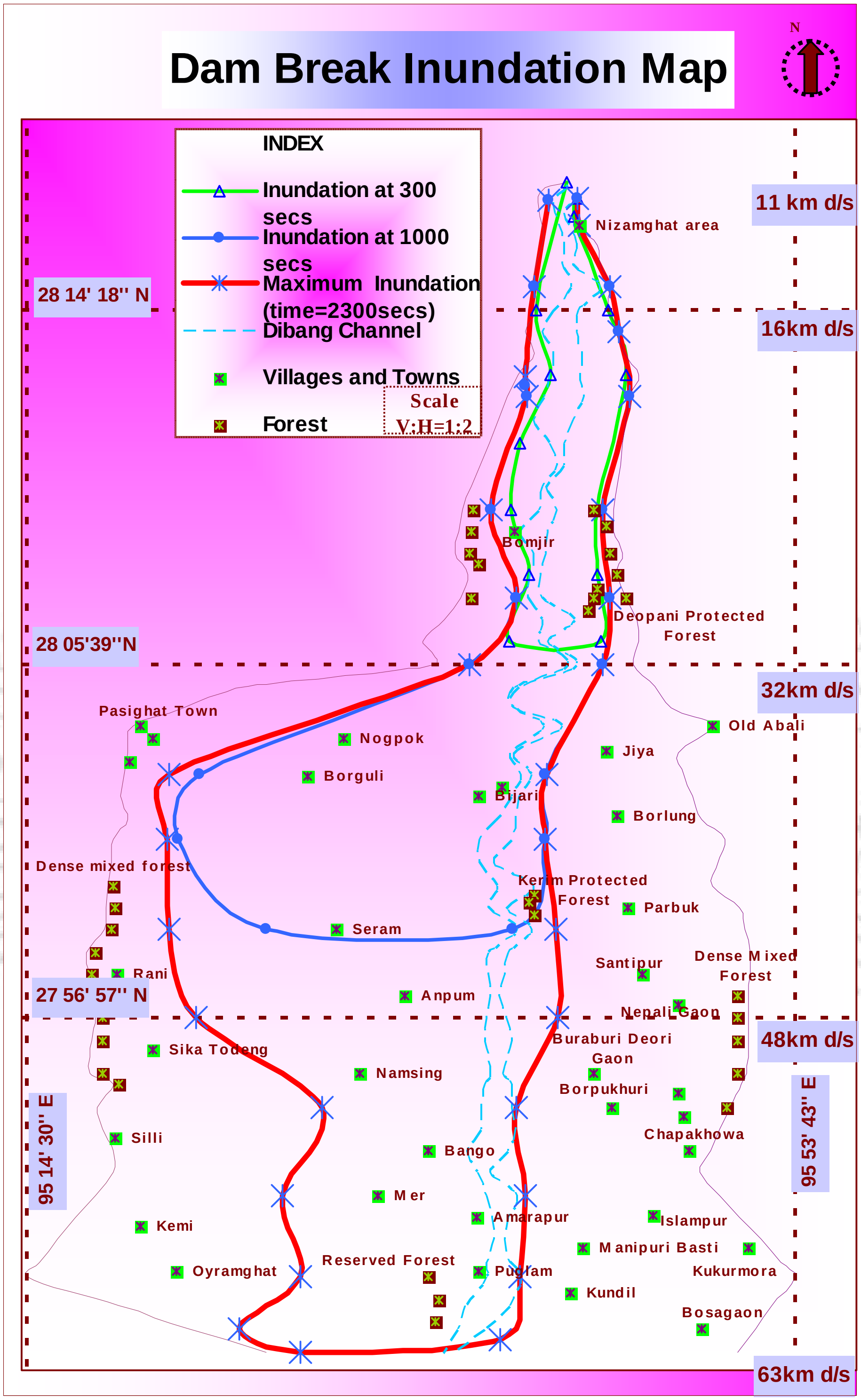

Figure 9.6: Predicted Downstream Inundations due to Dibang Dam Failure by Simplified Floodplain 1D model

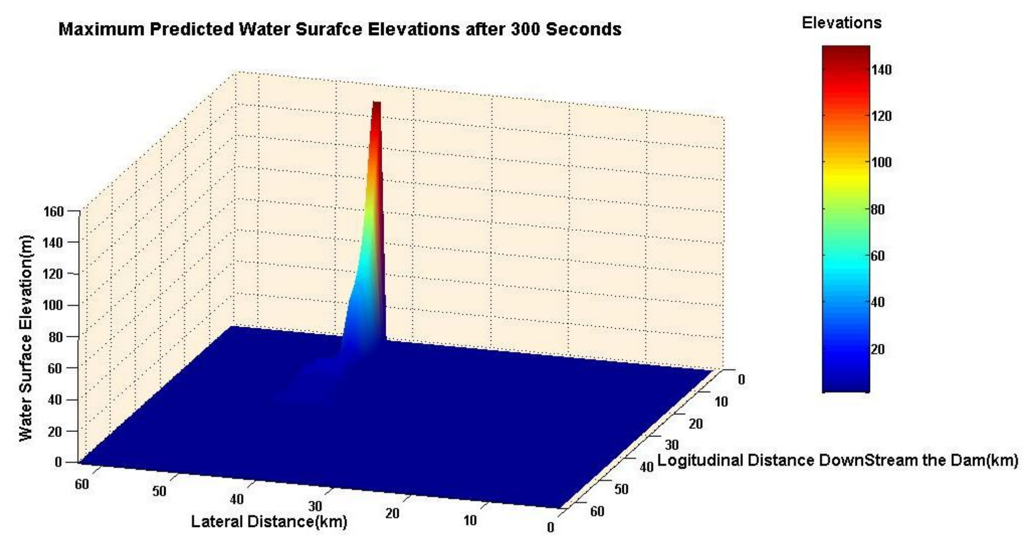

Figure9. 7: Maximum Predicted Depths by Simplified Floodplain 1D model after 300 seconds of the Dibang Dam failure-3-D view.

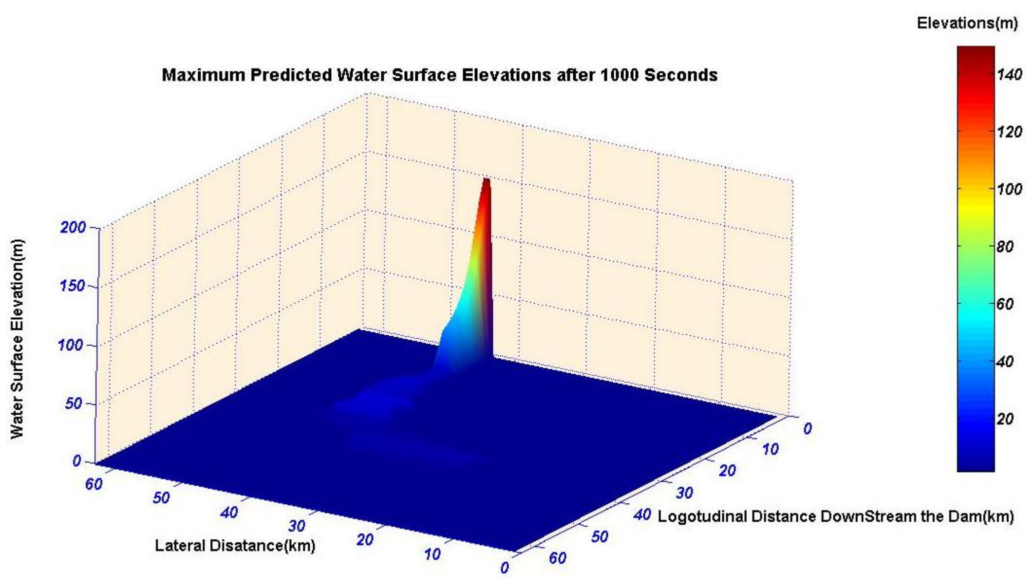

Figure9. 8: Maximum Predicted Depths by Simplified Floodplain 1D model after 1000 seconds of the Dibang Dam failure-3-D view

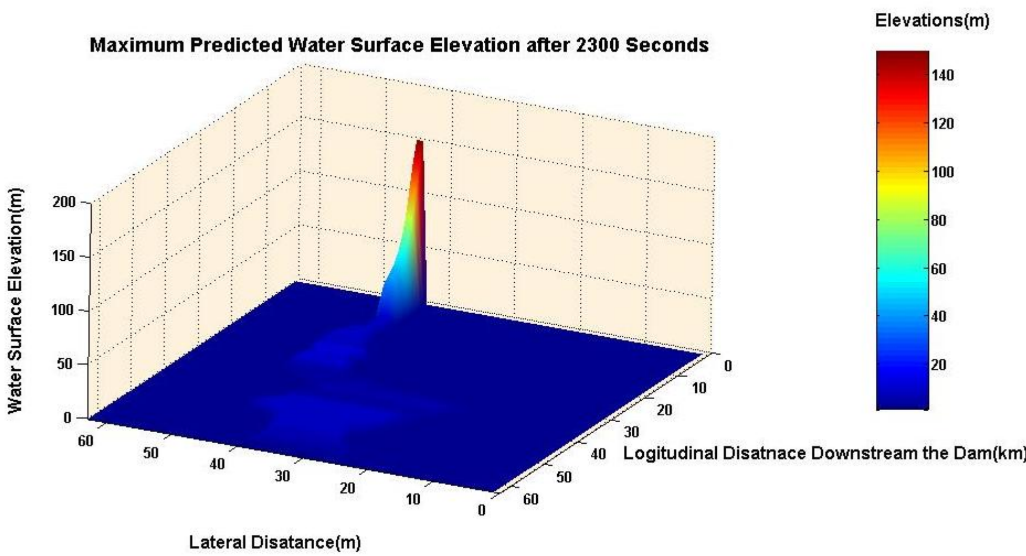

Figure9. 9: Maximum Predicted Depths by Simplified Floodplain 1D model after 2300 seconds of the Dibang Dam failure-3-D view

9.3.3. Inundation of Important Places Downstream the Dam

Some of the important palces downstream the dam comprising of villages, forests, roads etc .the maximum predicted flood depth, their time of occurrence, and maximum possible velocity of flood can be summarized in table 9.1.

Table 9.1 Predicted flood parameters downstream Dibang dam

Sl no.	Downstream distance (Km)	Villages, Forests	Maximum Flood Depth (m)	Time of Peak (seconds)	Max Velocity (m/second)
1	25	Bomjir	10.179	543.801	32.679
2	29	Deopani p forest	5.19	668.30	26.83
3	35	Nogpok	4.15	897.40	22.063
4	37	Bijari	12.466	1019.701	21.169

5	41	Kerim P forest	4.667	1123.901	19.997
6	43	Seram	4.166	1257.401	18.888
7	46	Anpum	4.527	1406.301	11.954
8	50	Namsing	3.500	1602.800	4.8330
9	50	Buraburi Deori	2.500	1602.800	4.8332
10	51	Loikhopur	1.500	1661.901	3.521
11	54	Bango	0.750	1798.501	2.437
12	56	Mer	3.654	1890.200	1.711
13	57	Amarpur	3.335	1934.200	1.039
14	60	Probha R Forest	2.201	2012.788	0.078
15	61	Puglam	0.500	2022.801	0.055

Whereas some other villages downstream (e.g., village Bolung, Parbuk, Santipur, Kukumora, Jiya Nrpali basti, Majui, Rampur, Chapakhowa, Telikola) nearby the Roing-Tinsukia road and some others nearby the Guwahati Pasighat road (village Rani, Sika Todeng,Sika Bamin, Silli, Oyang, Kemi), are not likely to be affected by the flood due the failure of the proposed dam in the river Dibang. The Guwahati Pasighat road is going to be inundated up to 41km distance downstream the dam.

