DAM BREAK FLOW ANALYSIS OF HIRAKUD DAM

A DISSERTATION

submitted in partial fulfillment of the
requirements for the award of the degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By
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CANDIDATE'S DECLARATION

I here by certify that the dissertation entitled, "**DAM BREAK FLOW ANALYSIS OF HIRAKUD DAM**", which has been presented in partial fulfillment of the requirement for the award of the DEGREE OF MASTER OF ENGINEERING IN WATER RESOURCE DEVELOPMENT and submitted in the Water Resources Development Training Centre, University of Roorkee, Roorkee is an authentic record of my own work carried out during the period from 16th July to 31st December 2000, under the supervision of Dr U.C. Chaube, Professor, WRDTC, University of Roorkee and Dr. P.K. Mohapatra, Scientist 'B', N.I.H., Roorkee.

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

Place: Roorkee

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LIST OF SYMBOLS

x = Distance along the longitudinal direction.

t = time co-ordinate.

A = Cross-sectional area.

h = flow depth in meter.

Q = Discharge in Cumecs.

q = Lateral inflow per unit length of channel.

g = Acceleration due to gravity.

S_o = Bed Slope.

S_f = Friction slope.

n = Manning's roughness Co-efficient.

R = Hydraulic radius.

Δx = Spacing parallel to the t-axis represent location of cross- sections.

Δt = Spacing parallel to the x-axis represent time line.

i,j = Subscript for numerical rectangular grid identification designates the x position and particular time level.

k = Represents any variable Q or A.

θ = Weighting factor.

N = Total numbers of nodes.

Km = Kilometer.

Sq. km = Square Kilometer.

m = Meter.

mm = Millimeter.

M.Cum. = Million Cubicmeter.

M.Hect.M. = Million Hactremeter.

hr =Hour

E = East.

N = North.

min = Minute.

Fig. = Figure

SYNOPSIS

In the present work, dam break flow analysis of Hirakud dam is attempted, due to a hypothetical failure case. For this purpose, a widely adopted computer model 'DAMBRK' is used. In this model, complete St. Venant equations in one-dimension is used for channel routing. The numerical solution of the governing equations is performed by an efficient four point Preismann Scheme. The required data set is prepared on the basis of information collected from various sources, for unavailable data, suitable assumed valued are used. To determine the Manning's roughness (n), a reach located downstream of the study area is used. The outflow hydrograph at the dam site and different locations downstream of dam are presented. A sensitivity analysis is performed to study the effects of breach width, time of breach formation and bed roughness, on the flood wave propagation due to dam break flow. Based on the available topographic information of the study area, a flood inundation map is also prepared.

INTRODUCTION

1.0 GENERAL

The dam is one of the most important hydraulic structures to store excess water in the reservoir. Its main function is to release water for irrigation, and to moderate flood flows through river and thus, to protect the downstream areas from loss of life and property. Other important purposes include water supply, hydropower generation and navigation etc. Usually, the downstream and adjoining areas of a dam are highly cultivated due to high fertility of the flood plains and/or availability of water through canal networks and consequently become thickly populated because of various developmental activities.

According to International Committee on Large Dams (**ICOLD**), there are more than 35000 large dams existing throughout the world and many more are under construction. In India, about 3000 major, medium and minor dams are in existence. Dams are found to be much more vulnerable during and immediately after the construction, especially within five to seven years and beyond this period the risk of failure decreases, also it seems to be increased after its useful life. Every development has to pay a price, and is so with the dams in terms of their failures. There are many instances of dam break in India and abroad. Three major causes of dam failure are overtopping failure (inadequate spillway, misuse of road embankment etc.), foundation failure (fault movement, settlement etc.) and piping as well as seepage of embankment dams. The overtopping failure is caused due to the rise of water level in the reservoir beyond its specified value. Settlement and fault movement are the causes of foundation failure. Due to the presence of a dam, there is a feeling of safety and the area becomes thickly populated, and thus, a dam becomes a potential source of disaster due to the risk of its failure. When a dam fails, the sudden release of the reservoir water forms a catastrophic flood and it results in the devastating loss of human lives as well as valuable properties in the downstream region.

The characteristics of a dam break flood are different from those of an ordinary natural run-off generated flood. Very high peak discharges during a short time, occurrence of bores or shock waves, fast and violent flooding of the banks resulting in strong two-dimensional effects, presence of mixed flow regimes, flooding with abnormal dissipation effects, and transport of debris/solid materials accumulated in the reservoir are associated with a dam break flood. The behavior of the bed friction causing turbulence during such a flow is not well understood, and so with the mechanism of dam failure. It leads to difficulties in calibrating any model to study dam break flows. Thus, the dam break flood analysis becomes a special problem to be dealt very carefully.

The processes involved especially during an earth dam failure are very dynamic and complicated. A prior knowledge of this devastating flood resulting from dam rupture can be used as a basis for rational zoning in the downstream valley of the dams as well as in the preparation of emergency action plan (EAP) well ahead of any possible catastrophe. One of the aspects of the EAP is to describe the anticipated dam failure scenarios and corresponding arrival times of dam break flood waves at different locations of interest downstream areas that could be inundated.

Dam break flood analysis is, now-a-days, a part of dam design and safety analysis, including downstream and upstream risk evaluation. It occupies a very important place in water resources engineering practices, and is useful in (a) establishing the required dam spillway capacity, (b) environmental and safety impact evaluation of dams or other special structures built in a river valley, (c) Valley planning and zoning, (d) implementation of operational emergency and safety procedures, such as warning systems and evacuation plans downstream dams, and (e) solving special and unexpected problems arising from very high risk of a dam or other river obstruction failure.

Analysis of a dam break flood can be performed using analytical, experimental, and/or numerical models. The advantages and disadvantages of these methods are presented in Table 1 (Anderson et al, 1984). However, a numerical model is the most convenient tool for fast and systematic study. Among various numerical models available for dam break flood analysis, **DAMBRK** is one of the most widely used computer programs. The significance of this model is to represent the dam failure and the application of hydrodynamic theory to predict the dam break wave formation and its downstream progression. The basic computer program was first developed by Fread (1979) and was published by National Weather Service, U.S.A. The model has wide practical applicability and is being used by a large number of studies in the area of dam break flow. Geometrical features of the channel and reservoir, time and shape of dam-breach, flow in a compound cross-section, presence of other hydraulic structures are taken into account in this model. It can also function with various levels of input data and is computationally efficient.

Table 1 : Advantages and disadvantages of three methods

Approaches	Advantages	Disadvantages
Analytical model	1. Handy, general information which is usually in formula form 2. Used for preliminary studies 3. Computers are not essential	1. Restricted to single geometry and physics 2. Usually restricted to linear problems.
Experimental model	1. Capable of being most realistic	1. Equipment required 2. Scaling problems 3. Measurements difficulties 4. Operating cost 5. Time consuming.
Numerical model	1. No restriction to linearity 2. Complicated physics can be treated 3. Time evolution of flow can be obtained 4. Different scenarios 5. Less time for analysis	1. Truncation errors 2. Boundary condition 3. problems 4. Computer costs 5. Development of a code is time consuming 6. A number of processes are still not modelled adequately.

1.1 REVIEW OF LITERATURE

The literature in dam break flows is vast. There are various aspects to these studies. However, only routing aspect is presented here and other aspects are beyond the scope of this dissertation. For the sake of simplicity, previous studies in dam break flows are presented here under three categories, viz, (1) Analytical models, (2) experimental model and (3) Numerical models. Excellent review articles have been presented by Basco (1989), Almeida et al. (1994) and Singh (1996).

1.1.1 Analytical Models

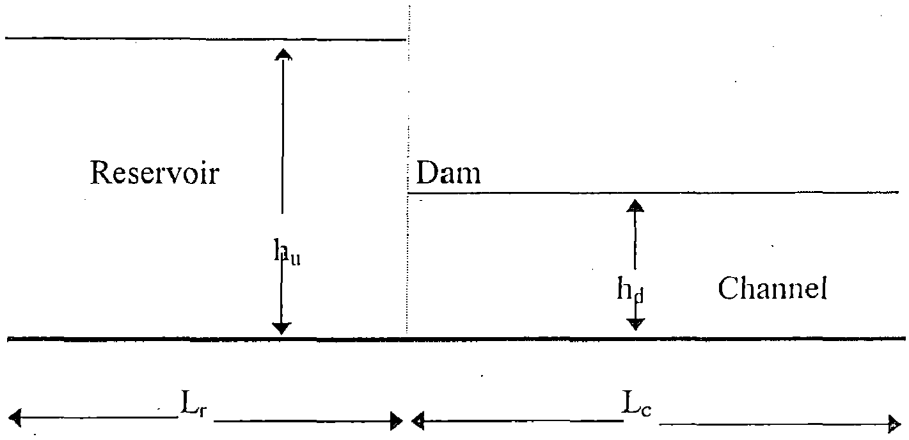
Studies to understand the basic mechanics of dam break flows (DBF) are very old and date back to the earliest attempt by Ritter in 1892. Ritter derived an analytical solution for the hydrodynamic problem of instantaneous dam-break in a frictionless and horizontal channel of rectangular shape. In Ritter's solution, both the reservoir and the channel was assumed to be infinite and the channel downstream was assumed to be dry. The flow depth (h) and velocity (u), at any place downstream of the dam are functions of distance (x) , time (t) and reservoir water level (h_o). The analytical solutions given by Ritter (1892) are ;

$$h = \frac{1}{9g}\left(2c_0 - \frac{x}{t}\right)^2 \tag{1}$$

$$u = \frac{2}{3}\left(C_0 + \frac{x}{t}\right) \tag{2}$$

Where, wave celerity, $C_0 = \sqrt{gh_0}$. According to these equations, the flow depth and the discharge attained at the dam-site are constant in time and represent critical flow condition there. The shape of the free surface is a parabola and the tip speed is twice that of the disturbance propagated upstream. Later, Dressler (1952) and Whitham(1955) included the effect of the bed resistance in the analysis of DBF and derived analytical expressions for the velocity and height of the wave-front. Pohle(1952) considered two dimensional flow in x and z direction Using Lagrange representation, he concluded that in the initial regime, the vertical acceleration is the predominant parameter. When the vertical acceleration is decreasing, the effect of channel cross-sectional geometry, bed friction and bed slope become more important and the wave profile will then converge to one-dimensional analytical solution.

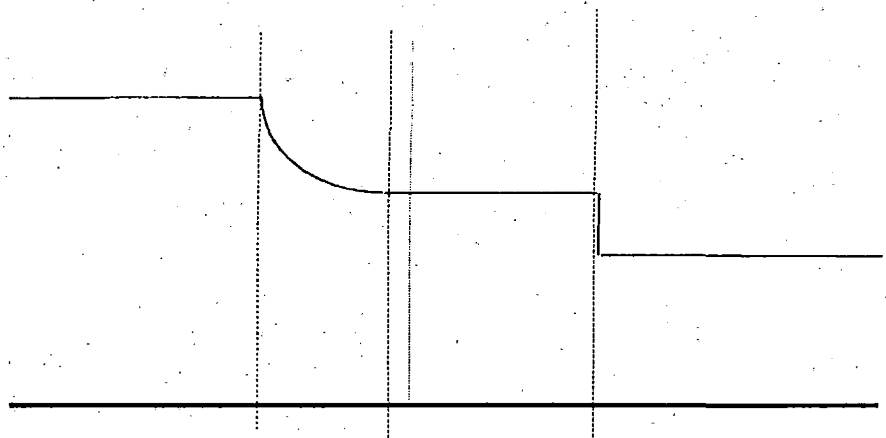
Stoker(1957) extended the Ritter solution to the case of wet-bed condition in the downstream. He derived analytical expressions for the surface profile in terms of the initial depths upstream and downstream of the dam. In Stoker's solution, there are four distinct zones, viz. two undisturbed zones, one each in the upstream and downstream side, one drawdown zone and one zone with a constant bore height (Fig. 1). In Stoker's solution, the velocity of bore propagation and the constant bore height are attained instantaneously. The analytical equations derived by Hunt(1982, 1987) considered finite length reservoirs. However, Hunt's solution was based on the assumption of a kinematic wave.



(a)Surface profile (prior to dam-break)

Figure 1: Stoker Solution for DBF

(Fig. 1/--contd/--



(b)Surface profile (time, t, after dam-break)

Figure 1: Stoker's Solution for DBF

1.1.2 Experimental Model

The complexity of the unsteady flow due to a dam failure necessitates for more accuate modeling, than the analytical models. Experimental modeling is one of the methods to analyse the real flow phenomenon. Some important purposes of experimental modeling are verification of computational models, complete analysis of real cases, and more understanding of the DBF problem.

Escande et al.(1961) presented detailed results obtained from expreimental studies using a 1.6 km reservoir and 12 km long downstream reach with fixed bed. They presented the front wave profile, due to the sudden failure of a dam for different conditions as well as the variation of front wave velocity with bed roughness, initial reservoir head and initial channel flow. One of the complete set of laboratory data on dam break flows was collected at the U.S.A. Army Engineers, Waterways Experiment Station

(WES,1960). Rajor (1973) presented results for dam break flows, obtained through experimental modeling of real valleys and of prismatic and non prismatic channel.

Dressler (1954) experimentally showed that the depth at the dam site does not attain a constant value instantaneously as predicted by Ritter. It takes approximately nine non-dimensional time units to reach the constant Ritter's value. He also found that the tip speed of dam break flow is less than $2\sqrt{gh_0}$.

The DBF along an alluvial channel may change the valley geomorphology. Simons et al. (1980) presented experimental data to assess change in flood stage, resistance to flood, and transport of deposited sediment following failure of a dam. He concluded that in general, when a dam fails the interaction between the water and sediment transport and the river stability is not well understood.

All the above experimental studies are for straight channel reaches, however, the natural channels are seldom straight and meander in the channel alignment produce lateral gradients in the flow surface. Miller and Chaudhry (1989) presented the experimental results for dam break flows in meandering channels. Memos (1983) presented on experimental results that at a partial dam failure (breach width less than valley width) three-dimensional effects are dominant during the first instance of the break. Martin (1983) presented the results of a total dam break in a rectangular and in a channel with divergent side walls (dry bed). Similar observations were also presented for convergent and divergent channels by Townson and Al-Salihi (1989) and Bellos et al. (1992).