9.4. Flood Damage Estimation

The evaluation of socioeconomic impacts of floods can be done using different methodological procedures. The unit loss model is based on property-by-property assessment of potential damage. Some countries like UK, Australia have established detailed methodologies for estimation of tangible losses. In Australia and many other countries, damage assessment methodologies vary in different regions within the country according to individual studies. The relationship between flood parameters and possible damage are derived based on historical flood damage information, questionnaire survey and laboratory experiences (Dutta et al 2003). One of the most detailed works has been carried out for England and Wales (Penning-Rowsell and Chattrton, 1979) for the residential and commercial services, industrial and agriculture sectors, based on the priority analysis of damages. Another more recent example carried out for Japan is the work of Dutta, Herath and Musiake (2003).

9.5. Formulation of the Flood Damage Estimation Model

To develop a relationship between the different flood parameters and possible damages associated with it, detailed historic flood damage information, questionnaire surveys etc. are required. In this study a generalized model has been developed for total damage estimation with the limited available data of flood damage of Assam.

9.5.1. Generalized Damage Estimation Model

Historical flood data available for Assam from 1953 to 2005 have been used. Statistical regression analysis has been done using least square technique of curve fitting to develop a generalize damage estimation model. It can be used in the future also in different contexts related to estimation of flood damages in the northeastern region of India

Flood damage data available for Assam as given by the Department of Revenue, Government of Assam are presented in figures 9.10, 9.11, 9.12, and 9.13.

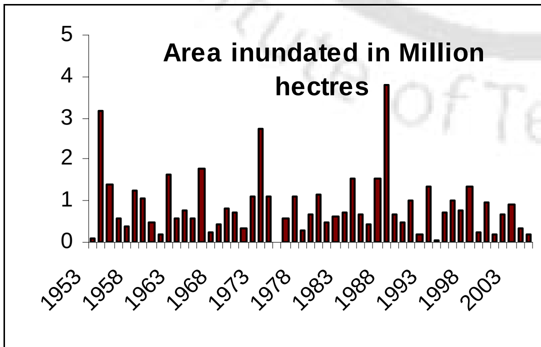

Figure 9. 10 Flood data of Assam - I (1953-2003) Area Inundated (source: Revenue department Assam)

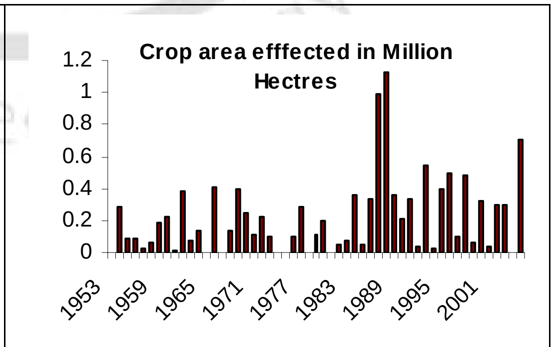

Figure 9. 11 Flood data of Assam - II (1953-2003) Crop Area Effected (source: Revenue department Assam)

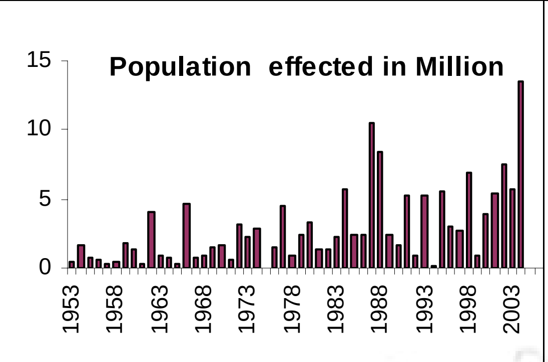

Figure 9. 12 Flood data of Assam - III (1953-2003) Population Effected (source: Revenue department Assam)

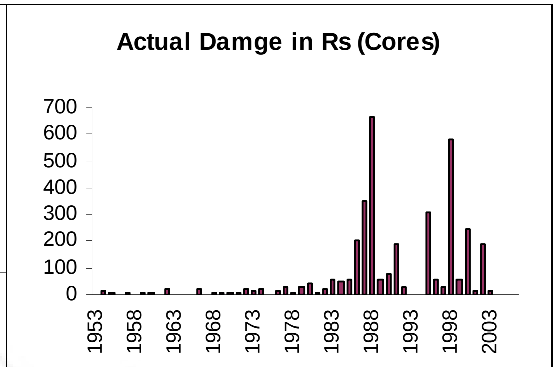

Figure 9. 13 Flood data of Assam - IV (1953-2003) Actual Damage (source: Revenue department Assam)

The total damages in actual values (Cores of Rupees) have been converted to the present value of 2006 considering the standard of economic survey of Government of India rate of 2005-2006 and presented in figure 9.14. The scatter plots of damages Vs area inundated, crop area affected and population affected have been shown in figures 9.15, 9.16, 9.17.

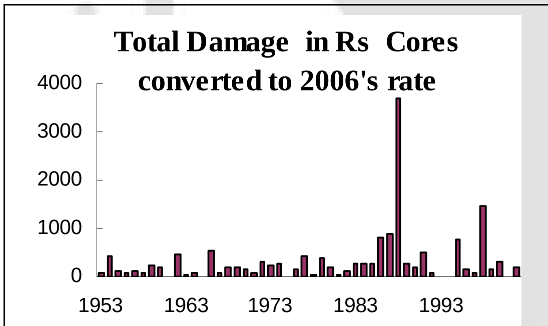

Figure 9. 14 Total Flood Damage (in 2006’s Rate)

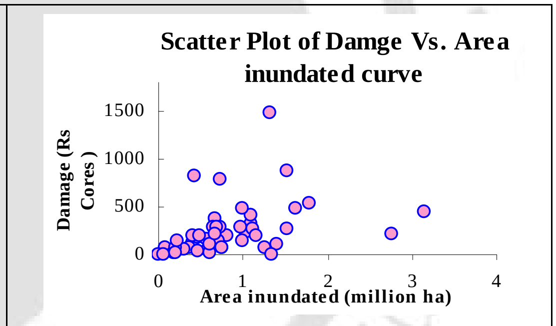

Figure 9. 15 Damages due to Flood Inundations

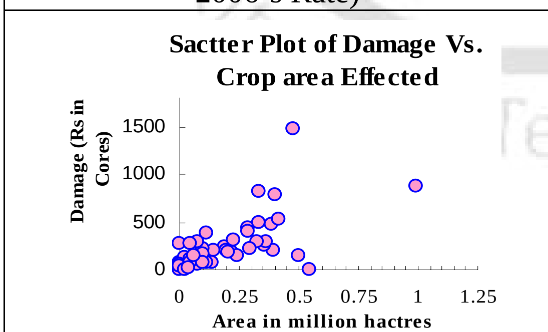

Figure 9. 16 Damages due to Crop Area Inundated by Flood

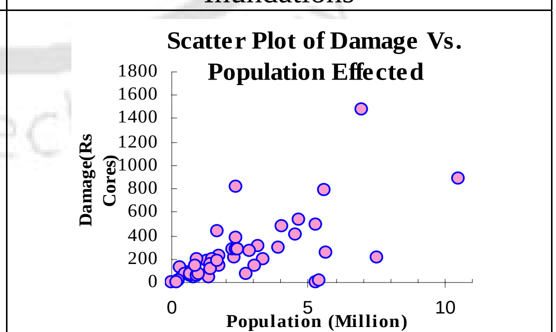

Figure 9. 17 Damage and Population Effected

The Multiple Regressions have been done for damage estimation with all the flood data available for Assam since 1953 to 2005, considering the Area inundated, Crop area affected and Population affected. Least square method of curve fitting has been used. In case of hydrologic events, coefficient of correlation value should be greater than 0.6 (Patra K C 2003). When the independent parameters are tested separately with damages then the linear correlation coefficients for area inundated, crop area affected and population affected are 0.645, 0.752 and 0.606 respectively. Hence multiple regressions have been done for linear relationships here for flood damage estimation.

The equation obtained for damage estimation is as follows:

$$\mathbf{D_{total} = 186.6 + 195.6 * A_{rea\_inun} + 150.2 * A_{crop} - 2.1 * P_{opulation\_eff}}$$

With correlation coefficient= 0.75

Where, $\mathbf{D_{total}}$= Total Damage in Cores of Rs in 2006's Rate

$\mathbf{A_{rea\_inun}}$ = Area Inundated in Million Hectares

$\mathbf{A_{crop}}$ = Crop area affected in Million Hectares

$\mathbf{P_{opulation\_eff}}$ = Population affected in Million.

For instantaneous failure of Dibang dam using the inundation map as obtained by simplified floodplain model and presented in the previous section, provides:

(i) Area Inundation, $\mathbf{A_{rea\_inun}}$ =164 000 hectares

(ii) Area Crop, $\mathbf{A_{crop}}$ = 114.8 hectares

(iii) Population affected, $\mathbf{P_{opulation\_eff}}$ = 0.0018 million

Hence, in case of instantaneous failure of Dibang dam considering the probable area of inundation, the population affected and crop area affected, using the above damage estimation regression equation, the total damage is:

$\mathbf{D_{total}}$= 391.12 Cores Rupees (as per rate of 2006).

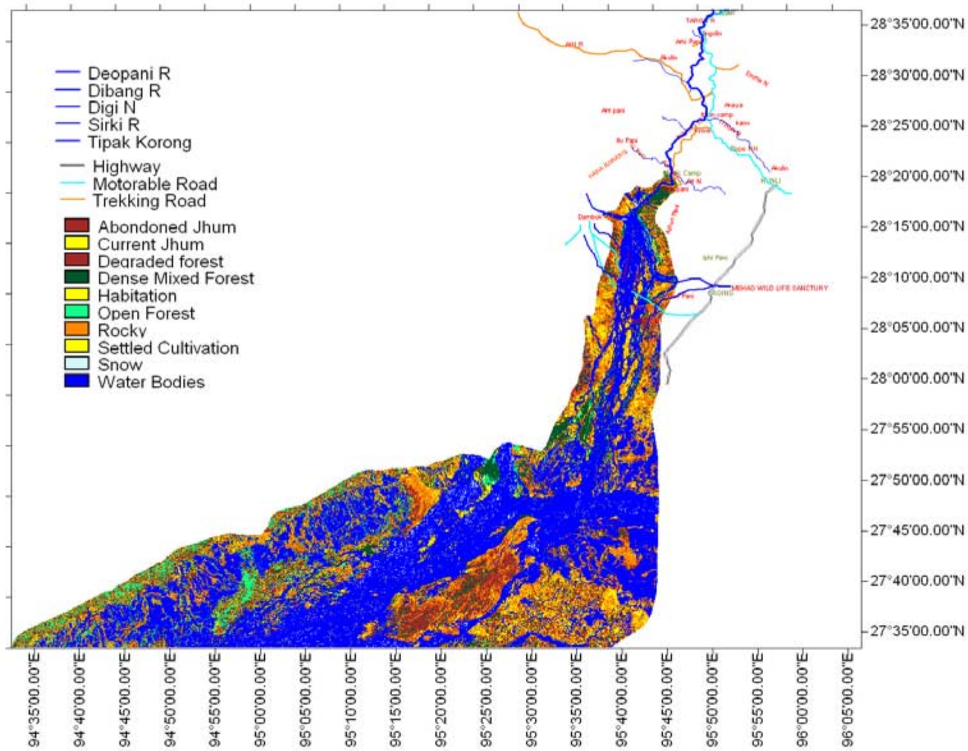

Figure 9.18: Land Use Map downstream the Dibang Dam

9.6. Necessary Disaster Management Plan Recommended

To Mitigate flood disaster of this kind, an extensive disaster mitigation planning is required. Following are some of such steps that are felt essential for the area considered in this study.

1) A disaster management cell, dedicated for this area is required to be formed involving the following:
    a. Dibang Dam Authority
    b. District administration
    c. Police
    d. Public Works Department
    e. Water Resources Department
    f. Agricultural Department
    g. Department related to Rural Development
    h. Departments related to Forestry and Wildlife

i. Local people and NGOs of that locality

2) Developing a communication network in the entire area. Telephone, wireless system and computer network may be used for transmitting information to different levels. This is essential for quick transmission of information regarding initiation of dam failure to the probable affected area. To the village area the information must be sent in a way understandable to the affected people.
3) Road network connecting the probable submerged area and the safe area need to be developed for evacuation of the people in event of disaster.
4) All responsible officials/groups/ individuals of the area need to be trained so that each of them can perform their responsibility without failure during disaster. Training programme for different levels need to be conducted. To ensure effectiveness of such training, drill should be carried out.
5) Houses in the probable submerged area need to be redesigned depending on the expected depth and velocity and should be renovated accordingly. Arrangement should be made to subsidize such renovations.
6) Farmers should be encouraged to do crop insurance.
7) It is observed that the some portion of the Passighat-Guwahati road gets submerged during dam failure. Planning should be made to elevate these portions to the required level in phase manner.

9.7. Conclusion

It may be concluded from the above analysis, that the solutions of dam-break flood models may be regarded as one of the important flood forecasting and flood hazard mitigation methods. The forecasting helps in planning the downstream area of the dam accordingly for dwelling houses, road communications, power line transmissions etc. to minimize flood hazard against the probable dam-break flood.

Damage estimation has been done based on historic data of flood damage containing historical flood data for the state of Assam, India from 1953 to 2005, containing information regarding inundated area, population affected, crop damage and total loss. However, this estimate does not include the various environmental losses.

A broad strategy towards preparation of a disaster management plan for mitigating flood disaster in the event of dam failure has been proposed.

Chapter-5

Model Validation and Its Applicability in Different Terrain Conditions

5.1. Introduction

To assess the applicability and validity of the numerical models for sub-critical, super-critical and sub/super critical mixed flow conditions, results obtained by the conservative formulation of the dam break flow with diffusive scheme have been compared with laboratory data and is presented in this chapter.

In many practical applications, the steep slopes of the natural river terrain may result in strong source terms in the system of governing equations. Hence, another aim of the study in this chapter is to test whether the numerical models developed provide stable flow profiles in any type of natural river valley. With this aim in mind, the application of developed numerical model is examined for another hypothetical dam break situation due to the failure of the proposed dam in another Indian river Kynsy.

5.2. Experimental Dam Break Hydraulics of Das (1978) and Barr and Das (1980)

Experimental data for a rectangular flume under sub-critical flow condition has been used here for comparison of the numerical results

5.2.1 Details of Das (1978) Experiment

The experiment was conducted in the Hydraulics, Hydrology and Coastal Dynamics Laboratory of the University of Strathclyde, Glasgow. Two rectangular channels had been adopted for experiments. The experimental data for bigger rectangular flume made of perspex are compared for sub-critical conditions. The flume was 110 feet long, 5 feet wide and 1.5 feet deep. The reservoir upstream of the gate is 25 feet long. The depths of flow at different channel sections have been measured at different time intervals for different upstream and downstream depths. In this study the flow profile for original upstream depth of 0.5 feet after 22seconds of dam break/gate removal has been considered for comparison with the computed results. Data collected at eight observation stations (at x= -24feet, -18feet, -11feet, -0.5 feet, 13.5 feet, 33.5feet, 56.5feet ,71.5feet) are obtained from the above author for the purpose of comparison . The experimental setup of the small flume with wire mesh roughness and comparison of the computed flow profile obtained by conservative formulation of the diffusive scheme with the experimental values of the bigger channel are presented in figure 5.1 and figure 5.2 respectively.

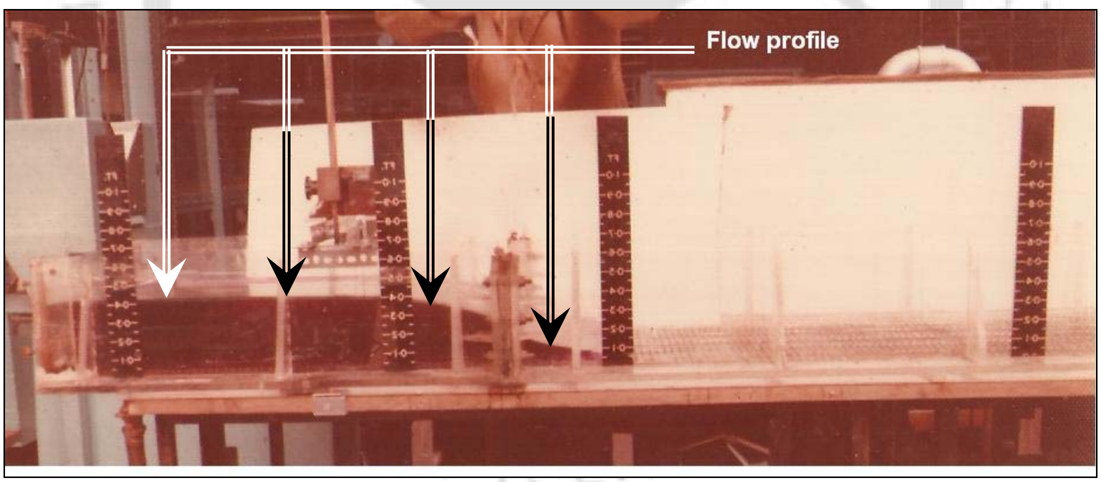

Figure5. 1: Flow Profile in the Flume just after the Removal of the Gate in Dam Break Experimental Setup of Barr and Das

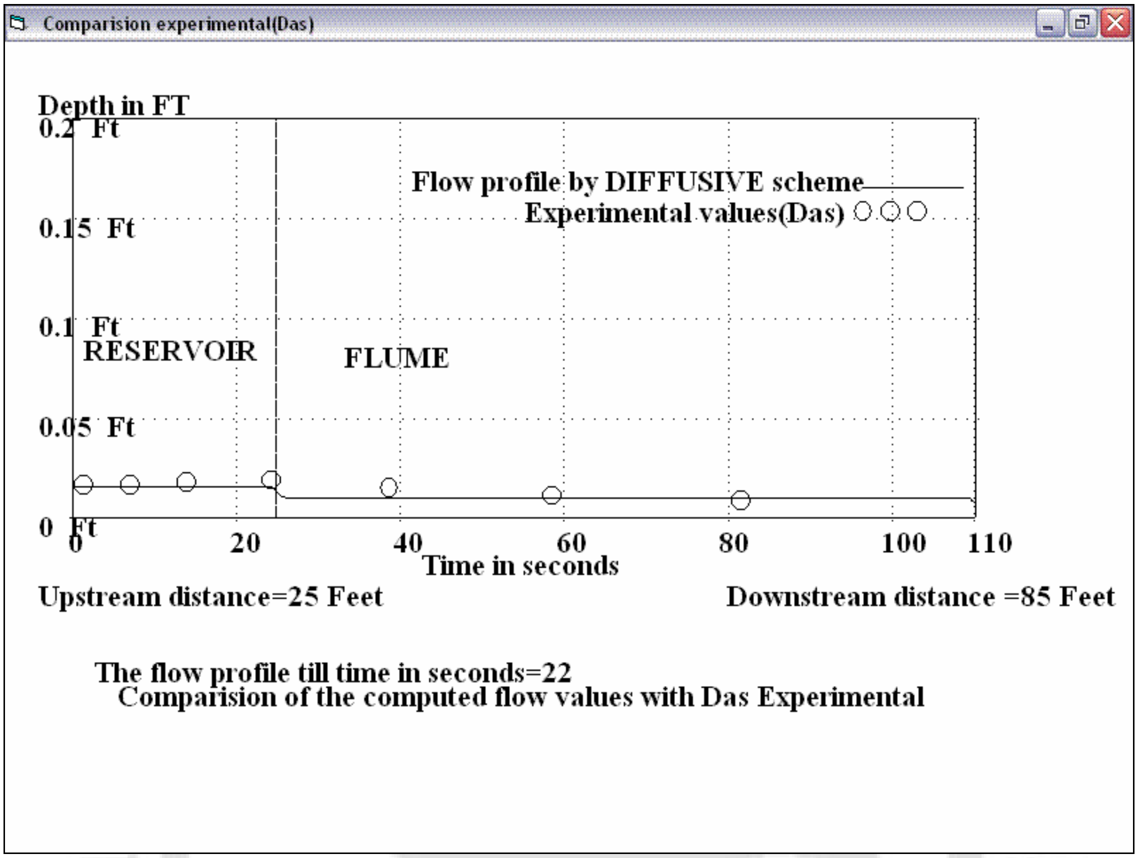

Figure5. 2: Comparisons of the Flow Profile Computed by "Diffusive scheme" with the Experimental Values of Das

5.3. Experimental Dam Break Hydraulics Dam Break Hydraulics of Bellos (1990) and Bellos et al (1992)

Experimental data for rectangular flume with varying width under the mixed (sub/super) flow condition with wet bed downstream have been used here for comparison of the numerical results.

5.3.1 Details of Bellos(1990) Experiment

The flume used by Bellos(1990) is shown in figure 5.3 .The flume was made of steel and glass ,with a gate fixed at the section of minimum width. The dam failure was simulated by opening the gate as quickly as possible, i.e., within 0.1 second. The water levels are monitored at eight different observation stations along the centre line of the channel. Data collected at five of these observation stations (x= 0.0m, 4.5m, 8.5m, 13.5m,18.5m) are obtained from the above author for the purpose of comparison of the proposed model. The experiments were conducted, with slopes varying from 0 to 10 % for dry downstream. Experiments were also conducted in horizontal bed where water levels were 0.053m and 0.101m downstream of the gate. The water level downstream of the gate was controlled by a variable height weir at the downstream end of the flume. Two data sets are taken here for comparisons.