1.1.3 Numerical Models

A numerical model is the most convenient tool for a fast and systematic analysis of dam break flow. Generally in a numerical model, the dam break flow is simulated by three consequential steps, i.e. (i) routing of the inflow hydrograph from the reservoir inlet to the dam site, (ii) dam break mechanism and (iii) routing of the dam break flow in the downstream channel.

All the numerical models may be categorized, depending on the equations used to model the phenomenon, numerical scheme used to solve the equations, and, implementation of different boundary physical conditions.

Governing equations used:

In most of the numerical models, available in literature, one dimensional St.Venant equations are used as the governing equations (Fennema & Chaudhry) 1987, Molls and Molls (1998), Fread 1988). One dimensional St.Venant equations assume a hydrostatic pressure distribution along vertical plane. Basco (1989) pointed out limitations to the St.Venant equations in dam break flows analysis. In some studies, one-dimensional Boussinensq equations are used to simulate the dam break flow (Carmo et al. 1993, Gharangik and Chaudhry 1991, Mohapatra and Singh 2000). Two-dimensional St.Venent equations in x,y plane are used for dam break flow analysis by some researchers (Fennema and Chaudhary 1990, Alcrudo and Gracia-Novarro 1992, Mohapatra and Bhallamudi 1996). Two dimensional Navier –Stokes equations in x and z plane to study dam break flows are also presented in literature (Hirt and Nichols 1981, Tome and McKee 1994 and Mohapatra et al. 1999).

Numerical methods:

Different numerical methods available in the literature are (I) finite element method, (ii) finite difference method and (iii) method of characteristics. Detailed descriptions of the above methods are available in Chaudhry (1993).

DAMBRK model is the National weather service (NWS) dam break flood forecasting model developed by Fread (1979,1988). In this model, the expanded form of St.Venent equations are used for routing of dam break floods in channels. This model allows the failure timing interval and terminal size and shape of breach as input. It gives the extent of and the time of occurrence of flooding in the downstream valley by routing the outflow hydrograph through the valley.

SMPDBK is a simplified version of the dam break flood forecasting model presented by Wetmore and Fread (1984) for quick prediction of downstream flooding caused by a dam failure. This model is an useful forecasting tool in a dam failure emergency when warning response time is short, little data are available and large computer facilities beyond one's reach. It is also very useful for preparing disaster contingency plans. This model consists of three main components: (i) calculation of the peak discharge at the dam, (ii) approximation of the downstream channel as a prismatic channel and (iii) determination of the peak flood at specified cross section of the downstream channel.

MIKE 11 is a software package developed at the Danish Hydraulic Institute (DHI) for the simulation of flow, sediment transport and water quality of estuaries, rivers, irrigation systems and similar water bodies. It is developed especially for application of micro computers. It offers an unique and user-friendly tool for dam break flow analysis.

This model consists of a number of modules which in principle operate independently and give a rational and user friendly execution and enhance the flexibility of the package. Presently, MIKE 11 is also available with MIKE – SHE. It is workable in a window environment, the graphics facilities are excellent, and compatibility to GIS make it a versatile software for dam break flow. MIKE 21 also developed at Danish Hydraulic Institute uses two dimensional St. Venant equations, governing the flow.

TELEMAC system (Hervouet 1996) uses finite element method to solve the governing flow equations for the analysis of the dam break flows. Complicated river geometry can be considered by this computer program. A module solving Boussinesq equations is also available in TELEMAC.

Besides the above computers softwares for dam break flows, some research papers dealing with advanced techniques are presented below.

Alam et al. (1995) presented the collocation method in conjunction with Quintic Hermite elements to solve the system of flow equation for DBF. Quntic Hermite eliments are used to provide the high resolution required in the solution of discontinuities for producing satisfactory stable solution. This model can simulate both sub-critical and super-critical flows in different parts of the channel or in a sequence in time.

A general mathematical model was developed by Molls and Chaudhry (1995) to solve unsteady, two-dimensional depth averaged equations. This model uses boundary fitted coordinates and includes effective stresses. It may be used to analyze sub and super- critical flows. The time differencing is accomplished using a second order accurate Beam and Warming method, while the spatial derivaties are approximated by second order accurate central finite differencing. The equations are solved on a non-staggered grid using an alternating-direction implicit scheme. The model is used to analyze a wide variety of hydraulics problems including a dam break simulation.

A characteristics – based upwind, explicit numerical scheme is developed by Jin and Fread (1997) for one-dimensional unsteady flow modeling of dam break flows into the (NWS) FLDWAV model, in combination with the original four point implicit scheme. The new explicit scheme is extensively tested and compared with the implicit scheme and provides improved versatility and accuracy in some situations, such as waves due to large dam break and other unsteady flow with near critical mixed flow regimes. A technique for implicit – explicit multiple routing is introduced to incorporate the advantages of using both schemes.

A high resolution time marching method was presented by Mingham and Causon (1998) for solving the two dimensional shallow water equations. This method uses a cell centered formulation with collocated data rather than a space-staggered approach. Spurious oscillations are avoided by employing monotonic upstream scheme for conservation laws (MUSCL) reconstruction with an approximate Riemann Solver in a two-step Runge-Kutta time stepping schemes. A finite volume implementation on a boundary conforming mesh is chosen to accurately map the complex geometries. These features enable the model to deal with dam break phenomena involving flow discontinuities, sub-critical and super-critical flows. The method is applied to several bore wave propagation and dam break flow problems.

A list of 23 numerical models for dam break flows is presented in Table 2 (Molinaro and Fillippo 1992).

Table 2 : List of 23 Dam- Break Numerical Models

Sl. No.	Agency	Name of Modes
1	USA/National Weather Service	DAMBRK (original)
2.	USA/National Weather Service	SMPDBK (simplified dam-Break)
3.	BOSS	BOSS DAMBRK
4.	HAESTED METHODS	HAESTED DAMBRK
5.	Binnie & Partners	UKDAMBRK
6.	USA/COE-Hydrologic Engineering Centre	HEC-Programs
7.	Tams	LATIS
8.	Institute of Water Resources and Hydroelectric Power Research (IWAR), PR China	DKB 1
9.	Institute of Water Resources and Hydroelectric Power Research (IWAR), PR China	DKB 2
10.	Royal Institute of Technology, Stockholm	TVDDAM
11.	Cemagrer	RUBBER 3
12.	Delft Hydraulics	WENDY
13.	Delft Hydraulics	DELFLO/DELQUA
14.	Consultin Engineers Reiter Ltd.	DYX.10
15.	ANU-Reiter Ltd.	DYNET-ANUFLOOD
16.	ENEL Centro di Ricerca Hydraulics	RECAS
17.	ENEL Centro di Ricerca Hydraulics	FLOOD2D
18.	ENEL Centro di Ricerca Hydraulics	STREAM
19.	Danish Hydraulic Institute	MIKE 11
20.	ETH Zurich	FLORIS
21.	Danish Hydraulic Institute	MIKE 21
22.	EDF-Loabratoire National Hydraulique	RUPTURE
23.	EDF-Loabratoire National Hydraulique	TELEMAC

1.1.4 Previous Works on Hirakud Dam

A preliminary report on dam break flow analysis of Hirakud Dam was carried out using MIKE 11 model (Sahayam 1998). This report was based on one breach condition assumed in the earthen portion in between the two spillways. An inflow hydrograph with Q_{peak} = 69632 cumecs, time to peak = 54 hrs and time base = 216 hrs was used to study the dam break flow in this preliminary report.

The breach parameters used in the above report are: initial breach level = 195.68 m, final breach level = 160.38 m, initial breach width = 0.1 m, final breach width = 250 m, breach development times = 3 hrs, side slope of breach = 0.75.

Some important results of the above study are presented in Table 3.

Table 3. Results of earlier dam break flow study of Hirakud Dam

Chainage from dam (km)	Max. water level (m)	Time of occurrence hrs-min
0	167.78	79-57
6	160.29	80-33
30	141.75	82-12
310	38.16	106-57

In the above report, following suggestions were recommended for future studies.

i) Sensitivity analysis has to be carried out assuming different breach conditions especially the breach width, depth and breach development time in the linear mode.

ii) A systematic study using erosion based failure mode also need to be done.

iii) Studies based on the breaches on the spillway portion which will fail instantaneously have to be also studied.

iv) Based on all the studies a somewhat realistic breach has to be selected and adopted based on which a detailed innundation map and emergency action plan need to be prepared.

1.2 OBJECTIVES AND SCOPE OF THIS STUDY

In the present study, an attempt has been made to study the dam break flows resulting due to a hypothetical failure of Hirakud Dam. For the above purpose, a numerical model 'DAMBRK' has been used.

The objectives of the study are:

i) calibration of model parameters such as Manning's roughness coefficient (n) and computational distance step size (Δx),

ii) routing of flood wave movement due to hypothetical failure of Hirakud Dam, upto 40 km distance downstream of the Hirakud dam,

iii) estimation of time of travel of the dam break flow,

iv) sensitivity analysis showing effects of various breach parameters and bed roughness values, and

v) preparation of flood inundation map for the worst possible scenario.

Scope of the study is as given below:

Importance of dam break flow study, various issues and literature review relevant to dam break flows are described in the present chapter. Mathematical modeling using the governing equations (DAMBRK model) and their numerical solutions (reservoir routing, dam failure, breach outflow and channel routing) have been presented in the next chapter. A short description about the study area and data availability, are presented in Chapter-3. Input data and computed results of the present study have been discussed in Chapter- 4. In the last chapter, important conclusions and recommendations for future study have been presented.

MATHEMATICAL MODELING

In this dissertation work, dam break flow analysis of Hirakuad Dam is attempted. For this purpose, a hypothetical dam failure scenario is assumed and a computer model DAMBRK is employed. A short description of the DAMBRK model is presented in this chapter.

2.1 DAMBRK

The DAMBAK model is the National Weather Service (NWS) dam break flood forecasting model, developed by Fread (1979, 1988). This model consists of four main components: (1) flow routing in the reservoir, (2) dam failure model describing temporal and geometrical variation of the breach, (3) breach outflow, and (4) flood routing through the downstream valley.

2.1.1 Reservoir Routing

The specified hydrograph at the inlet of the reservoir is routed up to the dam section. For the above purpose the Reservoir Storage Routing Method is used. This is a well known method for reservoir flood routing and can be found in text books (Chaudhry 1993, Subramanya, 1989). In addition to this, if the dynamic property of the reservoir is significant, it can be routed by dynamic routing using St. Venant equations. However, in this case the cross-sections of the reservoir has to be provided.

2.1.2 Dam Failure

The failure time and terminal size and shape of the breach are given as input for the model. The shape is specified by a parameter z as shown in Figure 2, identifying the side slope of the breach (i.e. 1 vertical :z horizontal slope). Rectangular, triangular, or trapezoidal shapes may be specified through this parameter. The final breach size is controlled by the parameter and the terminal width (b) of the breach bottom. The breach bottom width is assumed to start at a point and then increases at a linear rate over the failure time interval (T_b) until the terminal width is attained and the breach bottom has eroded to the final elevation (h_{bm}) which is usually but not necessarily, the bottom of the reservoir or outlet channel section. If the failure time 'T_b' is less than ten minutes, the width of the breach bottom starts at a finite value of 'b' rather than a point. This corresponds to instantaneous failure. The breach may form due to overtopping ($h > h_o$) or piping ($h < h_o$), where, h_o is the dam height.

2.1.3 Reservoir Outflow

The reservoir outflow consists of breach flow (Q_b), flow through spillways (Q_s), flow above the dam (Q_d), if any, and the turbine flow (Q_t).

$$Q = Q_b + Q_s + Q_d + Q_t \tag{3}$$

Q_b and Q_d are calculated using broad-crested weir formulas. Q_s is either calculated by broad-crested wier formula or specified by the spillway rating curve. Q_d is specified as input.

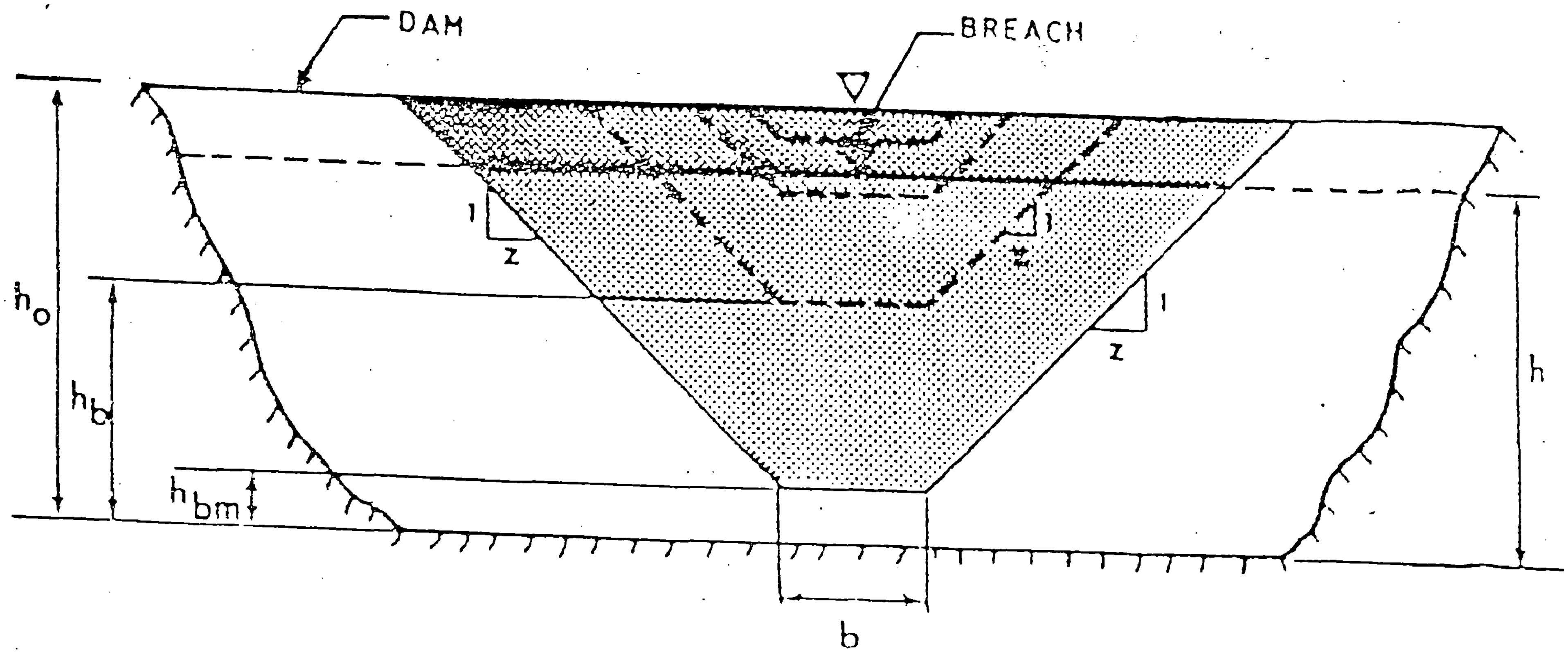


Fig. 2: Front view of dam showing formation of breach

2.1.4 Channel Routing

In the present model, the breached outflow is routed through the downstream channel by dynamic wave routing. The one-dimensional St. Venant Equations for unsteady flow are solved by four point Preissmann scheme. The governing equations and the numerical scheme are given below.

Goverming equations:

The governing equations are the expanded form of the one-dimensional St. Venant equations.

Continuity equation:

$$\frac{\partial Q}{\partial x}+\frac{\partial S_c(A+A_0)}{\partial t}-q=0 \tag{4}$$

Momentum equation:

$$\frac{\partial(Q)}{\partial t}+\frac{\partial(\beta Q^2/A)}{\partial x}+gA(\frac{\partial h}{\partial x}+S_f+S_e)=0 \tag{5}$$

In the above equations (Eqs. 4 and 5), h = water surface elevation, Q = discharge, x = longitudinal distance along the channel, S_c = sinuosity factor which vary with h, A= active cross sectional area of flow, A_0 inactive cross-sectional area, q = lateral inflow per unit distance along the channel, β = momentum correction factor, g = acceleration due to gravity, S_f = boundary friction slope, and S_e = expansion/contraction slope. The expansion co-efficient is taken as –0.05 to –0.75 and the contraction co-efficient as 0.05 to 0.4.

The friction slope 'S_f' used in Eq. (5) is calculated using the Manning's equation.

$$S_f = \frac{n^2 Q^2}{A^2 R^{\frac{4}{3}}} \tag{6}$$

Where, n = Manning's roughness coefficient and R = Hydraulic radius = A/P, where P is the wetted perimeter.

The above governing equations are associated with certain assumptions for their derivations. These assumptions are (Chaudhry 1993):

- The pressure distribution is hydrostatic, this is a valid assumption if the stream lines do not have sharp curvatures.
- The channel bottom slope is small, so that the flow depths measured normal to the channel bottom and measured vertically are approximately the same.
- The flow velocity over the entire channel cross section is unform.
- The channel is prismatic i.e. the channel cross section and the channel bottom slope do not change with distance. The variation in the cross section or bottom slope may be taken into consideration by approximating the channel into several prismatic reaches.
- The head losses in unsteady flow may be simulated by using the steady, state resistance laws, such as the Manning or Chezy equation, i.e., head losses for a given flow velocity during unsteady flow are the same as that during steady flow.

Numerical solution:

The governing equations described in the previous sub-section, are a set of non-linear partial differential equations (Chaudhry 1993). For simplified and idealized cases, analytical solutions of these equations are available. Therefore, solutions of these equations is obtained by numerical methods. The expanded St. Venant Eqs. (4-5) constitute a system of partial differential equations with two independent variables, x, and t, and two dependent variables, h and Q. The remaining terms are either functions of x, t, h, and/or Q, or they are constants. Eqs. (4-5) may be solved numerically by performing two basic steps. First, the differential equations are represented by a corresponding set of finite-difference algebraic equations, and second, the system of algebraic equations is solved satisfying the prescribed initial and boundary conditions.

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

The time derivatives are approximated by a forward difference approximation.

$$\frac{\partial k}{\partial t} = \frac{k_i^{j+1} + k_{i+1}^{j+1} - k_i^j - k_{i+1}^j}{2\Delta t_j} \qquad (6)$$

where k represents any variable (Q, h).

The spatial derivatives are approximated by a forward finite difference approximation using weighting factors, θ and 1-θ.

$$\frac{\partial k}{\partial x} = \theta[\frac{k_{i+1}^{j+1} - k_i^{j+1}}{\Delta x_i}] + (1-\theta)[\frac{k_{i+1}^j - k_i^j}{\Delta x_i}] \qquad (7)$$

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

$$k = \theta[\frac{k_i^{j+1} + k_{i+1}^{j+1}}{2}] + (1-\theta)[\frac{k_i^j + k_{i+1}^j}{2}] \qquad (8)$$

When the finite-difference operators defined by Eqs. (6-8) are used to replace the derivatives and other variables in Eqs(4-5), the following weighted four point implicit, finite difference equations are resulted.

$$\theta\left[\frac{Q_{i+1}^{j+1} - Q_i^{j+1}}{\Delta x_i}\right] - \theta\, q_i^{j+1} + (1-\theta)\left[\frac{Q_{i+1}^j - Q_i^j}{\Delta x_i}\right] - (1-\theta) q_i^j \quad +$$

$$\left[\frac{S_{c_i}^{j+1}(A+A_o)_i^{j+1} + s_{c_i}^{j+1}(A+A_o)_{i+1}^{j+1} - s_{c_i}^j(A+A_o)_i^j - s_{c_i}^j(A+A_o)_{i+1}^j}{2\Delta t_j}\right] = 0.... \qquad (9)$$

$$\left(\frac{(Q_i)^{j+1} + (Q_{i+1})^{j+1} - (Q_i)^j - (Q_{i+1})^j}{2\Delta t_j}\right) + \theta\left[\begin{array}{l}\dfrac{(\beta Q^2/A)_{i+1}^{j+1} - (\beta Q^2/A)_i^{j+1}}{\Delta x_1} \\ + g\,\overline{A}^{j+1}\left(\dfrac{h_{i+1}^{j+1} - h_i^{j+1}}{\Delta x_i} + \overline{S}_f^{j+1} + S_e^{j+1} + \overline{S}_i^{j+1}\right)\end{array}\right] + (1-\theta)$$

$$\left[\frac{(\beta Q^2/A)_{i+1}^j - (\beta Q^2/A)_i^j}{\Delta x_i} + g\,\overline{A}^j\left(\frac{h_{i+1}^j - h_i^j}{\Delta x_i} + \overline{S}_f^j + S_e^j + \overline{S}_i^j\right)\right] = 0 \qquad (10)$$

In the above Eqs (9-10), terms with over bars represent the average quantities between i and i+1 nodes.

Eqs. (9-10) cannot be solved in an explicit manner for the unkowns, as there are four unknowns and only two equations. However, if Eqs (9-10) are applied to each of the (N-1) nodes between the upstream and downstream boundaries, a total of (2N-2) equations with 2N unknowns can be formulated. The required two additional equations, for the system to be determinate, are obtained using boundary conditions. The resulting system of 2N nonlinear equations with 2N unknowns is solved by a functional iterative procedure, the Newton-Raphson method (Anderson et al. 1984).

Upstream boundary condition:

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

$$Q_1 = QI(t) \tag{11}$$

In which Q_1 is the flow at section 1 (the most upstream cross section), and QI(t), are specified at either constant or variable time intervals. For a required time level, discharges are linearly interpolated from the table of discharge versus time.

Downstream boundary conditions:

In the most downstream cross section, a known relationship between flow and depth, for each time level, must be specified. Depending on the physical characteristics of the down stream section, this model allows the appropriate specification of one of the following four downstream boundary equations, (1) single-valued rating, (2) generated dynamic loop rating, (3) critical flow rating and (4) water level time series.

Initial conditions:

In order to solve the unsteady flow equations, the flow variables (h and Q) must be known at all cross sections at the beginning (t=0) of the simulation. This is known as the initial condition of the flow. The model assumes the initial flow to be steady and non-uniform. The flow at each cross section is initially computed as:

$$Q_i = Q_{i-1} + q_{i-1}\Delta x_{i-1} \qquad (12)$$

using the known steady discharge at the upstream boundary of the channel. 'q_i' is the specified lateral inflow, at t=0, from tributaries existing between the specified cross sections spaced at intervals of Δx along the valley. The flow depths are computed by solving the steady state gradually varied flow equation.

Stability:

In this model, an implicit formulation is used. For θ is greater than 0.55, this scheme is unconditionally stable. However, to reduce large truncation errors, time step 'Δt' is computed by the model using the following equation.

$$\Delta t = t_r / M \qquad (13)$$

where, t_r is the time of rise of specified hydrogaph and M = 20 till the breach is just about to begin. Thereafter, Δt is calculated by

$$\Delta t = \tau/M \qquad (14)$$

where, 'τ' is breach time and M = 20.

Limitations to the Model: ?

There are some limitations to the assumptions which are listed below.

1. Flow is one – dimensional.
2. The channel boundaries are rigid, i e. cross sections do not change shape due to scour/deposition.
3. A constant value of Manning's roughness co-efficient is assumed for unsteady flow.

4. Breach parameters, such as side slope of breach, breach width and time of breach do not represent the exact breach phenomenon.
5. The model excludes uncertainty associated with the volume losses due to infiltration and detention.

Data requirement:

In this model, the input data requirement has been categorized into two groups. The first data group pertains to the dam like the breach, spillways and physical characteristics of the reservoir. The breach data consists of: (i) time of breach formation, (ii) final bottom breach width, (iii) side slope of breach, (iv) final elevation of breach bottom initial elevation of water level in the reservoir, (v) initial elevation of water level in the reservoir, (vi) elevation of water when breach begins to form, and (vii) top elevation of dam.

The required spillway data are:

i) elevation of uncontrolled spillway creast,

ii) co-efficient of discharge of uncontrolled spillway,

iii) elevation of centre of submerged gated spillway,

iv) co-efficient of discharge of crest of dam, and

v) constant head independent discharge from spillway/dam.

The reservoir data consists of a table describing storage features of the reservoir i.e. surface area or volume versus elevation.

The second group of data pertains to the routing of the outflow hydrograph through the downstream channel. These are;

i) inflow hydrograph ordinates and corresponding time interval to the reservoir,

ii) total computation time,

iii) cross-section indicating varation of top width against elevation,

iv) Manning's roughness coefficient (n),

v) expansion – contraction coefficient bewteen cross sections,

vi) sinuosity factors, and

vii) minimum computational distance (Δx).

The mathematical model described above is used in this work to study the dam break flow of the Hirakud Dam due to a hypothetical failure case. Before presenting the result of this work, the study area and data availability are presented in the next chapter.

STUDY AREA

3.1 THE MAHANADI RIVER BASIN SYSTEM

The Mahanadi River is one of the major rivers of the country flowing east and draining into Bay of Bengal. Among the Peninsular rivers, it ranks second to the Godavari in terms of its water potential and flood producing capacity. The river originates at a distance of 6 kms from Pharsiya village near Nagri town in Raipur district of Madhya Pradesh.

Total length of the river from the head to its outfall into the sea is 851 Km out of which 357 km of length are in Madhya Pradesh and 494 km are in Orissa. The principal tributaries of the river are, the Seonath, the Koonk, the Hasdeo, the Mand, the IB, the Ong and the Tel river. A basin map of the Mahanadi river system showing the details is given in Fig. 3 . Total catchment area of Mahanadi river basin is 1,41,720 square km and at Hirakud it is 83400 square km. It covers four states namely Madhya Pradesh, Orissa, Maharashtra and Bihar, having the distribution of 73138 sq.Km, 65770 sq.Km., 238 sq.Km and 634 sq.km, of drainage area respectively.

3.2 HIRAKUD DAM

The objectives of the study reported in this dissertation is to examine the various aspects of dam break flow analysis with reference to multipurpose Hirakud dam project in Orissa, one of the pioneer achievements of the independent India which is completed in the year 1955-56.

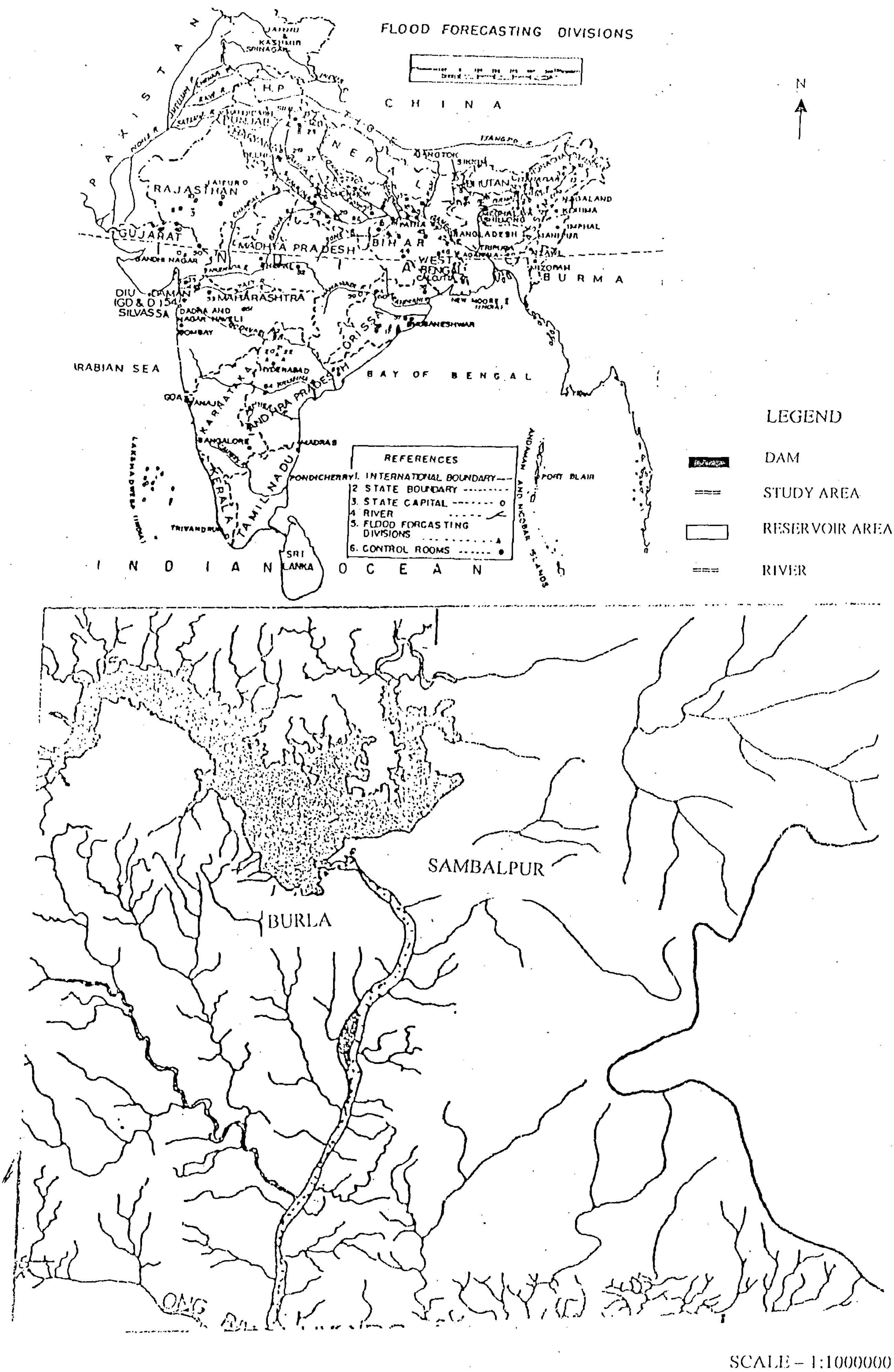


Fig. 3 : Index Map Showing Study Area

Hirakud dam consists of 4800 m long main dam of earth, concrete and masonry portion being flanked by earthen dykes in both left and right sides. The main dam from right to left is as below.