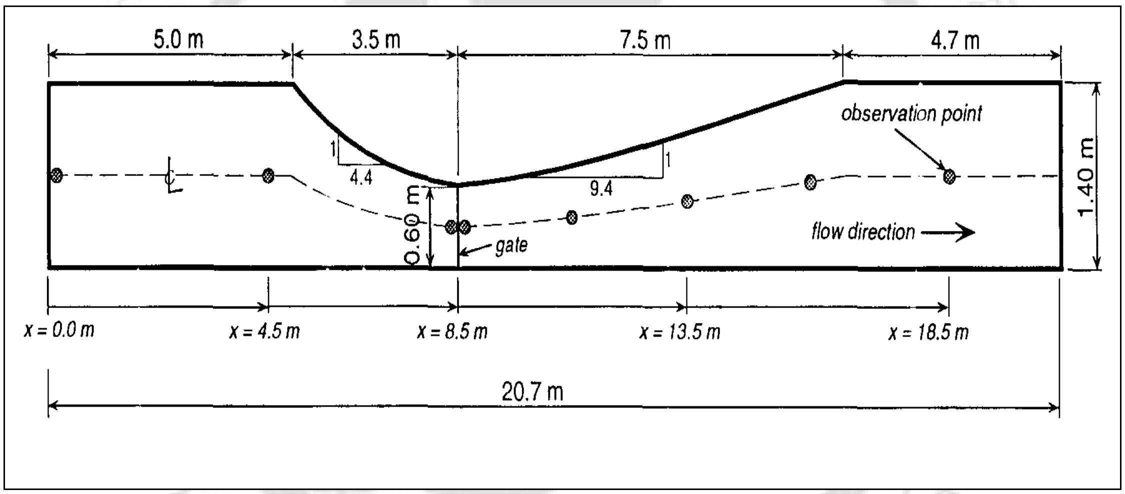

Figure5. 3: Plan of Bellos's Experimental Channel

(i) The data are taken for the flow resulting from the instantaneous gate opening where the upstream to downstream depth ratio was "3" and the flow was under supercritical condition The data collected in three observations stations at x= 4.5m, 8.5m, 13.5m, are taken here for comparison. Figure 5.4 shows the comparison of the computed flow depths by diffusive scheme with Bellos experimental data. Even though the profiles obtained are slightly diffusive in nature they are well comparable with the experimental data. At x=13.5 m ,after 18 seconds there are some differences with the numerical values which may be due to a wave reflected off of the weir at the downstream end.

(ii) Another data set of Bellos experiments has also been taken in this study. This data set was also used by Hicks et al (1997) in their study. They tested for capability of Characteristic Dissipative Galerkin (CDG) finite element scheme and Box finite difference (BFD) scheme for computing dam break flow when supercritical flow occurs within a channel reach. It has been quite interesting to observe that under this complex flow conditions where the BFD scheme with non-conservative formulation, was found to fail (Hicks et al, 1997) to give a solution after 8 seconds, the conservative formulation of the diffusive scheme results smooth solutions comparable to the experimental values. The numerical values have shown discrepancies at the dam site for the fist few seconds, which was also observed in Hicks et al (1997) finite element solutions. At x=13.5m, after 18 seconds there is some minor differences between the experimental and numerical values. Similar differences were also found in the finite element solutions of Hicks. This may be due to a wave reflected off the weir at the downstream end of the experimental set up that has not been considered in the numerical analysis. From the observations of the comparisons of this experimental test case, it is obvious that the formulation of the governing equations plays an important role in success of a mathematical model.

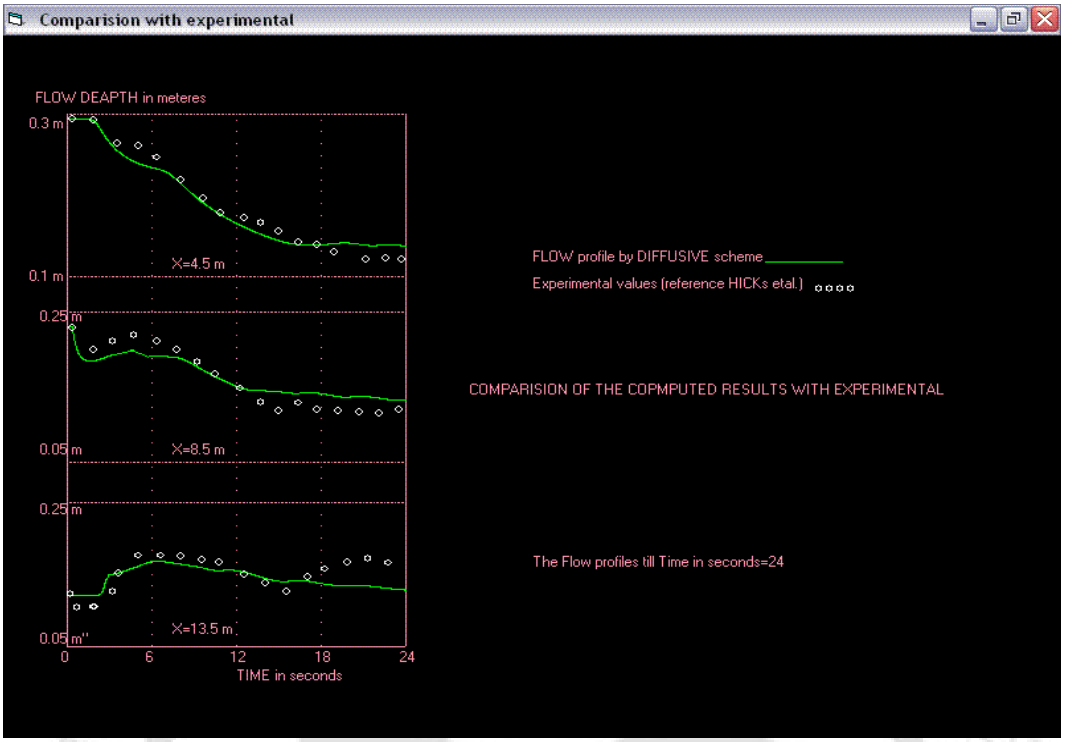

Figure 5. 4: Comparisons of the Flow Depth Hydrographs Computed by "Diffusive scheme" with the Experimental Values of Bellos (Case I)

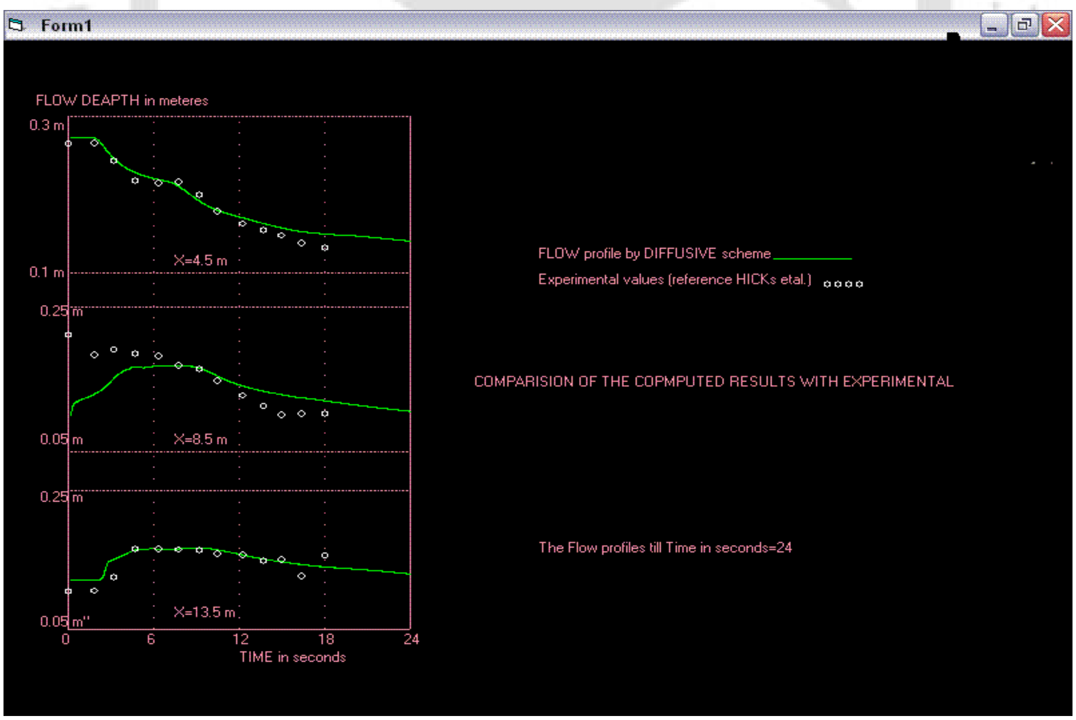

Figure 5. 5: Comparisons of the Flow Depth Hydrographs Computed by "Diffusive scheme" with the Experimental Values of Bellos (Case II)

5.4. Assessing Robustness of the Developed Models through its Application in another Real River Channel

The river Kynsy originates from Sohiang in west Khasi hill district of Meghalaya. It flows southward and drains into Bangladesh. It has a narrow channel with very steep bed slopes.

Two river channels with significant difference in channel topographic characteristics have been considered here. The resultant source terms in both the cases will be reasonably different, so the aim here is to test that whether the numerical model developed here also provides stable flow profiles in any type of natural river terrain. The river Dibang has a wide floodplain from 11 km downstream of the dam up to the confluence with Brahmaputra with gentle bed slope, while river Kynsy has a narrow channel with very steep bed slopes

5.5. Salient Features of Kynsy Dam

(i) **Height of proposed dam:** 117 m

(ii) **Elevation of River Bed:** Varies from 973.00 m to 20.20m.

(iii) **Channel distance considered down stream of the dam for computations:** 53,000m

(iv) **Extension of the reservoir**: 7,000 m

(v) **Channel roughness: (**Manning's coefficients) varies from values 0.03- 0.035

(vi) **Breadth of the channel**: varies from 85m –335m

5.6. Flow Profiles of Kynsy Dam Failure

The first order conservative diffusive scheme, which has been found to provide acceptable solution with least effort in chapter 4, has been taken for computating flood movement due to the hypothetical failure of Kynsy dam. Smooth stable flow profiles (figure 5.6) are observed for Kynsy dam failure also as obtained in Dibang Dam failure.

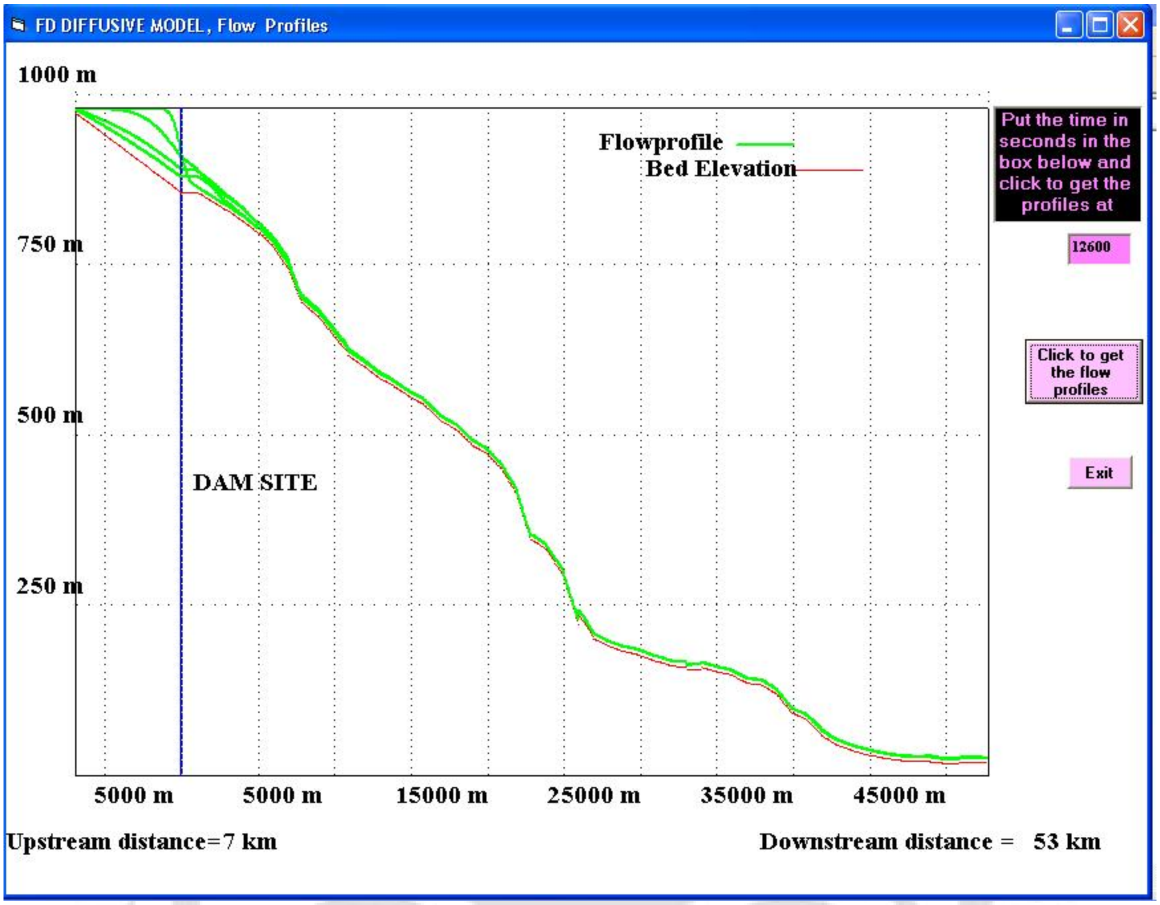

Figure5. 6: Flow Profiles in the Natural Channel of River Kynsy, after, 60, 1000, 6000, 126000 seconds of the Dam Failure

5.7. Source Terms in the River Channels

The variations of the bed slope “S_0”; and friction slope “S_f” and the resultant source term i.e., S= g A (S_b- S_f) with respect to time along the longitudinal section of the real river channels are plotted.