Types

1. Masonry Power dam-gravity type = 289 m
2. Right concrete spillway = 360 m
3. Earth dam of zoned filled type = 2298 m
4. Left concrete spillway = 500 m
5. Earth dam of zoned filled type in left = 1353 m

The maximum height of the dam in concrete portion is 61 m and that in earth portion is 59.44 m. The provision of 64 sluices of size (3.66 x 6.2) m controlled by vertical slide gates operated from on operation gallery size 3.35 m x 6.10 m along the body of the dam is a special feature of this project. There are 21 crest bays in the left spillway having 15.54 wide and 6.1 m high radial gates.

The other salient features of the project are listed in Appendix-1.

3.3 STUDY AREA

In the present dissertation work, a study area of length 40 km along the river Mahanadi, from downstream of Hirakud dam, has been considered. The flood plain area on both side of the river are also taken into account. The width of the river near the dam site is nearly a kilometer. It ranges from 1 to 4.6 kms at downstream locations. Average bed slope of the river in this area is 1 in 3000. The general soil characteristics of this area consitute rocky, boulder mixed, red soil with clay base. In adjacent to the dam site, two towns Burla and Hirakud, are situated. A district headquaters i.e. Sambalpur town is placed at a distance of 16 km from the dam site. Besides, there are small developed villages situated in this region . The inhabitants of local area are mainly dependent on agriculture. The agricultural land is so fertile that the quantum of food produced from the nearby area makes the state self-sufficiency.

3.4 DATA AVAILABILITY

As per the requirement of this work, data have been collected from various sources. These are listed in Table 4. However, no data is available for bed roughness of the study area. The breach parameters are also not available as Hirakud dam has never failed.

Table 4 : Available data for the dam break study of Hirakud dam

Sl. No.	Data	Source
1.	Surface area and elevation for the reservoir	Chief Engineer, Designs & Dam Safety, Bhubaneswar
2.	Spillway rating curve	-do-
3.	Dam elevations	-do-
4.	Length of dam	-do-
5.	Spillway length	-do-
6,	Spillway elevations	-do-
7.	Cross-section of Mahanadi river d/s of Hirakud	-do-
8.	Flow measurements at Tikarpara and Naraj for one flood event	NWDA, Bhubneswar
9	Contour map of study area	Survey of India, Topo Sheets (1:50,000)

In the next chapter the results of the present study are described.

RESULTS AND DISCUSSION

In this chapter, the results of the present study i.e. "Dam break flow analysis of Hirakud Dam", due to its hypothetical failure are presented. For the purpose of presentation, the results are described in four parts; (i) outflow hydrographs at downstream locations, due to dam break flow, (ii) time taken for the flood wave to reach downstream locations and duration of flooding, (iii) sensitivity analysis showing effects of bed roughness and various breach parameters, and (iv) flood inundation map due to a worst scenario case. Also presented are the input data used in the programme, calibration of model parameters and routing of design flood through the spillway.

4.1 INPUT DATA

The data requirement for the mathematical model used in this study and data availability for the study area have been presented in Chapter 2 and 3 respectively. In this section the used data, both available and assumed, are presented. A case of hypothetical dam break is considered in the present study and the flood due to this is routed upto 40 km downstream of dam. The inflow hydrograph to the reservoir is assumed (Fig. 4). In this figure the recession limb of the hydrograph is shown. It is assumed that the rising limb of the hydrograph results in filling the reservoir upto top of dam. It may be noted that, the design discharge for the spillway is 42450 cumecs and the peak discharge in the inflow hydrograph is 60,000 cumecs. The area-elevation relationship for the reservoir is presented in Fig. 5. Although the capacity of the reservoir (volume) corresponding to

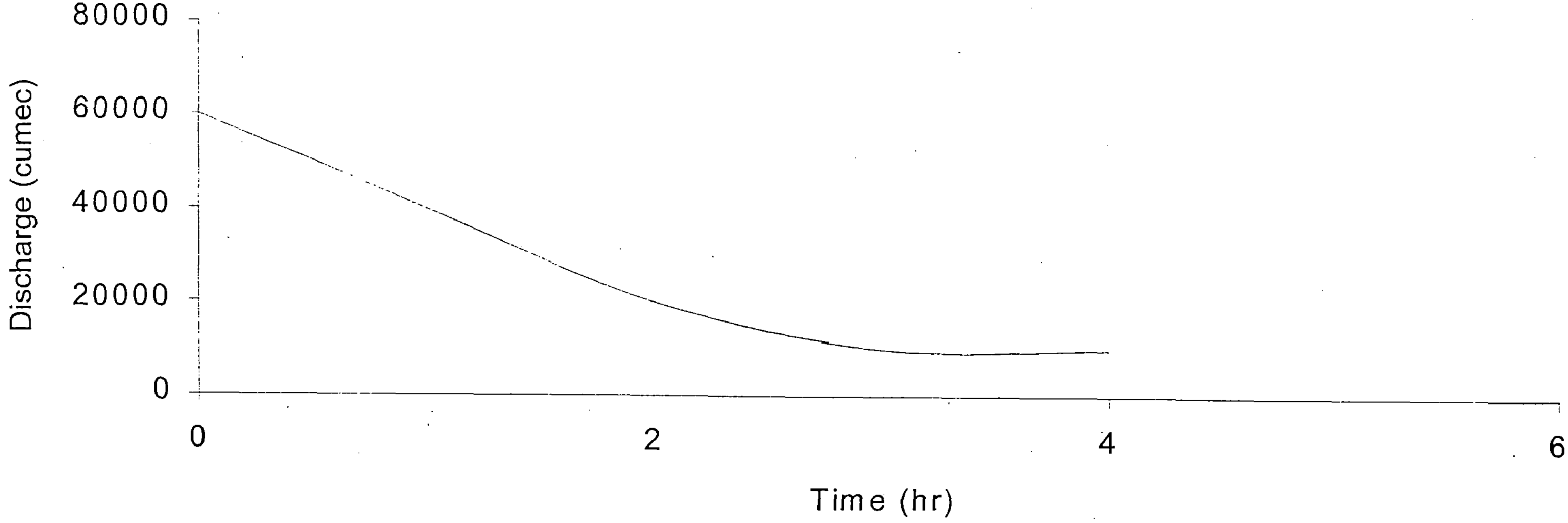


Figure 4 : Assumed inflow hydrograph to the reservoir for hypothetical dam break of Hirakud Dam

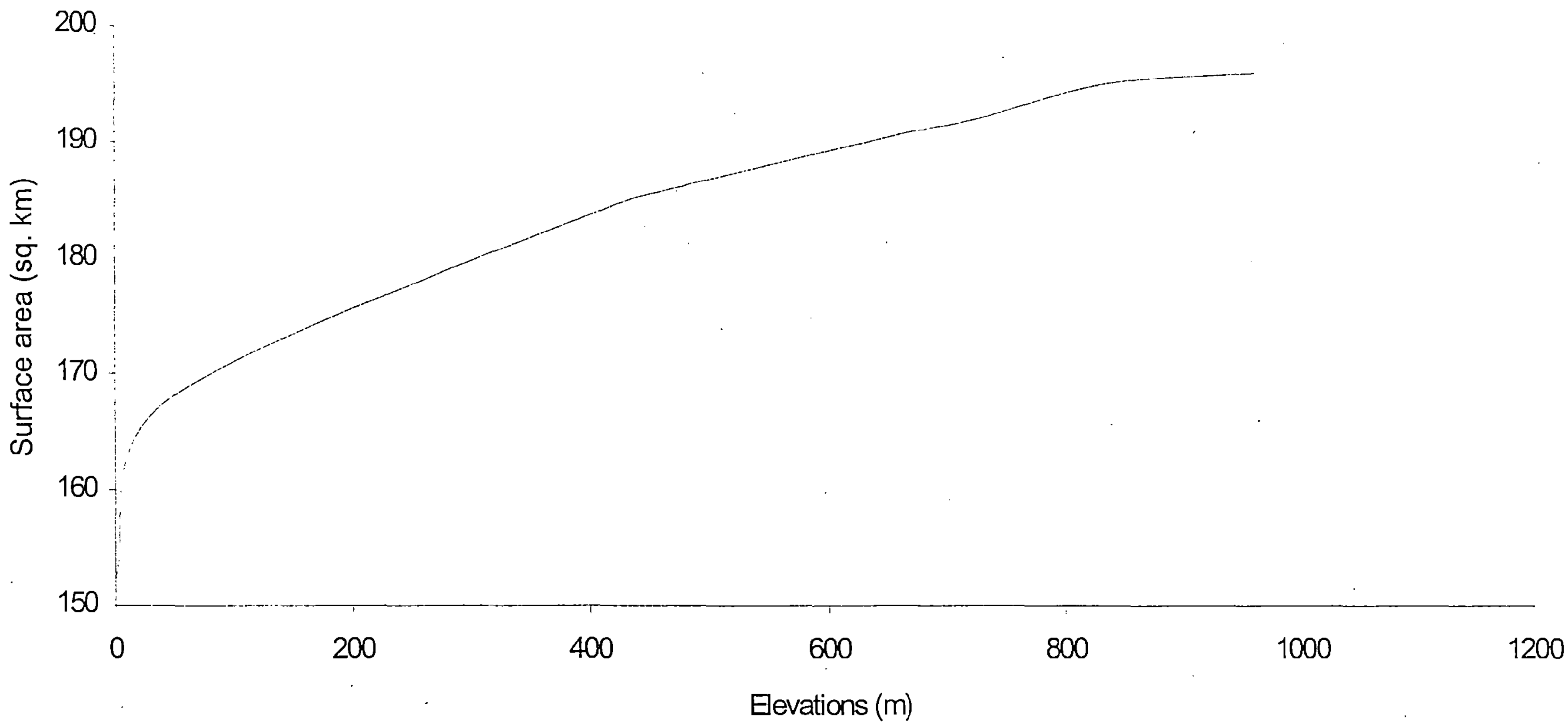


Figure 5: Surface area - elevation for Hirakud reservoir

different elevations are available for a more accurate computation, the surface area elevation relationship of the reservoir is provided to the model as input. The discharge through the spillway is shown as a rating curve in Fig. 6. In this figure, x-axis represents the discharge in cumecs and y-axis the head in metre over spillway.

The dam is assumed to break by overtopping failure. The breach parameters are assumed and the used values are : breach width = 250 m, time to breach =1 hr, and side slope of breach = 0 which corresponds to a vertical breach section. The final level of breach corresponds to the channel bed level (152.40 m) at dam site. The deepest bed levels along the river, downstream of the dam, are presented in Fig. 7. The cross sections covering the flood plain area, located at nine different locations are presented in Fig. 8.(a-i). In these figures, widths (m) and elevations (m) are shown in the x and y axis respectively. 'O' in the x-axis represents the ground level at the left bank of Mahanadi river. These cross sections are obtained from different project documents and survey of India topo sheets. The bed roughness coefficients are 0.035 and 0.050 for the main river and for the flood plains, respectively. The computational distance step sizes (Δ x) are different for different reaches. An input data file used for the computer programme is given in Appendix –2.

4.2 CALIBRATION

The model parameters i.e. bed roughness 'n' and distance step sizes 'Δx' are calibrated before applying the model for dam break flow analysis.