5.7.1 Dibang Dam Failure

In case of river Dibang the bed slope ranges from 0.2% to 0.5%. As the flow changes to high supercritical just downstream the dam site remarkable variations in friction slope have been observed. The variations in the total source term in case of Dibang are hence also significant just downstream the dam as shown in figure 5.7.

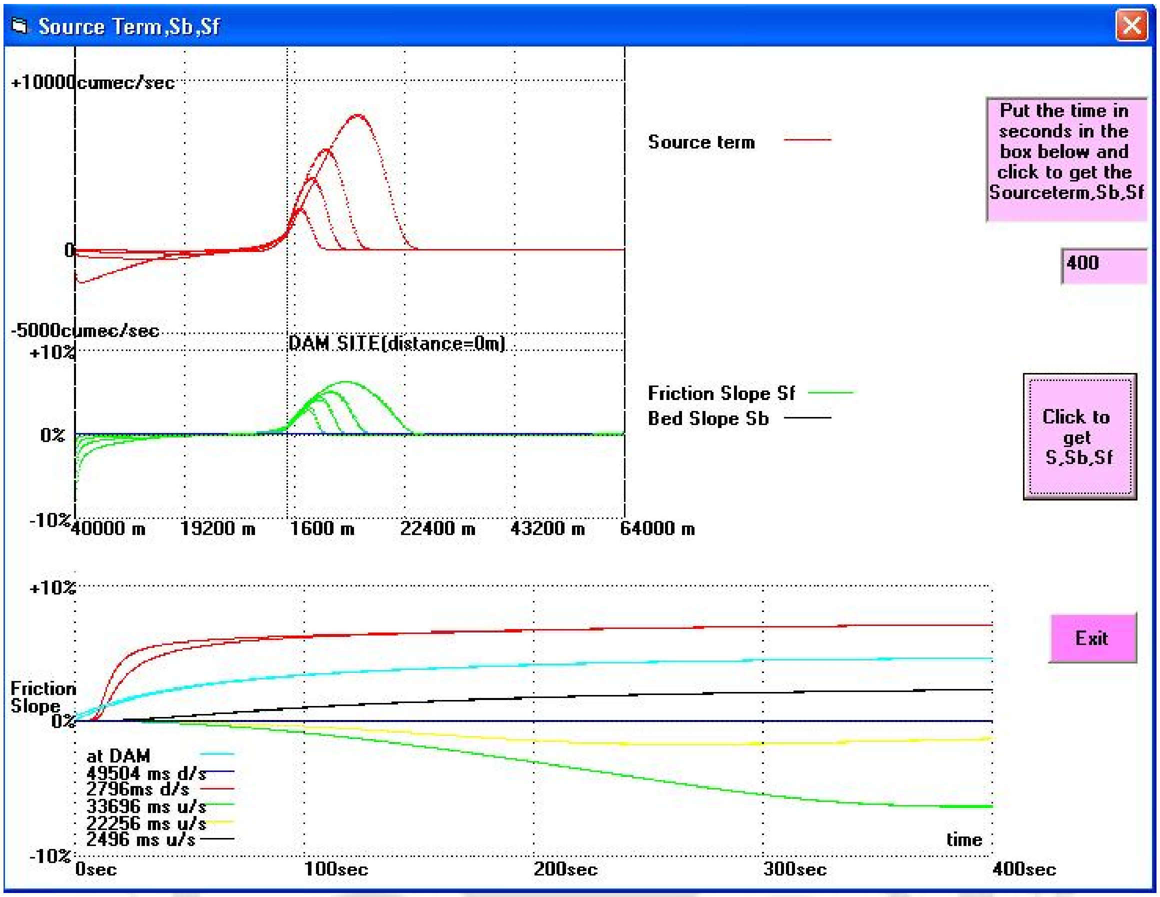

Figure5. 7: Plot of Source Term, Bed Slopes, and Friction Slopes in the Channel of River Dibang

5.7.2 Kynsy Dam Failure

The natural terrain of the River channel of Kynsy is quite complex. While In some reaches there are very steep bed slopes having S_b value as high as 5.15%, bed is completely horizontal in some other sections.

In the upstream reach of the river channel significant variation in the source term(S) is observed in figure 5.8, where the values of "S" are plotted after 2,40,1000,6000 and 12600 seconds respectively, of the Kynsy dam failure, along the longitudinaldirection.

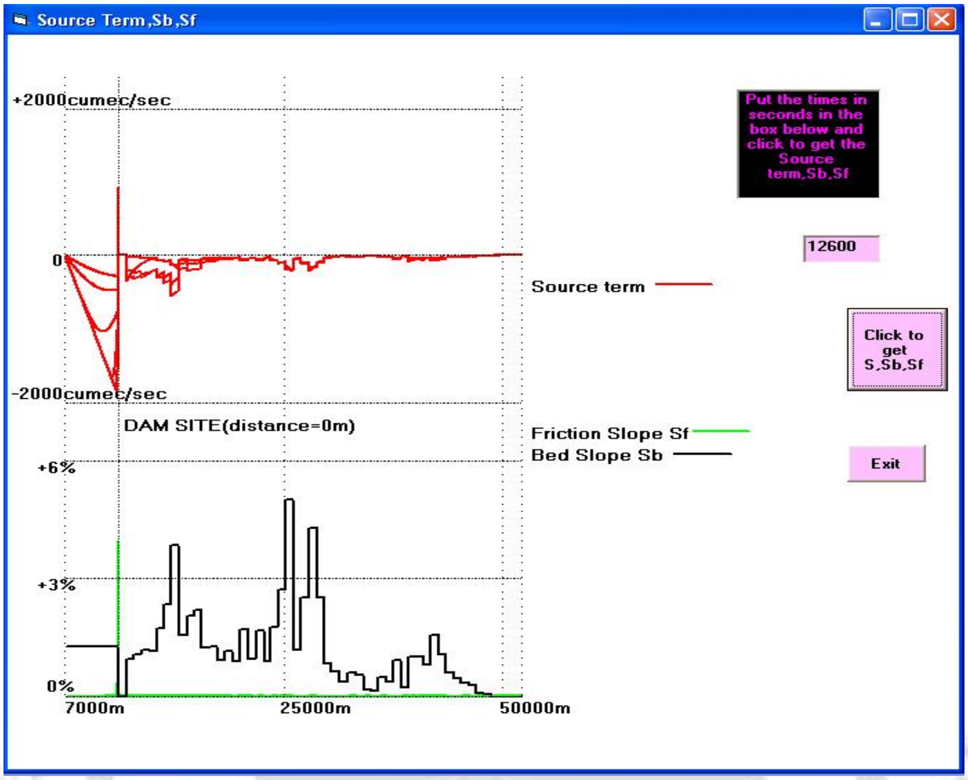

Figure5. 8: Plot of Source Term, Bed Slopes, and Friction Slopes in the Channel of River Kinsy

5.8. Conclusion

The diffusive scheme, the simplest among the finite difference schemes with conservative form, has been used to assess the validity of the model developed in this study. The model has been found to be quite capable of handling sub-critical, super-critical and mixed flow conditions.

Analysis of the applicability of the developed model in different terrain conditions has clearly revealed that the conservative formulation with the Diffusive scheme provides stable profiles in the natural terrains of both the rivers, one in steep mountainous valley another in a comparatively flat terrain with a channel in a wide floodplain. Smooth logical profile could be computed in both the channels though considerable variation in the source term with respect to space and time has been observed. Hence, considering capability and advantage of using FD Diffusive scheme, numerical formulations of 1-D simulation have been done with diffusive scheme in the subsequent chapters.

Chapter-10

Conclusions, General Discussion and Recommendations for Further Studies

10.1 Introduction

Although a brief conclusion has already been presented in each of the previous chapters, a comprehensive overall discussion and general conclusion on all the works performed under this study have been given in this chapter. The literature review reveals that some works have been done for simulating dam break flow in natural floodplain topography, yet there exist quite a lot of avenues for extension of author's work. Therefore, some guidelines for extension of author's work have also been laid down in this concluding chapter.

10.2 General Discussions and Conclusions

10.2.1 Review of literature has revealed that dam break problem remains a topic of continued interest since 1892 which started with simple case such as rectangular frictionless channels, till date for mathematical simulation of a real dam break flood with natural complex channels and floodplains. It appears that large number of numerical solutions both in 1-D and 2-D models have been developed. But little work has been done on the following points:

(i) The application of the developed models in natural floodplain topography.

(ii) Analysis of performance of the numerical schemes in different real river floodplain topographies.

(iii) Development of dam break models for simulating the flood in complex river valley and to select the most suitable one considering the complex ground terrain conditions.

Hence, in this study hypothetical flood due to the proposed large dam in the Himalayan River Dibang has been considered and a study has been made in the above-mentioned points.

10.2.2 Gradually varied flow equations for open channel i.e., continuity and momentum equations both in conservative and non-conservative forms have been used for 1-D formulations. 2-D formulation has been done with 2-D continuity and momentum equations in conservative form.

10.2.3 The performances of first order Diffusive finite difference (F.D.) scheme, second order modified two-step Predictor Corrector F.D. scheme and MacCormack scheme with Total Variation Diminishing (TVD) limiters have been used here for computing flood movement due to failure of the proposed Dibang dam. The 2-D model has been developed using two-step modified predictor corrector scheme.

10.2.4 It has been observed that the FD diffusive scheme formulated with non-conservative form of the governing equations fails to provide solutions after 22 sec. The two-step modified predictor correction scheme with non-conservative formulation has produced oscillatory profiles. Therefore, the analysis clearly demonstrates the merit of using conservative form of the governing equations for computing dam-break flow in a highly non-prismatic natural channel.

10.2.5 The significance and need of using numerical refinement by applying corrections such as “TVD” limiters in real field situations are investigated. The use of TVD to the second order scheme has made the implementation complex. Runtime is also burdensome even in one-dimensional calculation for the real field situation. Moreover, the refinement achieved in the flow profile by applying TVD correction may not be quite significant in a natural channel, as there always remains an obvious error in the input terrain data.

Average values for different input parameters such as bed slope, channel width, and channel roughness are generally taken between two successive grid points. The application of the presented simple FD schemes i.e., first order diffusive and second order modified predictor-corrector, is quite justified as observed from the computed profiles in real riverbed topography.