4.2.1 Determination of Bed Roughness

Due to non availability of data for the study area, observed values of discharges for one event at Tikarpara and Naraj are used to determine 'n'. It may be noted here that,

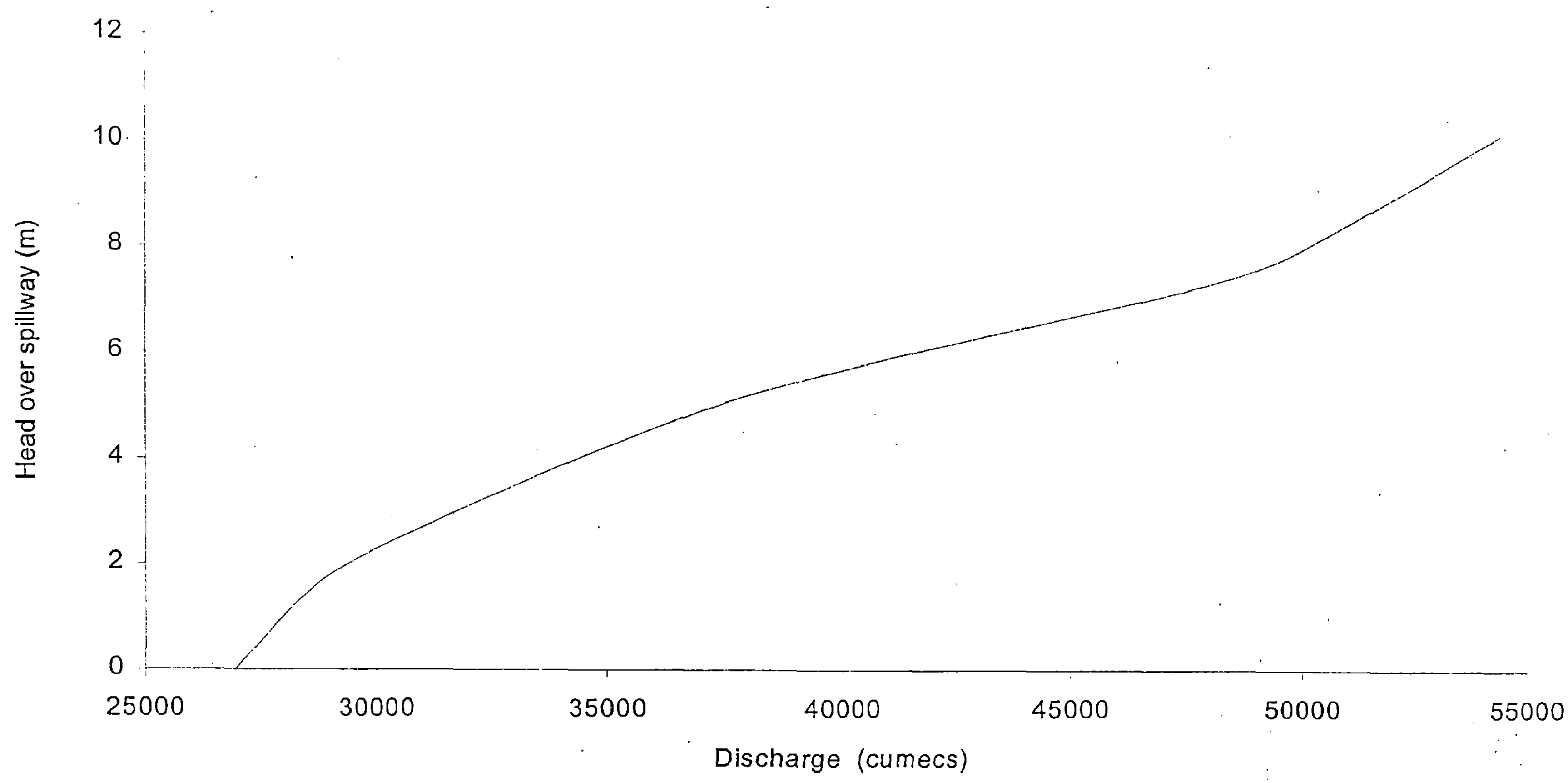


Figure 6: Spillway rating curve for Hirakud dam

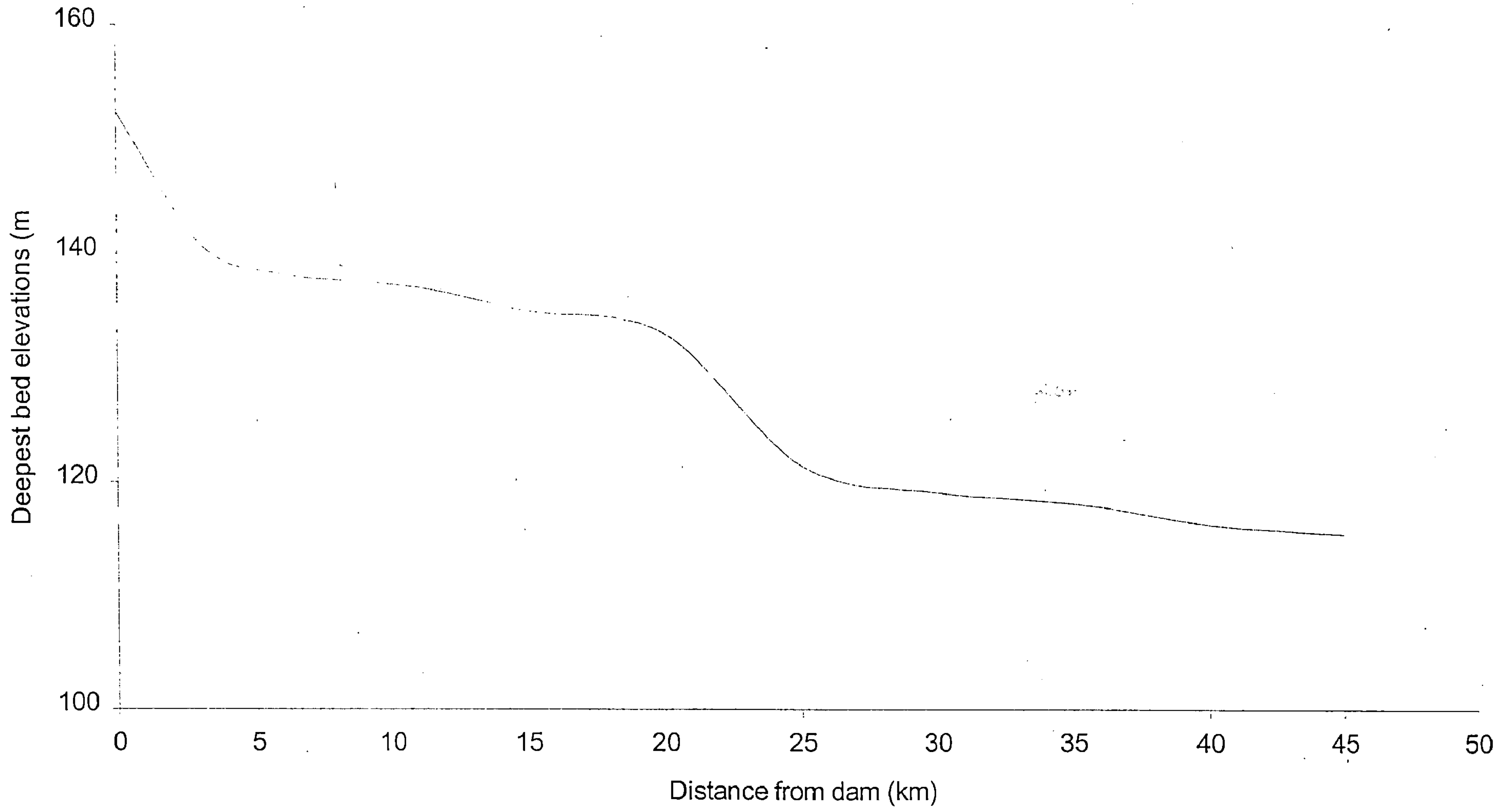
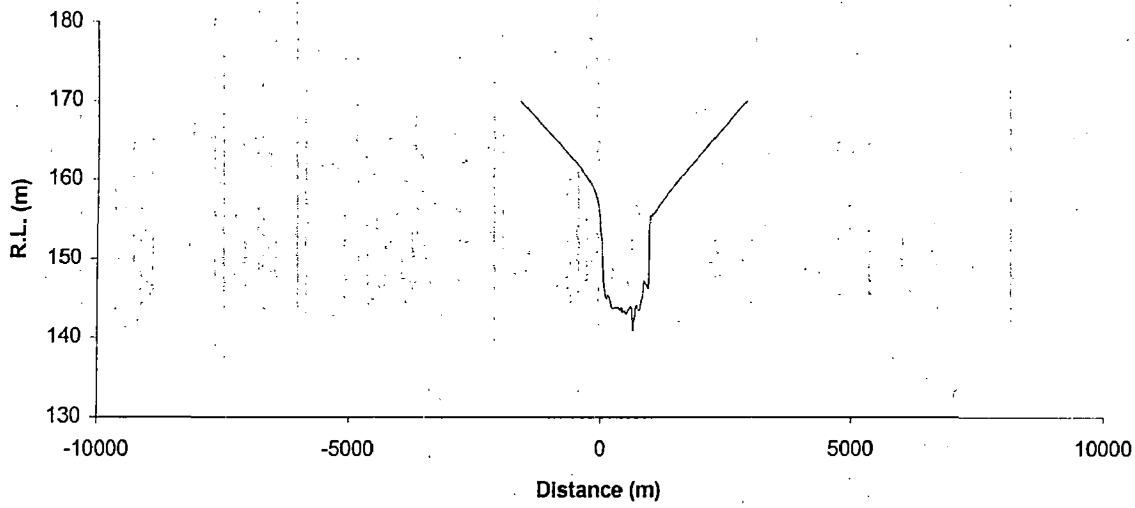
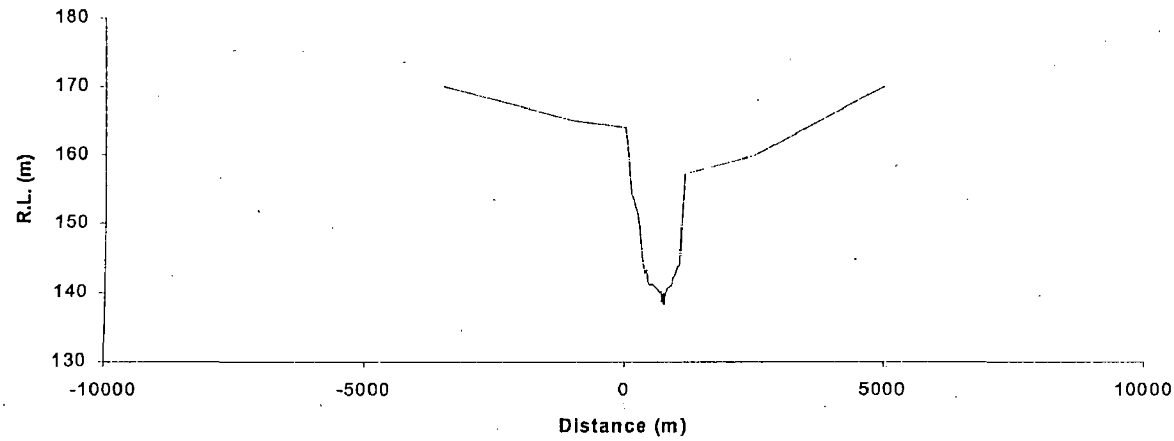


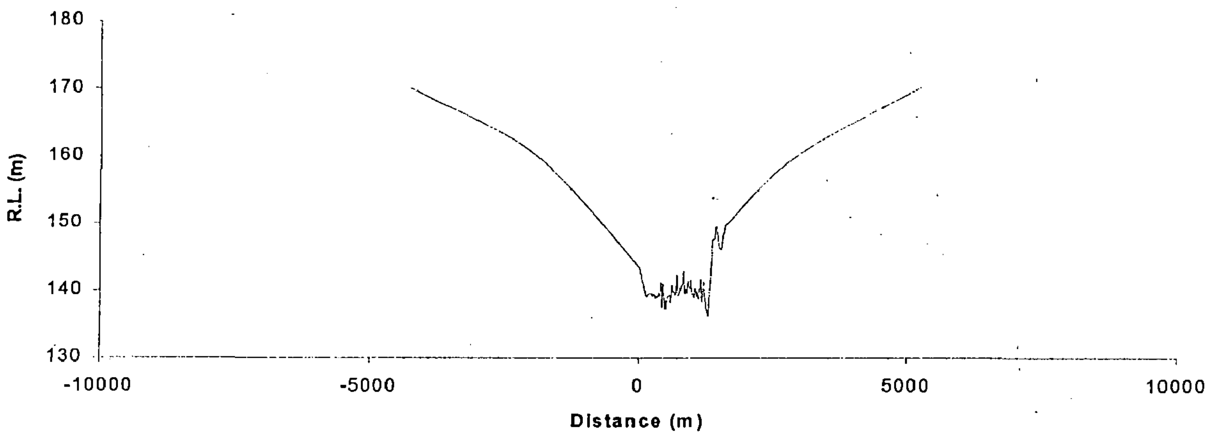
Figure 7: Deepest bed elevations downstream of Hirakud dam



a: At 3 km

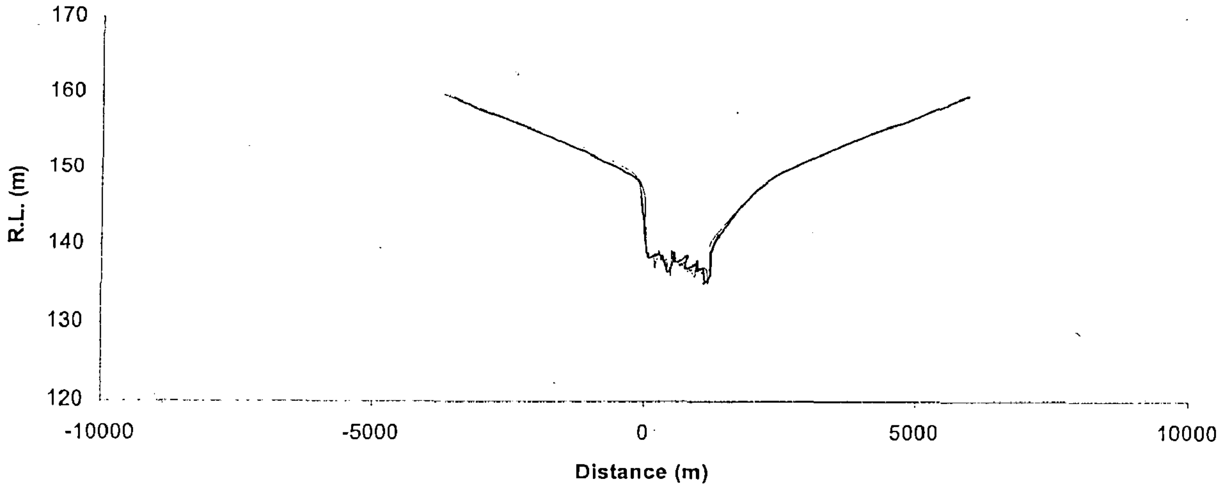


b: At 6 km

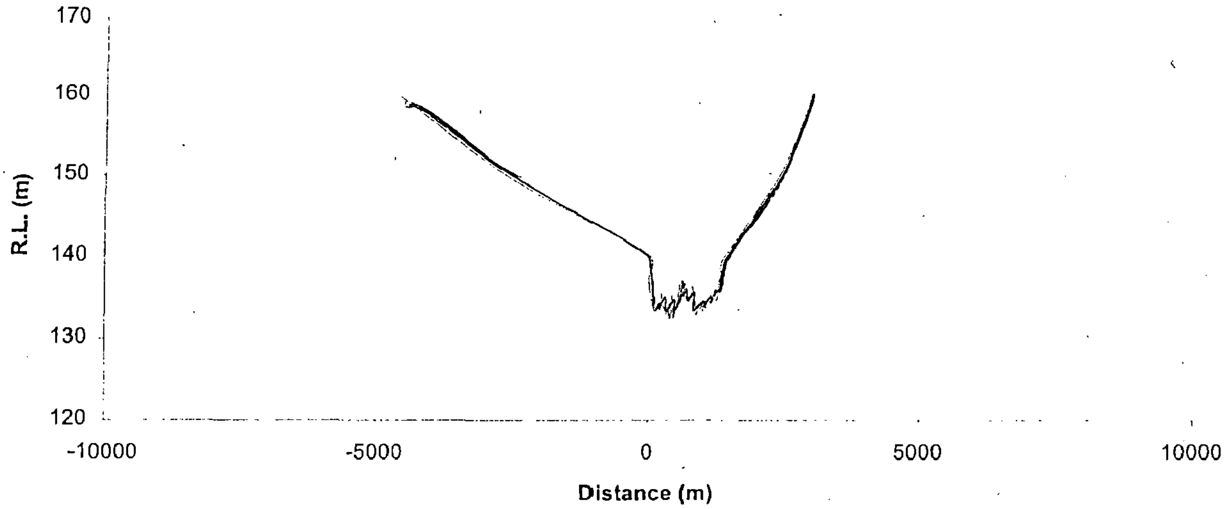


c: At 11 km

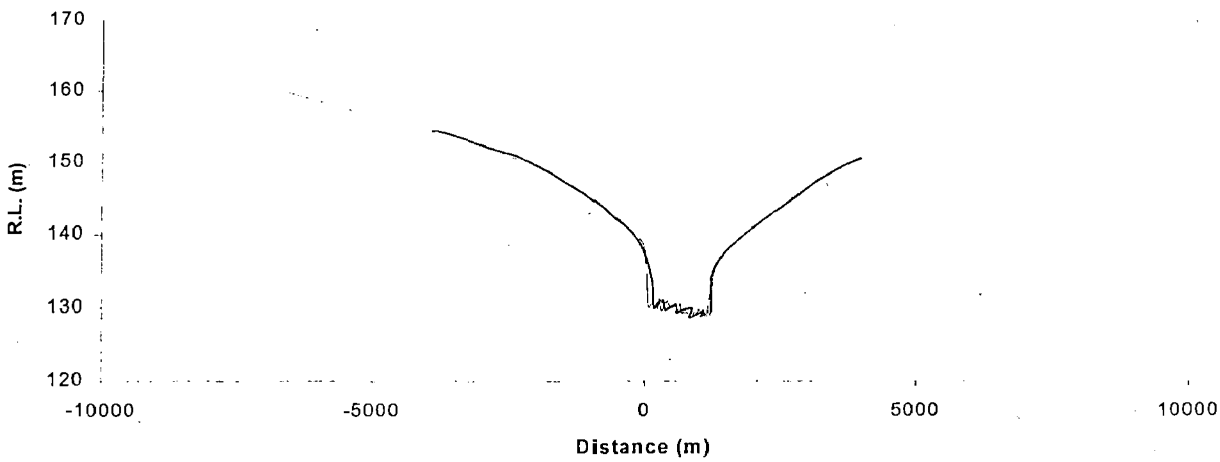
Figure 8: Cross sections downstream of Hirakud dam (contd.)



d: At 15 km

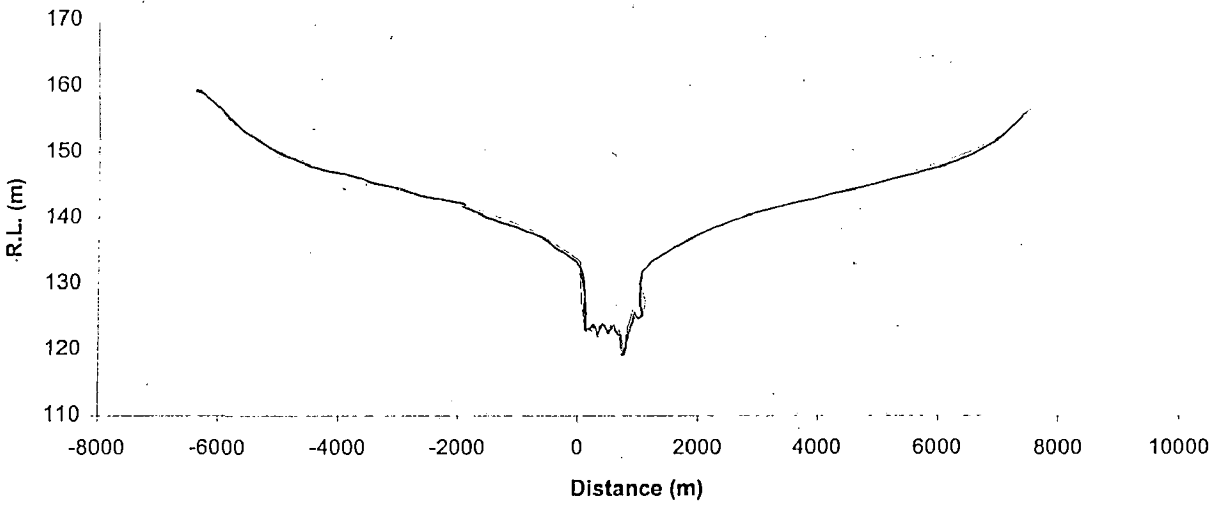


e: At 20 km

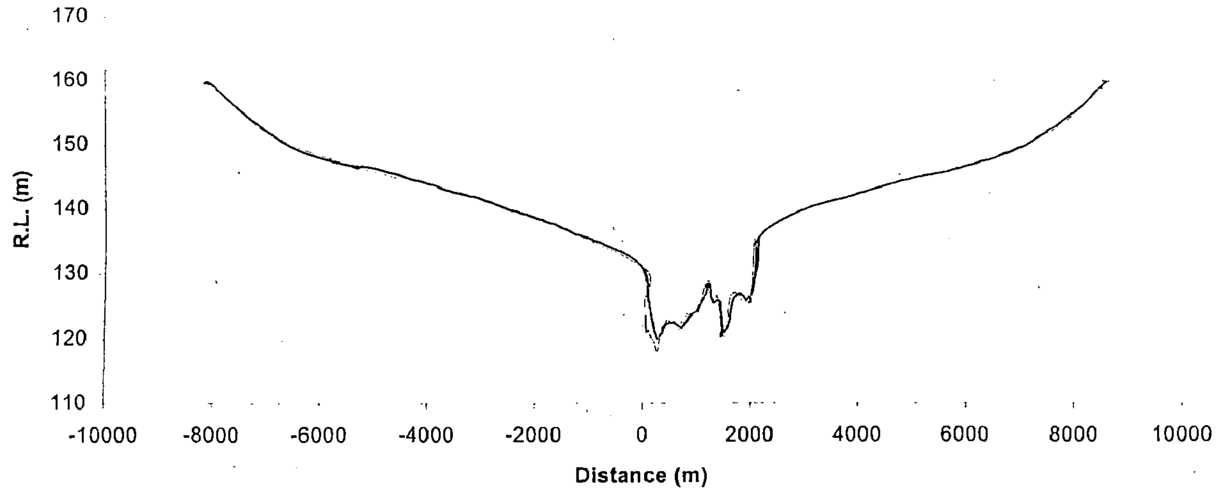


f: At 25 km

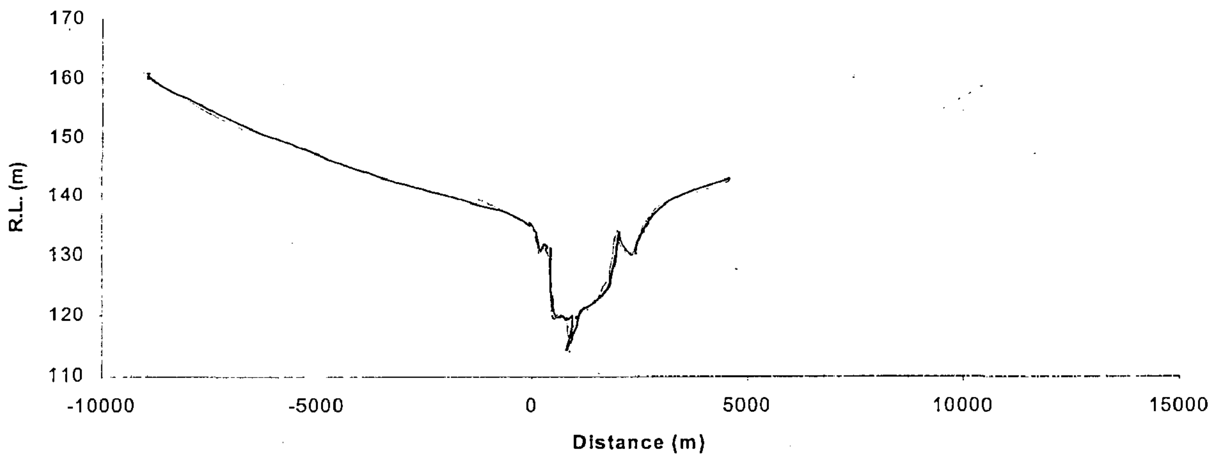
Figure 8: Cross sections downstream of Hirakud dam (contd.)



g: At 30 km



h: At 35 km



i: At 40 km

Figure 8: Cross sections downstream of Hirakud dam

Tikarpara and Naraj are situated at distances of 200 km and 310 km from the dam site, respectively. In the observed values, the discharge at Naraj (downstream) is always higher than that of Tikarpara (upstream). This may be due to contribution form tributaries located in between the reaches. An approximate method to take into account the effect of tributaries' flow is used here. First, the difference between the observed hydrograph is calculated. Half of this amount is added to the observed hydrograph at Tikarpara. This modified hydrograph at Tikarpara is routed upto Naraj using the mathematical model. The computed hydrograph at Naraj is again modified by adding the half of the difference of hydrographs obtained earlier. Different 'n' values are used and the best match between the observed and the computed hydrographs at Naraj is found to be in case of 'n' i.e. n = 0.035. In Fig. 9, the computed discharge hydrograph obtained using DAMBRK at Naraj is shown. In the same figure, the observed values are also presented. The match between the two is found to be satisfactory.

4.2.2 Computational Distance Step (Δ x)

The results of any numerical study depend on the computational step size (Δ x). In the present case, the model is executed for the study reach, using different 'Δ x' values (0.25, 1, 5 km). It is observed form the present numerical studies that the effect of 'Δ x' is very nominal on the discharge computation (Table 5). In the rest of the studies, different Δ x values as defined in the input file (Appendix 2) are used. This is due to the wave celerity criterion and/or due to expansion/contraction criteria. These values are suggested by the model at the time of test runs.

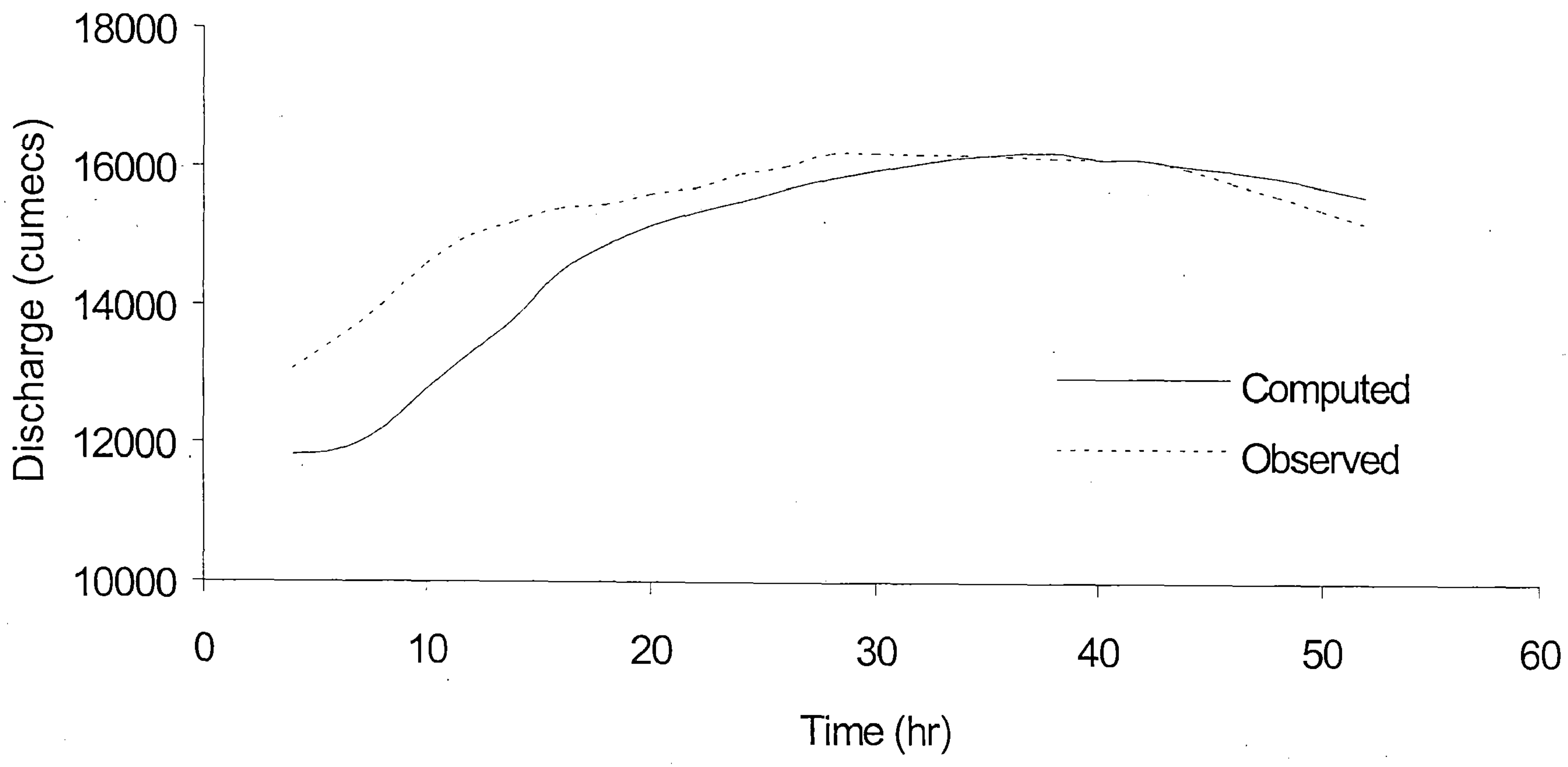


Figure 9: Computed and Observed hydrographs at Naraj (n= 0.035)

Table 5: Effect of 'Δx' on computed hydrograph at distance 40 km from Hirakud dam

Time (hr)	Discharge (cumecs)		
	$\Delta x = 0.25$ km	$\Delta x = 1$ km	$\Delta x = 5$ km
0	54492	54492	54492
1	54492	54492	55186
2	60921	60871	59707
3	84074	83810	82862
4	114090	114388	117161
5	134032	134073	134421
6	134007	134027	133948
7	131007	131065	130776
8	126918	126896	126689
9	122428	122455	122340
10	117925	117913	118043
11	114530	114338	114296
12	111144	110894	111417
13	107841	107747	108077
14	104704	104636	104228
15	101695	101667	100940
16	98822	98789	98160
17	96106	96093	95467
18	93566	93560	92945
19	91233	91224	90713
20	89055	89061	88817
21	86230	86220	86332
22	80456	80446	80696
23	72862	72900	73135
24	66450	66486	66805

4.3 ROUTING OF DESIGN DISCHARGE

Before considering the dam break flow, a case is considered for the design discharge through the spillway. This flow is routed in the study area i.e. upto 40 km downstream of Hirakud dam. The reservoir water level for this case is assumed to be at F.R.L. i.e. 192.02 m. The result of computations using DAMBRK is presented in Fig. 10. In this figure, maximum water elevations attained at different locations downstream of dam are shown. The inundation map due to this design flood is shown in Fig 17, as dotted line.