10.2.6 . To assess applicability and validity of the numerical models for sub-critical, super-critical and sub/super critical mixed flow conditions, results obtained by the conservative formulation of the dam break flow with diffusive scheme have been compared with laboratory data. For subcritical flow comparisons experimental data of Das (1978), Barr and Das (1980) and for mixed and supercritical flow conditions, Bellos (1990) and Bellos, Saulis and Sakkas (1992) have been considered. The model has been found to be quite capable of handling sub-critical, super-critical and mixed flow conditions.

10.2.7 In many practical applications, the steep slopes of the natural river terrain may result strong source terms in the system of governing equations. Hence, to test whether the numerical models developed provide stable flow profiles in any type of Natural River valley, the developed numerical model is examined for another dam break situation due to failure of the proposed dam in another Indian River Kynsy. Considerable variations are there in the source term in different river sections in a particular instant of time as well as in the same section in different time periods. The conservative formulation with the Diffusive scheme provides stable profiles in spite of the consideration of the natural terrain of both the rivers, one in steep mountainous valley another in a comparatively flat terrain with a channel in a wide floodplain.

10.2.8 The simulated output depends a lot on the terrain or channel assumptions. Therefore, to get insight into the effect of channel characteristic on the flood prediction three 1-D models and one 2-D model have been formulated considering the natural ground terrain of Dibang. Comparison of the results obtained with the three models will enable to assess the performance of the proposed models in complex natural floodplain topography. 1-D models are The River channel 1-D model (Computational channel considering the River Channel of Dibang), Simplified floodplain 1-D model (Computational channel considering the floodplain downstream of the dam) and Composite Channel 1-D Model: (Computational channel with compound channel section).

10.2.9 The graphical user interface (GUI) software has been developed in Visual Basic. The visual basic has been used as programming language as it has enabled the author to get graphical output in desired form without any additional graphical package. Field engineers or planners can use the developed software without much knowledge of computer programming.

10.2.10 It has been observed that 1-D river channel model over predicts the maximum flood depth, velocity and under predicts the time of peak arrival at the downstream sections when the river enters the plain compared to the computations by simplified floodplain 1-D model channel in case of an instantaneous failure of such a high dam.

10.2.11 Once the depth of flow crosses the depth of the original river channel, it will start flowing to the nearby floodplain. The proper prediction of the possible extend of inundation downstream of the dam is quite important, as it consists of villages, roads, dense forest etc. and therefore, for such practical purposes, for proper prediction of the dam break flood, it may be quite realistic and logical to select the computational channel in such a manner that it takes into account the wide floodplain when the river enters the floodplain.

10.2.12 The maximum probable depths are estimated slightly more in case of the simplified floodplain 1-D model while comparing with composite channel. The times of peak arrivals are more in case of the composite channel as equivalent Manning's "n" is taken as surface resistance considering the details of downstream land use. It is interesting to note that the estimations of the above two parameters given by the model with simplified floodplain 1-D channel sections are in safer side from flood prediction, early warning and managerial points of view. The velocity in the composite channel reduces due to the high surface resistance of the flood plain.

10.2.13 Additional work required to undertake in 2-D modelling are the data preparations for the input. In case of Dibang Dam, 1-D model has 1001 computational points with 1000 intervals of 106 m, each whereas the 2-D model has total 6, 01,601 computational points with 1000 x 600 intervals of 106m each. The flood levels by the 1-D model are predicted slightly higher than that by the 2-D model in most of the ground point. In the 2-D model the speed of the flood wave is slow. Hence, the time of peak wave arrivals in case of the 2-D model are found to be noticeably overestimated compared to the 1-D model computations.

10.2.14 The majority of the studies have been carried out in laboratories where channel bed is assumed to be fixed under such a highly transient flow. Dam-break hydraulics of natural rivers is indeed complicated, and involves not only water flow, but also morphological change in the riverbed, which in turn modifies the predicted flow profiles. Although recently very few numerical models are proposed for mobile beds also, applications were only in simple laboratory channels.

10.2.15 The mass and momentum conservation equations for the water-sediment mixture and the mass conservation equation for sediment are the governing equations. Different investigators tried different solutions of dam break flow on movable bed, some coupled together the three equations and some tried it by solving the flow and sediment equations separately termed as decoupled solutions. Here both coupled and decoupled models have been tried for Dibang dam failure.

10.2.16 There is a considerable change in the riverbed at the dam site under the highly transient flow condition occurring due to the failure of a large dam. Consideration of bed mobility in the dam break flow model modifies the free surface flow profiles and the hydrographs. The rate of bed deformation signifies the need for the coupled modeling of the strongly interacting flow-sediment-morphology system under dam break flow. The decoupled model which was illustrated earlier to provide reasonable solutions cannot at all be recommended for large dam break with high transient flows and bed deformations. In case of Dibang dam failure the decoupled model fails to provide solutions after 20 seconds.

10.2.17 The assessment of the simplified 1-D model developed for dam break flood prediction has been done by using this model to simulate real world dam break benchmark problem. Simulations have been done for Malpasset dam break (1959 France) and computed data are compared with the real dam break peak depths and data available from the physical model of Malpapsset dam break maximum flood predicted and the actual dam break flood level has shown that computed values are comparable with estimation on higher side except in three ground points. Incase of comparison of the computed values with the physical model test case the predictions of the flood depth are slightly higher and peak arrival times are lower in simplified floodplain 1-D model.

10.2.18 Hence the simplified floodplain 1-D model can be suitably used for computations of dam break flood in complex natural floodplain topography. It slightly overestimates depths and underestimates times of wave arrival, which are on safer side from managerial point of view. Moreover, the model is quite easy to implement as compared to the composite channel sections model or the 2-D model which require significant effort in preparing database of the channel cross-sections, ground elevations etc. The simplified floodplain 1-D model of the Dam-break flood has been used and the important flood parameters i.e., maximum flood depths, maximum flow discharges maximum flow velocities , time of the maximum flood depth arrivals and an inundation map depicting the aerial extend of flooding have been forecasted.

10.2.19 In this study available historical flood data for the state of Assam, India from 1953 to 2005, containing information regarding inundated area, population affected, crop damage and total loss, have been used. The total damages in actual (Rupees. in Cores) have been converted to the value of 2006 considering the standard rate of economic survey of Government of India 2005-2006. The multiple regressions analysis has been done to develop a generalize damage estimation model, suitable for flood damages estimation in the northeastern region of India. In case of instantaneous failure of Dibang Dam considering the predicted area of inundation, the population affected and crop area affected the estimated damage has been found to be 391.12 (Rupees in Cores as per rate of 2006). However, this estimate does not include the various environmental losses. Considering the high value of estimated damage several mitigation measures have been suggested. Due consideration to various practical constraints such as existing land use, education level of the affected people, available technology etc., has been given while suggesting the disaster mitigation measures in the event of dam failure.

10.2.20 Different important observations have been made from the presented study can be concluded as below:

(i) The Governing mathematical equations must be in conservative forms for complex natural channels to compute stable smooth flow profiles.

(ii) Simple numerical schemes should be used compared to higher order schemes for such complex real situations for computational efficiency. The slight refinement obtained in the computed results is insignificant compared to interpolations made for the terrain.

(iii) Huge dam break flood flow can not be confined in the river channel only but it inundates the floodplain also. Therefore, the selection of computational channel is to be considered very carefully and the floodplain also has to be considered.

(iv) Consideration of mobile bed in the dam break flow model modifies the free surface flow profiles and the hydrographs. The mobile bed can be significantly scoured in the initial stage; the dimensions of the scour hole are of quite comparable magnitudes to those of the flow itself.

(v) Comparisons of model results obtained with *composite channel consideration* and *simplified flood plain consideration* have shown that the flow depth and the velocity are slightly overestimated and the time of peak arrival is underestimated in *simplified flood plain* as compared to the *composite channel*.

(vi) In the development of 2-D numerical model for Dibang dam-break, additional effort is required for input data preparation. The computational time required is also significantly higher in the 2-D model than that of the 1-D model.

(vii) The flood levels in the 1-D model are predicted higher than that in 2-D. The speed of wave is computed low in 2-D model.

(viii) In the comparison of the numerical model with laboratory data, it has been quite interesting to observe that under the complex flow conditions where the Box Finite Difference scheme with non-conservative formulation, was reported to fail (Hicks et al, 1997) to give a solution after 8 seconds, the conservative formulation of the diffusive scheme results smooth solutions comparable to the experimental values.

(ix) Damage estimation from the failure of Dibang dam has been made by developing a general flood damage estimation relationship by method of multiple regressions from the flood data available for Assam since 1953 to 2005. Mitigation measures considering existing land use and various other practical constraints have been suggested.

10.3 Recommendations for Future Work

Study of dam break flood had been made in the past by different investigators as discussed in chapter -2. But the extensive literature review reveals that very little work has been done for simulating dam break flood in complex real river valley. During this study, it has therefore, been felt that a lot of avenues are still there for further works in this line.

10.3.1 Further Works on Numerical Modeling

In the present study the performance of the finite difference schemes have been examined in complex real river valley. Other Numerical techniques like Finite Volume Method, Finite Element Method and Implicit schemes of Finite Difference Method can be used for simulating dam break flood in the same situation.

10.3.2 Further Works on Mode of Failure

The Dibang dam which is taken under this study is a large gravity concrete dam. Hence instantaneous failure has been considered here for simulation to consider the extreme devastating flood. However, in case of earth and rock fill dams the mode of failure or breach may be gradual. Hence, a comprehensive model has been suggested for future study incorporating mode of failure based on materials of dam and historical data (Singh 1996) available for breach

10.3.3 Further Works on Simulating Dam Break Flow in Mobile Channel Bed

The literature review also shows that quite a volume of work on the dam-break flood model analysis at this moment on a fixed bed has already been done. In the real world dam break flood occurs in mobile boundary channel but work on this mobile bed till now appears to be not sufficient. The author has developed a numerical model on this mobile bed of the river Dibang along with the main work. Coupled flow and sediment equations are formulated here for simulating the flow parameters and bed profiles in a natural river due to the failure of a large dam incorporating morphological changes of the riverbed. However, minor numerical difference may be there in the simulated profiles with the real one, as it is difficult to pinpoint roughness parameter for natural rivers, especially when sediment transport is involved and formulations for the sediment entrainment and deposition fluxes are empirical. Moreover, the different empirical formulae as available in literature have been used here to start a study for analysis of movement of the flow due the failure of a large dam in mobile bed and bank conditions. No modified forms of theses empirical formulae are available to use it in dam break case. Modification of those formulae is an obvious requirement as flows are highly transient and further extensive study appears to be essential. It is felt from this study that a wide scope exists to explore dam-break study in real mobile bed channel and therefore, suggestion has been made to explore dam-break flood analysis in this field.

10.3.4 Further Works on Compound Channel flow Interactions

Both channel roughness and the depth of the flow, and hence flow conditions are different in the main channel and in the flood plains. In general, floodplain flows are of relatively shallow depth with slow-moving flow adjacent to faster-moving flow in the main channel and hence transfer of momentum between the main-channel and the floodplain is essential. Although different investigators carried out study to shed light on this interaction of the flow conditions, most of their studies are confined to simple straight uniform compound channels in laboratories e.g. (Prooijen et al 2005). They are yet to be confirmed in real field cases where channel may comprise of curvature, roughened floodplains and steep slopes at the interfaces. Hence, suggestion has been made for further studies for transfer of momentum between the main-channel and the floodplain in real river valleys.