4.4 DAM BREAK FLOW

The data described in section 4.1 are used as input to study the dam break flow. Different results obtained from the output of the computer programme, are described below.

The reservoir depletion table due to the dam break flow is given in Table 6. The resulting hydrograph due to the hypothetical dam break at the dam site is presented in Fig. 11. In the same figure, outflow hydrographs at distances of 20 km and 40 km are also shown. The peak discharge at the dam site is 144565 cumecs and it gradually decreases to 101648 cumecs at a distance of 40 kms. As seen in Fig. 11, the peak discharge decreases and time to peak discharge increases as the dam break flood moves downstream. This indicates the general characteristics of a flood wave propagation. The stage hydrographs, for this dam break study, at dam site, 20 km, and 40 km are presented in Fig. 12. In Fig. 13, results of this case, for maximum water elevation are presented. A summary of results for this case is given in Table 7.

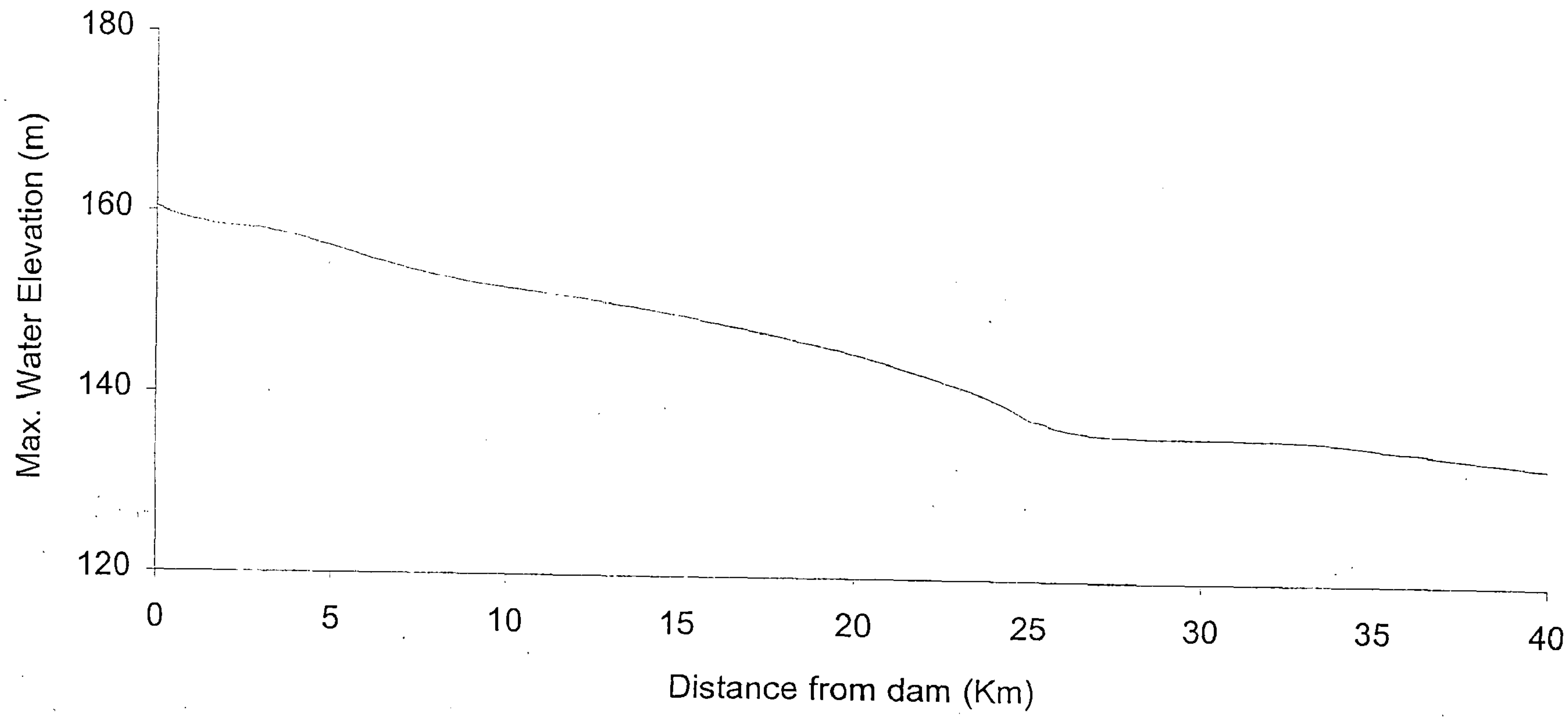


Figure 10: Maximum water elevation downstream of Hirakud Dam due to design discharge from the spillway

Table 6: Reservoir Depletion table due to dam break flow of Hirakud dam

I	K	TTP(I)	Q(I)	H2	YB	D	SUB	VCOR	OUTVOL	BB	COFR	QI(I)	QBRECH	QSPIL
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
1	0	0	54492	195.68	195.68	160.15	1	1	0	0	3.1	60000	0	54492
2	1	1	144564	195.56	160	167.1	1	1	288	250	3.1	40000	91118	53447
3	1	2	141842	195.09	160	166.6	1	1	803.8	250	3.1	20000	89348	52495
4	2	3	138591	194.53	160	165.95	1	1	1320.3	250	3.1	10000	87231	51360
5	2	4	135267	194	160	165.76	1	1	1785.5	250	3.1	10000	85222	50046
6	2	5	131742	193.43	160	165.55	1	1	2268.9	250	3.1	10000	83103	48640
7	2	6	127030	192.84	160	165.26	1	1	2758.4	250	3.1	10000	80920	46111
8	2	7	122840	192.35	160	165.01	1	1	3157.3	250	3.1	10000	79113	43728
9	2	8	117856	191.76	160	164.7	1	1	3622.1	250	3.1	10000	76972	40884
10	2	9	113538	191.21	160	164.42	1	1	4045.18	250	3.1	10000	74972	38566
11	2	10	109641	190.65	160	164.17	1	1	4459	250	3.1	10000	72970	36672
12	2	11	106090	190.13	160	163.94	1	1.01	4835.21	250	3.1	10000	71111	34980
13	2	12	102016	189.47	160	163.68	1	1.01	5304.55	250	3.1	10000	68780	33236
14	2	13	98720	188.93	160	163.46	1	1.01	5682	250	3.1	10000	66902	31817
15	2	14	97015	188.65	160	163.34	1	1.01	5876.6	250	3.1	10000	65933	31082
16	2	15	93604	188.04	160	163.11	1	1.01	6290	250	3.1	10000	63871	29734
17	2	16	90217	187.4	160	162.87	1	1.01	6728.5	250	3.1	10000	61680	28537
18	2	17	87118	186.7	160	162.66	1	1.01	7193.9	250	3.1	10000	59351	27768
19	2	18	85764	186.39	160	162.56	1	1.01	7396	250	3.1	10000	58337	27428
20	2	19	84411	186.08	160	162.46	1	1.01	7598	250	3.1	10000	57323	27088
21	2	20	70936	185.72	160	161.42	1	1	7838	250	3.1	10000	56144	0
22	2	21	57461	185.36	160	160.37	1	1	8078	250	3.1	10000	54965	0
23	2	22	55128	185.05	160	160.18	1	1	8290	250	3.1	10000	53880	0
24	2	23	52795	184.74	160	160	1	1	8502.7	250	3.1	10000	52795	0
25	2	24	51976	184.48	160	159.92	1	1	8678	250	3.1	10000	51977	0
26	2	25	51157	184.22	160	159.85	1	1	8853	250	3.1	10000	51158	0

27	2	26	50320	183.95	160	159.78	1	1	8674	250	3.1	10000	50322	0
28	2	27	49484	183.69	160	159.7	1	1	8495	250	3.1	10000	49485	0
29	2	28	48676	183.43	160	159.61	1	1	8892	250	3.1	10000	48676	0
30	2	29	47867	183.17	160	159.53	1	1	9289	250	3.1	10000	47868	0
31	2	30	47059	182.91	160	159.44	1	1	9685.93	250	3.1	10000	47059	0
32	2	31	46280	182.63	160	159.39	1	1	9867	250	3.1	10000	46280	0
33	2	32	45501	182.35	160	159.34	1	1	10054	250	3.1	10000	45501	0
34	2	33	44722	182.14	160	159.28	1	1	10237.4	250	3.1	10000	44723	0
35	2	34	43970	181.89	160	159.21	1	1	10401	250	3.1	10000	43971	0
36	2	35	43218	181.64	160	159.14	1	1	10565	250	3.1	10000	43219	0
37	2	36	42467	181.39	160	159.07	1	1	10727.8	250	3.1	10000	42468	0
38	2	37	41821	181.17	160	159	1	1	10869	250	3.1	10000	41822	0
39	2	38	41175	180.95	160	158.94	1	1	110.11	250	3.1	10000	41177	0
40	2	39	40530	180.73	160	158.88	1	1	11152.3	250	3.1	10000	40531	0

N.B. I-Time step from start of analysis, K- Iteration necessary to solve flow equations, TTP(I)- Elapsed time from start of analysis(hr), Q(I)- Total outflow from dam (cms), H2-Elevation of water surface at dam (m), YB-Elevation of bottom of breach (m), D-Depth of flow immediately downstream (m), SUB-Submergence coefficient, VCOR-Velocity correction, OUTVOL- Total volume discharged from time of breach (M.Cum.), BB- Breach width (m), COFR- Rectangular breach discharge coefficient, QI(I)- Inflow to reservoir (cms), QBRECH-Breach outflow (cms), QSPIL- Spillway outflow (cms).

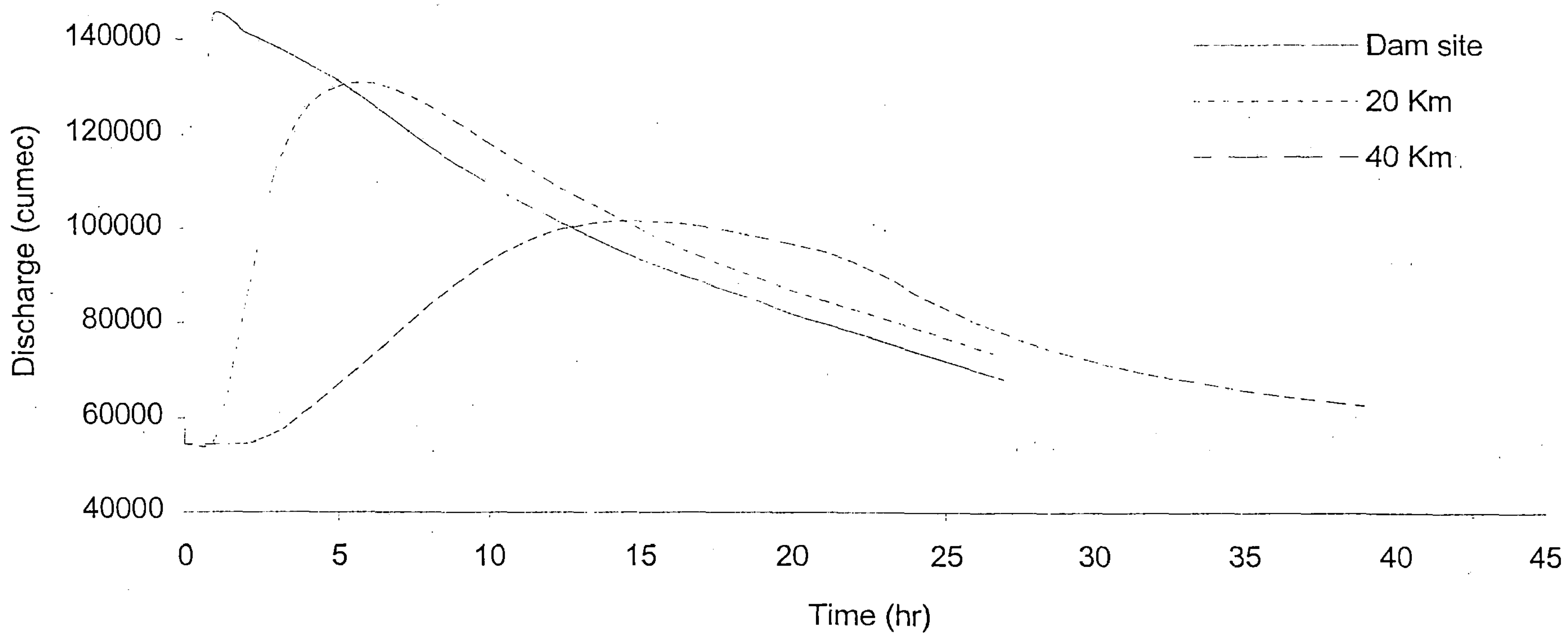


Figure 11: Outflow hydrographs at the dam-site, 20 Km and 40 Km, due to Dam Break of Hirakud Dam