10.3.5 Further Work on Transport of Breached Dam Materials

Total failure of earth or rock fill dam of considerable length and height involves the transport of huge sediment of dam materials along with sediment due to the erosion of bed and sides of mobile channel by high velocity. A model study of dam-break flood analysis considering all these bed and suspended loads seems to be an encouraging, interesting and challenging future study. This study may predict the modification of flood movement, its depth, inundation along with transportation, erosion and deposition of sediment and also aggradations and degradation of river bed.

10.3.6 Further work on Unsteady Channel Resistance

As described in section 6.2.6.1 unsteady variations of Manning "n" are not considered in this study. However, in the downstream in the floodplain when depth will be less the resistance will be more. Hence, future investigators of on this type of real field dam-break analysis may consider the different unsteady resistance effect to refine their models.

References

Akanbi, A. A., and Katopodes, N. D. (1988) "Model for flood propagation on initially dry land" *J. Hydraul. Engineering, ASCE*, 114(7), 689-706.

Ahrens, J. P. (2000), "The fall-velocity equation", *J. Water, Port, Coastal, Ocean Eng.*, 126(2), 99–102

Aureli, F., Mignosa, P., and Tomirotti, M. (2000) ''Numerical simulation and experimental verification of Dam-Break flows with shocks'' *J. Hydraul. Res*., 38(3), 197–206.

Aureli F. and Mignosa P. (2004) "Flooding scenarios due to levee breaking in the Po river " *J. Water Management" (ICE) ,WA* 157(1).

Barr D.I.H. and Das M.M. (1980) "Numerical simulation of dam burst and reflections with verification against laboratory data" *Proc. of the Institution of Civil Engineers,* Part 2, 69, 359-373.

Barr D. I. H. and Das M. M. (1981) "Simulation of surges after removal of separating barrier between shallower and deeper bodies of water" *Proc. of the Institution of Civil Engineers,* Part2, 71,911-919.

Bellos, C.V. and Stakkas J.G. (1987) "1D Dam break Flood Wave propagation on dry bed" *J. Hydraul. Engg.* ,113(2),1510

Bellos C.V. (1990) “Computation of 2D water movement in Open channels” PhD thesis, Democritus University of Thrace (in Greek).

Bellos C. V., Soulis J. V., and Sakkas J. G. (1992) "Experimental investigation of two dimensional dam-break induced flows" *J. of Hydraul. Research,* 30(1), 47-63.

Biswas, A.K. (1971) “Dam Disasters an assessment” *Eng journal Canada*,54(3),3-8.3,321-330.

Brown, P. P., and Lawler, D. F. (2003), “Sphere drag and settling velocity revisited ” *J. Environ.Eng.,* 129(3), 222–231.

Chow V.T. (1959) *Open Channel Hydraulic,* McGraw Hill, New York.

Craya, A. (1946) “Calcul Graphique Regime Variable dans les Canaux, la Houllie Balanche *’’New Ser. No. 1,Nov ’45- Jan’46*,pp 19-38.

Colby, B. R. (1963) “Discussion of ‘Sediment transportation mechanics: Introduction and properties” *J. Hydraul. Div.,* 89(1), 266–268.

Chung, J.A. (1980) “Practical Aspects of computational River Hydraulics” Ploman, London.

Cunge, J. A., Holly, F. M. Jr, and Verwey, A. (1980) “Practical aspects of computational river hydraulics” Pitman Advanced Publishing Program, Boston.

Chaudhry, M.H. (1993) .*Open-channel flow.* Prentice-Hall. India Pvt.Ltd. M-97, New-Delhi.

Cheng, N. S. (1997) "Simplified settling velocity formula for sediment particle", *J. Hydraul. Engg.*

Caleffi V., Valiani A., Zanni, A (2002) "Finite volume method for simulating extreme flood events in natural channels Simulation des évènements hydrologiques extrêmes dans les cours d'eau naturels'

Calenda G., Calvani L., Mancini C. P., Macchione F. and Morelli A.M. (2003) "Practical Aspects in Comparing Shock-Capturing Schemes for Dam Break Problems" *J. Hydraul. Engg*, 129(3),187-195

Cao Z; Pender,G; Wallis, S; and Carling,P (2004) "Computational Dam-Break Hydraulics over Erodible Sediment Bed", *J. Hydraul. Engg*, 130(7),689-703

CADAM (1998) Proc. ,1st CADAM workshop, Wallingford, March 1998.
CADAM (1998) Proc., 2nd CADAM workshop, Munich, October 1998.
CADAM (1999) Proc., 3rd CADAM workshop, Milan, May 1999.
CADAM (1999) Proc., 4th CADAM workshop, Zaragoza, November 1999.

Cao Z; Pender,G; Wallis, S; and Carling,P (2004) "Computational Dam-Break Hydraulics over Erodible Sediment Bed", *J. Hydraul. Engg*, 130 (7),689-703

Capart, H. and Young, D.L. (1998) "Formation of a jump by the dam-break wave over a granular bed" J. Fluid Mech. 372, 165-187.

Cao, Z. (1999) ''Equilibrium near-bed concentration of suspended sediment.'' *J.Hydraul. Eng.*, 125(12) 1270–1278

Das M M (1978) "The Effect of Resistance on Steady and Unsteady Free Surface Profiles".PhD thesis, Universityof Strathclyde, Glasgow,UK.

Dressler R. F. (1952) "Hydraulic resistance effect upon dam-break functions." *J Research,* National Bureau of Standards, 49, No. 3, 217-225.

Dressler R. F. (1954) "Comparison of Theories and Experiments for the Hydraulic Dam-break Wave". *International Association of Scientific Hydrology, Publication* ,38, 319-329.

Dronkers (1964) "Didal Computation in River and Coastal Water." North Holland, Amstar dam.

Dietrich, W. E. (1982), "Settling velocity of natural particles," *Water Resour. Res.*, 18(6), 1615–1626.

Diplas, P. and Vigilar, G. (1992), "Hydraulic geometry of threshold channels," *J. Hydraul Engg.*, 118 (4), 597-614.

Dutta, D., Herath, S. & Musiake, K. (2003). "A mathematical model for flood loss estimation", *J. Hydrology*, 277, 24-49.

Einstien,H.A. (1934) "Der hydraulische oder progile-radius'',Schweizerische Bauzeitung,Zurich,103,no.8(February 24):89-91.

Einstein, H. A. & Banks, R. B. (1950) "Fluid resistance of composite roughness. Transactions", American Geophysical Un-ion 31(4): 603-61**0**

Fread D.L., Harbaugh T.E.Brown and Rogers (1977) ''Simulation of the hydraulic events during and following the Teton Dam failure.'' Proc., Dam Break Flood Routing Workshop, Water Resources Council, 131–163.

Fennema R.J., Chaudhry, M.H. (1987) "Simulation of one-dimensional dam-break Flows". *J. Hydraul Res*; 25:41-51.

Fennema, R.J. and Chaudhry, M.H. (1990) "Explicit Methods for 2D Transient Free-SurfaceFlows", *J Hydraul.Eng.* 116(8), 1013-1034.

Francisco,Alcrudo et. al (1992) "Flux Difference Splitting For 1D open channel Flow equations" *International J. Numerical Methods in Fluids*, 14, 1009- 1018.

Ferreira R. M. L. and Leal J. G. A. B(1998)" 1D Mathematical Modeling of The Instantaneous -Dam-Break Flood Wave Over Mobile Bed: Application of TVD and Flux splitting Schemes" Proc, European Concerted Action on Dam-Break Modeling, Munich

Froehlich, D. C. (1995a) ''Embankment dam breach parameters revisited." *Water Resources Engineering, Proc. 1995 ASCE Conf. on Water Resources Engineering*, New York, 887–891.

Froehlich, D. C. (1995b) ''Peak outflow from breached embankment dam.'' *J. Water Resource. Plan. Manage.* 121(1), 90–97.

Fraccarollo and Armanin (1998) "A semi-analytical solution for the dambreak problem over a movable bed." Proc., European Concerted Action on Dam-Break Modeling, Munich,1998,

Fraccarollo, L. and Capart, H. (2002) ''Riemann wave description of erosion dam-break flows.'' *J. Fluid Mech.*, 461, 183–228.

Fagherazzi S.; Rasetarinera P.; Hussaini M. Y.; and Furbish D.J (2004) "Numerical Solution of the Dam-Break Problemwith a Discontinuous Galerkin Method" *J. Hydraul. Engg*, 130, (6), 532-539

Garcia R, Kahawita R.A. (1986) solution of the St.Venant equations with MacCormack finite-difference scheme.'' *Int. J. Numerical. Methods in Fluids*, 6, 259–274.

Garcı´a Navarro, P., Alcrudo, F., and Saviro´n, J. M. (1992) ''1-D open channel flow simulation using TVD McCormack scheme.'' *J.Hydraulic. Eng.*, 118(10)

Garcia-Navarro et al(1992) 'Flux-difference Splitting for 1D Open Channel Flow Equations,' *Int. J. Numerical Methods in Fluids*, 14, 1009-101

GRAHAM (1998) "Dam failure inundation maps – are they accurate?" Proc of the 2nd CADAM workshop, Munich, October 1998.

Gottardi, G. and Venutelli (2004) "Numerical Prediction of Dam-Break Flows in General Geometries with Complex Bed Topography" *J. Hydraul. Engg*, 130, (4), 332-340.

Hromadka (1985) "A two-dimensional dam –break flood plain model**"** *Adv. Water Resources*, 8, 7-14.

Hicks F. E., Steffler P. M., and Yasmin N., (1997) "One dimensional dam break solutions for Variable width channel" *J. of Hydraul. Eng.*, 123(5), 464-468.

Harten A, Hyman P. (1983) "Self-adjusting grid methods for one dimensional hyperbolic conservation laws." *J Comp Phys;* 50:235-69

Harten A. (1983), "High resolution schemes for hyperbolic conservation laws". *J.Comput Phys;* 49: 357–93.

Hunt, B. (1982)"Asymtotic solution of dam break solution" *J. Hydraul.Div.*,109, noHY1, 115-126.

Hunt, B. (1984) "Dam break solution" *J. Hydraul. Div.*, 110, no HY6, 675-685.

Hembree, C. H., Colby, B. R., Swenson, H. A., and Davis, J. R. (1952) "Sedimentation and chemical quality of water in the Powder River drainage basin", Wyoming and Montana., Circular 170, U.S. Geological Survey, Washington, D.C.

Han, Q. W., Wang, Y. C., and Xiang, X. L. (1981) Initial dry density of sediment deposit, *J. Sediment Res.*, 1

Horton, R.A. 1933 Separate roughness coefficients for channel bottom and sides, Engineering News Record 111, no.22 (November 30): 652-53.

HEC (2001) HEC – RAS Hydraulic Reference Manual, U.S. Army Corps of Engineers, Davis, CA.

Ikeda, S. (1982) "Incipient motion of sand particles on sand slopes" *J. Hydraul Div*, 108, No. HY1,95-114, Jan. 1982

Jha, A. K., Akiyama, J., and Ura, M. (1994). ''Modeling unsteady open channel flows, modification to beam and warming Scheme" J. . *Hydraul Engg.*, ASCE, 120(4), 461-476

Jin M and Fread DL (1997) Jin, M., and Fread, D. L. (1997) ''Dynamic flood routing with explicit and implicit numerical solution schemes.'' *J.Hydraul. Engg.*, 123(3), 166–173.

Katopodes, N., and Strelkoff, T. (1978). "Computing two-dimensional dam-break flood waves." *J. Hydraul. Div.*, 104 (9), 1269-1288.

Katopodes N.D. (1984) "A Dissipative Galerkin Scheme for Open-Channel Flow" *J. Hydraul. Engg.*, 110(4) 450-464.

Katopodes N.D. (1984) “Two dimensional Surges and shocks in open channels” *J. Hydraul. Engg.*, 110(6), 794-812

Komura, S. (1963), ‘Discussion of Sediment transportation mechanics:Introduction and properties of sediment.’ ” *J. Hydraul. Div*., 89(1) 263–266

Khodashenas, S.R. and Paquier, A. (1999),”A geometrical method for computing the distribution of boundary shear stress across irregular straight open channels, *J. of Hydraul Res.* 37 (3) 381-388

Khodashenas and Paquier (2002) “River bed deformation calculated from boundary shear stress” *J. Hydraul Res.*. 40, (5), 603-609

Khodashenas1 S R.(2006) “SIMULATION OF THE DEFORMATION OF A RIVER BY A 1D-3D MODEL”, Joint International conference of Civil and Building engineering ,June 14-16,Montreal,Canada.

Levin,L. (1952) “Evolution of Wave created by Breach of Large Dams”. Yogoslav, Nat.Committe on Large Dams II meeting, Trans., 104-118.

Liska R and Wendroff B (1999) “Two dimensional Shallow water equations by composite schemes” *Inter.J. Numerical Methods in fluids.*

Lane, E. W., and Koelzer, V. A. (1953),“ Density of sediments deposited in reservoirs.” Rep. No. 9 of a Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams, Engineering District, St. Paul, Minn.

Lotter, G. K. (1933) “Soobrazheniia k gidravlicheskomu raschetu rusel s razlichnoi sherokhovatostiiu stenok (Considerations on hydraulic design of channels with different roughness of walls). Izvestiia Vsesoiuznogo NuachnoIssledovatel’skogo Instituta

Gidrotekhniki (Transactions, All-Union Scientific Research Institute of Hydraulic Engineering) 9: 238-241. Leningrad.

Mac Cormack (1969) R.W. "The effect of viscosity in Hyper-velocity scattering", Amer. Institute of Aeronotics and Astronautics, paper 69-354. Washington: DC, 1969

MacDonald, T. C., and Langridge-Monopolis, J. (1984). ''Breaching Characteristics of dam failures.'' *J. Hydraul. Engg.*, 110(5), 567–586.

Macchione F and Morelli M. A. (2003) "Practical Aspects in Comparing Shock Capturing Schemes for Dam Break Problems" *J. Hydraul. Eng.*, 129(3), 187-195

Macchione F., Viggiani G. (2004) "Simple Modeling of dam failure in a natural river " *J. Water Management*, (ICE) WA157 ,2004, issue 01

Motayed, A.K and Krishanmurthy, M (1980)"Composite roughness of Natural Channels"*J. Hydraul Div*.,106,(6),1111-1116.

Nguyen, H. T. and. Fenton J. D. (2004) "Identification of roughness in open channels" Advances in Hydro-Science and -Engineering, Volume 6, M. S. Altinakar, S. S. Y.

Nguyen, H. T. and. Fenton J. D. (2005) "Identification of roughness for flood routing in compound channels", Proc. 31st Congress, Int. Assoc. Hydraulic Engg and Res., Seoul, Korea, 11-16 September, 2005, Published on CD

Nujic, M. (1995) "Efficient Implementation of Non-oscillatory Schemes for the Computation of Free Surface Flow", JHR, 33(1), 101-111

Ponce, V.M. (1981) "Modelling of Gradual Dam breaches" *J. Hydraul. Div*., 107 ,No.HY7.829-837

Pritchard, D. and Hogg A.J., (2002) "On sediment transport under dam-break flow" *J. Fluid Mech.* (2002), 473, pp. 265-274.

Paquire A. and Balayn P. (1998) "Unsteady cases of validation for a 1-D sediment transport model" Proc., European Concerted Action on Dam-Break Modeling, Munich, 1998,

Paquire, A. (2002) "Sediment transport models used by Cemagref during Impact project"Proc., Impact 1st Impact Workshop HR Wallingford - 16th - 17th May 2002

Patra K.C. (2003) "Hydrology and Water Resources Engineering" Narosa Publishing House, New Delhi India, page 58

Penning-Rowsell, E.C. & Chatterton, J.B. (1977)"The benefits of flood alleviation" Saxon House, Teakfield, Westmead, UK.

Prooijen B C. van; Battjes J A.; and Uijttewaal3 W S. J. (2005)" Momentum Exchange in Straight Uniform Compound Channel Flow" *J. Hydraul. Eng.*, 129(3), 187-195

Rahman M. and Choudhury M.H. (1998) ''Simulation of dam-break flow with grid adaptation.'' *Advances in Water Resoures*, .21(1), 1-9.

Ritter A. (1892) "Die fortpfazung der wasser-wellen Zeitschrift des Vereins deutscher ingenieure", 36, No. 33, 947-954.

Re, R. (1946) “Astudy of Sudden Watervreleases from a Body of water to a Canal by the Graphical Method’’, La Houllie, Blanche, Vol.1, No.3, 181-187.

Saint-Venant A. J. C. ((1871a). ‘‘Theorie dumouvement non permanent des eaux, avec application aux crues des rivieres et al’introduction de marees dans leurs lits.’’ Comptes rendus des seances del’ Academie des Sciences, 36, 174–154.

Saint-Venant A. J. C. (1871b) ‘‘Theorie du mouvement non permanent des eaux avec application aux crues des rivieres eta l’introduction de marees dans leurs lits.’’ Comptes rendus des seances de l’ Academie des Sciences, 36, 237–240.

Sakkas- Strelkoff (1973) ‘‘Dam break hydraulics in prismatic dry channel” *J. Hydraul. Div.*, Vol:99, noHY12

Sakkas- Streloff (1976) ‘‘Non dimesional dam break hydraulics profile” *J. Hydraul. Div.*, Vol:102, noHY2

S. Soares Frazão, Alcrudo F. and Goutal N. (1999) “Dam-break test cases summary” 4th CADAM meeting - Zaragoza, Spain, November 1999

Sarma A.K. (1999) “A study of two-dimensional Flow propagating from an opening in the river dike” PhD thesis, Assam Engg. College, Gauhati University, Assam, India.

Sanders B. F. (2001)**“** High-resolution and non-oscillatory solution of the St.Venant equations in nonrectangular and non-prismatic channels” *J. Hydraul. Research*, 39(.3), 321-330

Sarma A. K. and Das M. M. (2003) “Analytical solution of a flood wave from dike failure ” *J. Water and Maritime*, issue 1, 41-4

Saikia, M. D. and Sarma A. K. (2006[1]) "Numerical Simulation Model for Computation of Dam Break Flood in Natural Flood Plain Topography," *J. Dam Engineering*, June 2006, Vol.17, No.1, 31-50

Saikia, M. D. and Sarma A. K. (2006[2]),Numerical Model for simulating Flow and River Bed Profiles in a Natural River Under Dam Failure Condition, The 15th IASTED International Conference on Applied Simulation and Modeling(ASM 2006),June 26-28, 2006,Rhodes, Greece.

Sanders B. F. (2001) "High-resolution and non-oscillatory solution of the St.Venant equations in non-rectangular and non-prismatic channels", *J. Hydraul research*, 39(3), 321-330.

Singh V. P. and Scalatos, P. D. (1988), "Analysis of Gradual Earth Dam failure" *J.Hydraul Engg*, 114, (1), 21-42

Singh, V. P. (1996) ''Dam breach modeling technology" Kluwer, Dordrecht, The Netherlands 1, 1–9.

Shige-eda ,B. and Akiyama, J. (2003) ''Numerical and Experimental Study on two Dimensional Flood Flows With and with out Structures" *J. Hydraul. Engg.*, 129 (10) 817-821

Su, S.T., and Barness A.H. (1970) ''Geometrical and Frictional effects on sudden releases" *J. Hydraul. Div*.,

Sweby P. K. (1984) ''High resolution schemes using flux-limiters for hyperbolic conservation laws.'' SIAM J. Numer. Analysis, 21(5), 995–1011.

Swamee, P. K., and Ojha, C. S. P. (1991) “Drag coefficient and fall velocity of nonspherical particles.” *J. Hydraul. Engg*., 117(5), 660–667

Tseng M.H., Chu C.R.(2000) “The simulation of dam-break flows by an improved predictor– corrector TVD scheme” Adv Water Resour2000; 23:637–43

Valiani A., Caleffi V.; Zanni A. (2002) ‘‘Malpasset Dam-Break Simulation using a Two-Dimensional Finite Volume Method” *J. Hydraul. Engg*, 128 (5) ,2002, 460-472

Whitham, G.B. (1955)”The effect of hydraulic resistance in Dam break problem” Proceedings of the Institution of Civil Engineers, 113 (1) 29-46

Walder, J. S., and O’Connor, J. E. (1997) ‘‘Methods for predicting peak discharge of floods caused by failure of natural and constructed earth dams.’’ *Water Resources Research*.

Wahl, T. L. (1998) ‘‘Prediction of embankment dam breach parameters, a literature review and needs assessment.’’ Rep. No. DSO-98-004, Bureau of Reclamation, U.S. Department of the Interior, Denver, 60

Wang J. S., Ni H. G., and He Y. S. (2000) “Finite-difference TVD scheme for computation of dam-break problems” *J Hydraul. Engg* 2000; 126(4):253–62

Wu W and Wang S. S. Y.,(2006) Formulas for Sediment Porosity and Settling Velocity, *J. Hydraul. Engg.*, 132, (8), August , 2006.

Wu, W. and Wang, S. S. Y. (2006), ‘‘1-D Numerical Simulation of Morphodynamic Processes under Dam Break and Overtopping Flows” World Environmental and Water Resources Congress. 21-25 May. Omaha, Nabeska, USA, EWRI, ASCE.

Wu, W. and Wang, S. S. Y. (2007) "One-Dimensional Modeling of Dam-Break Flow over Movable Beds" *J. Hydraul. Engg*, 133(1), 2007.48-58.

Wang, K. P. Holz, and M. Kawahara (eds), Proc. 6th Int. Conf. on Hydro-Science and -Engineering, Brisbane, 31 May - 3 June 2004, University of Mississippi.

Yen B.C, (2002) "Open Channel Flow Resistance" *J. Hydraul. Eng*., 128 (1),20-39.

Yang, J.Y., Hsu, C.A. and Chang, S.H. (1993) "Computation of free surface flows; Part-I: One Dimensional Dam-Break Flow," *J. Hydraul. Research*, 31(1) 19-34.

Zhao H.W., Zhao D.H. Shen H.W (1994) "Finite volume Two Dimensional Unsteady Flow model for River Basins" *J. Hydraul. .Engg*, 129, (7),863-883**.**

Zhao D.H. and et al (1996), ''Approximate Riemann Solver in FVM for 2D Hydraulic Shock Wave Modeling" J. of Hydraul. Engg.122 (12), 692-702.

Zoppou .C. and Roberts S. (2000), "Numerical solution of the two dimensional unsteady dam break." Applied Mathematical Model, 24(7), 457-475.

Zoppou, C., and Roberts, S. (2003). "Explicit schemes for dam-break simulations." *J. Hydraul. Eng.*, , 129 (1) 11-34.

Zhou J.G., Causon, D.M., Mingham, C.G. and Ingram D.M.(2004) "Numerical Prediction of Dam-Break Flows in General Geometries with Complex Bed Topography" *J. Hydraul. Engg*, ASCE, 130, (4) 332-339.