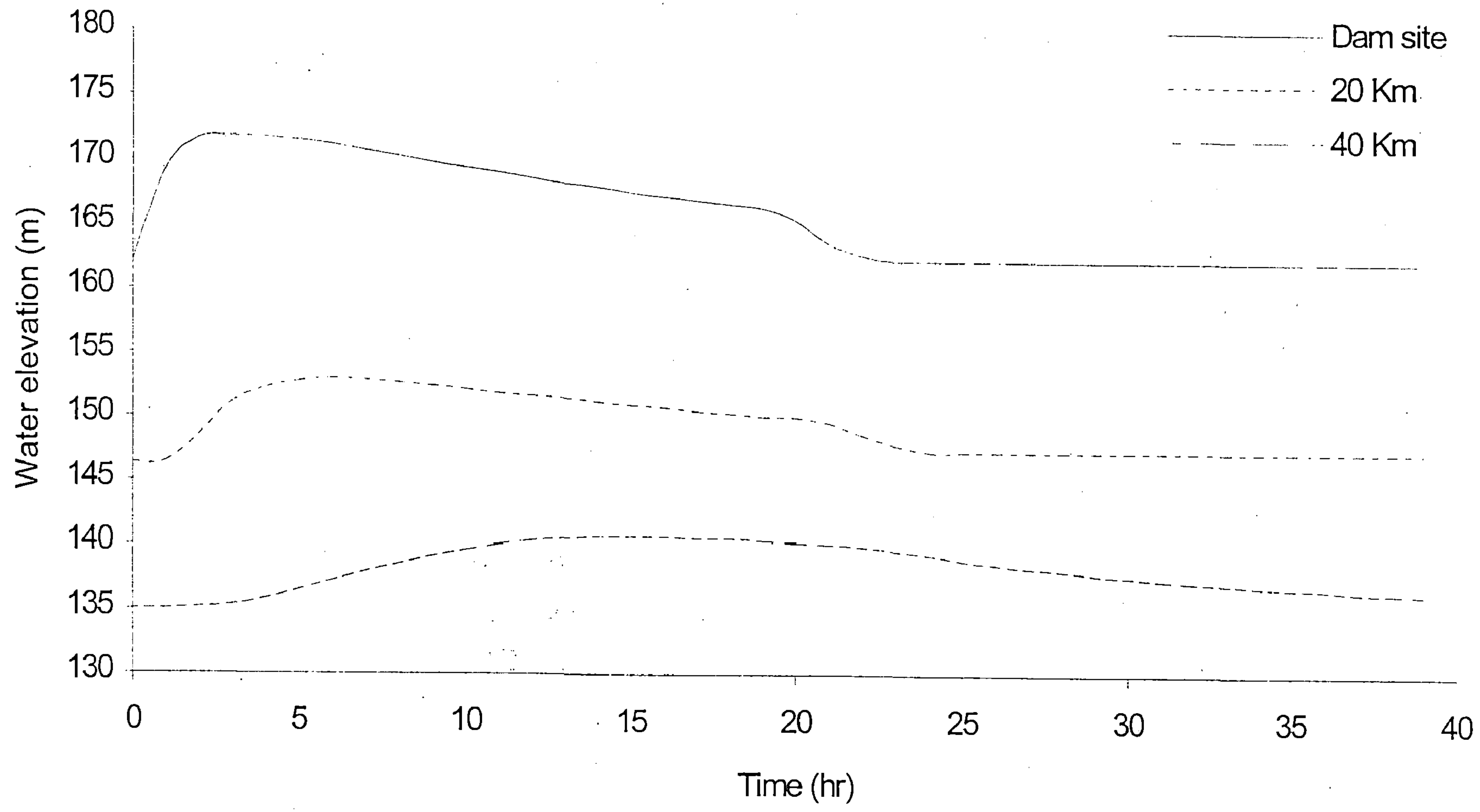


Figure 12: Stage hydrographs at dam site, 20 Km and 40 Km, due to dam break of Hirakud Dam

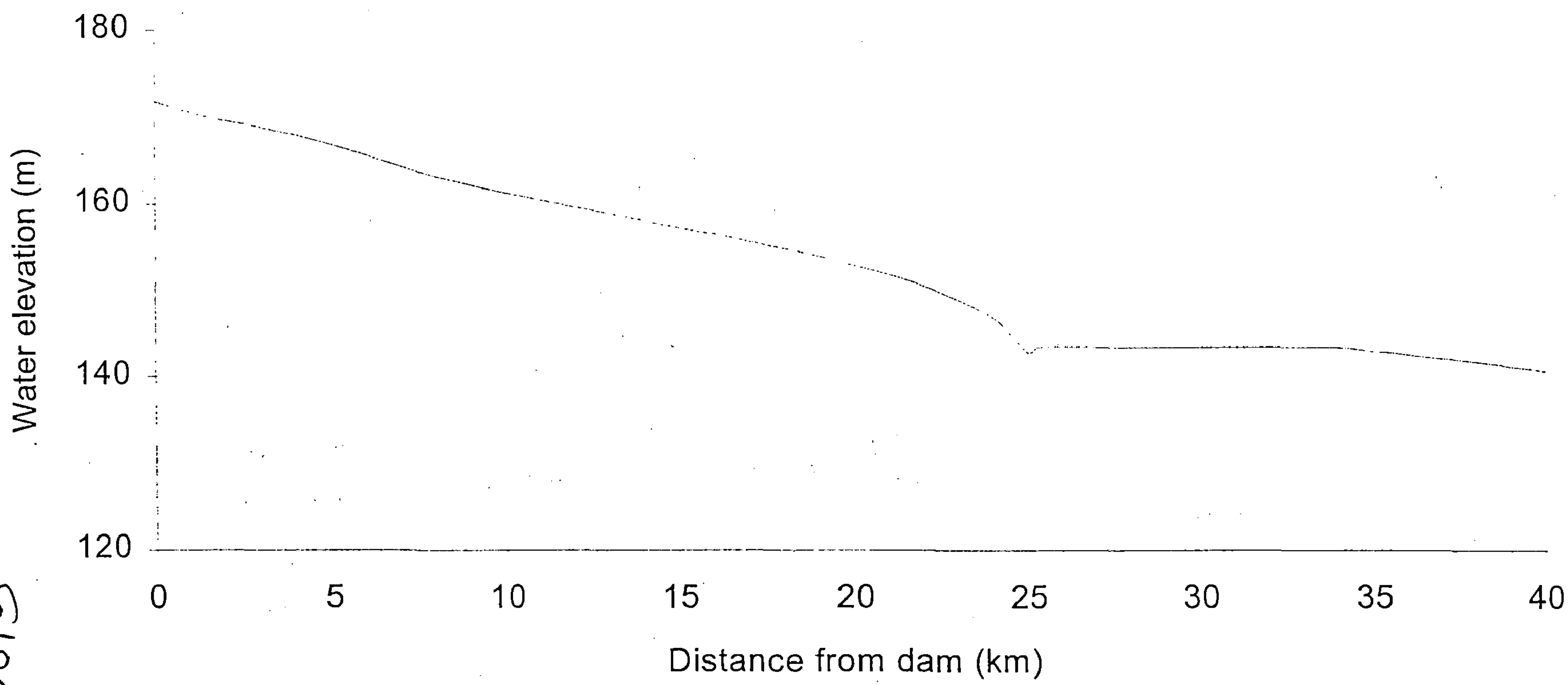


Figure 13: Maximum water elevation at different location due to dam break of Hirakud dam

Table 7: Summary of results for dam break flow

Distance from dam (km)	Peak discharge (cumecs)	Time to peak discharge (hr)	Max. water elevation reached by flood wave (m)	Time to max. water elevation (hr)
0	144565	1.0	171.83	3.0
20	130987	5.8	152.90	6.1
40	101648	14.825	140.65	15.275

4.5 SENSITIVITY ANALYSIS

In the used input data, many parameters are assumed. This may lead to uncertainties associated with the results. Therefore, a sensitivity analysis to study different model parameters is performed. In this dissertation work, breach width, time to breach and bed roughness coefficients are considered for sensitivity analysis of Hirakud dam break study.

4.5.1 Effect of Breach Width

The terminal width 'b' is related to the average width of breach ($\bar{b}$), by the following equation (ref. Fig. 2)

$$b = \bar{b} - Z h_d \tag{15}$$

The breach width and the breach time in an earthen dam depends an the material used and grade of compaction. However, it is observed that the average breach width ($\bar{b}$), generally, lies between h_d and 5 h_d.

Effect of different breach widths (250,150 & 50 m) on maximum water level elevation is presented in Fig. 14. An increase in breach width results in more discharge and high water level. A dip at 25 kms is due to the presence of a steep slope at that location.

4.5.2 Effect of Breach Time

Results are obtained using four different breach times (0.5, 1, 2 & 5 hr) for the dam break flow analysis. As the breach time increases, the maximum water levels decrease. This decrease in water level is nominal and the results are presented in Table 8. In Fig. 15, time to attain the maximum water level at different locations are presented.

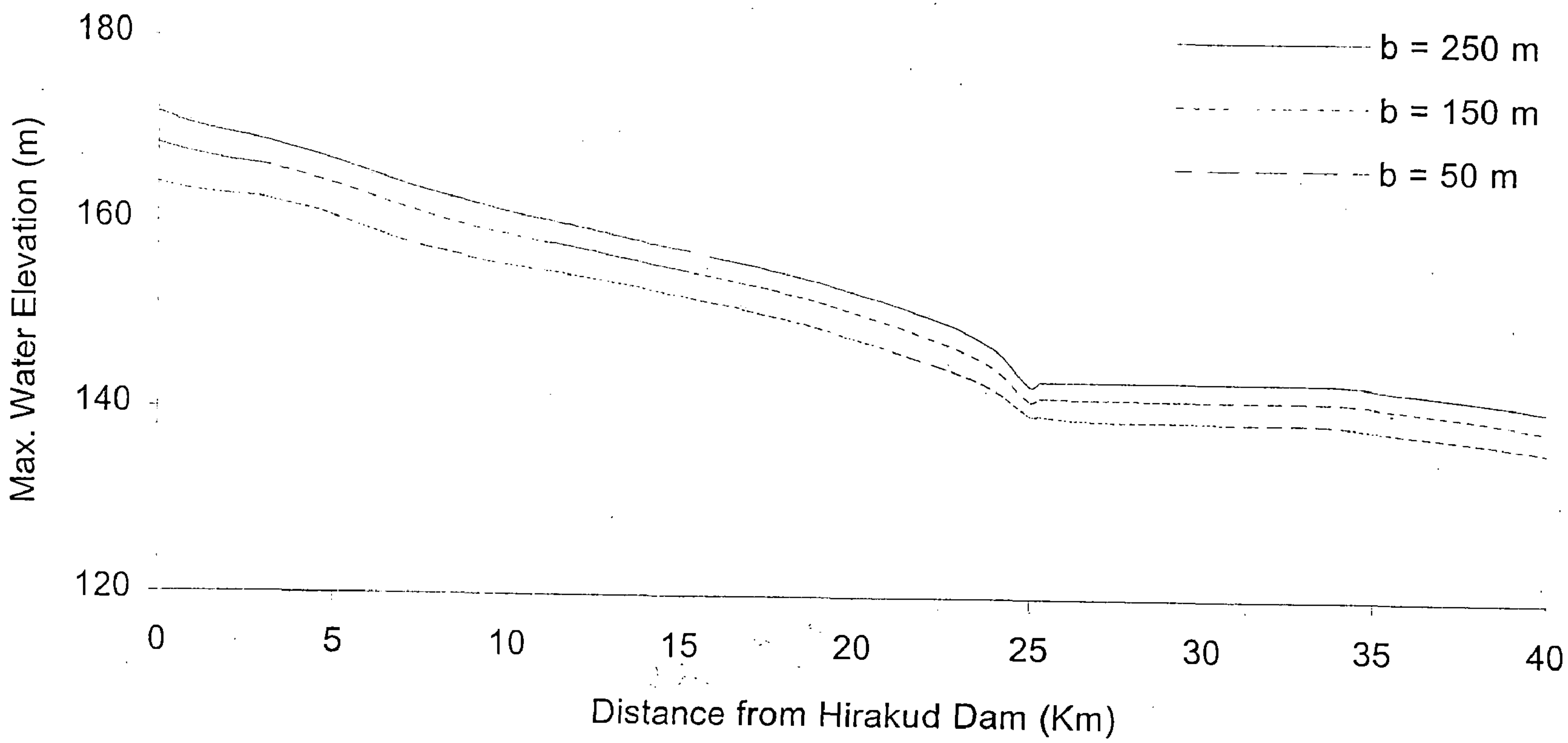


Figure 14: Effect of breach width on maximum water level elevation due to dam break flow

Table 8: Effect of breach time on maximum water levels at different locations

Distance (m)	Maximum water elevation (m) for different breach times (Bt) in hour			
	Bt = 0.5	Bt = 1.0	Bt = 2.0	Bt =5.0
0	171.87	171.83	171.74	171.41
0.99	170.58	170.55	170.46	170.14
1.98	169.66	169.63	169.55	169.25
2.999	168.88	168.85	168.78	168.51
3.999	167.92	167.89	167.82	167.55
4.999	166.85	166.81	166.74	166.48
5.999	165.59	165.56	165.48	165.22
6.832	164.55	164.51	164.44	164.17
7.665	163.58	163.55	163.47	163.2
8.498	162.69	162.66	162.59	162.31
9.331	161.89	161.86	161.78	161.51
10.164	161.16	161.13	161.06	160.79
10.998	160.52	160.49	160.42	160.15
12.088	159.65	159.62	159.55	159.29
12.997	158.92	158.89	158.82	158.56
14.088	158.05	158.02	157.96	157.7
14.997	157.35	157.33	157.26	157
15.997	156.57	156.54	156.47	156.22
16.996	155.75	155.72	155.66	155.4
17.996	154.89	154.86	154.79	154.55
18.996	153.96	153.93	153.87	153.62
19.996	152.93	152.9	152.84	152.58
20.829	152.01	151.98	151.91	151.65
21.662	150.98	150.95	150.88	150.62
22.495	149.79	149.76	149.69	149.41
23.328	148.36	148.33	148.25	147.97
24.161	146.46	146.43	146.35	146.06
24.995	142.95	142.92	142.86	142.68
25.289	143.47	143.43	143.37	143.17
25.583	143.51	143.48	143.42	143.21
25.877	143.52	143.48	143.42	143.2
26.171	143.51	143.48	143.41	143.2
26.465	143.5	143.47	143.4	143.19

26.759	143.5	143.47	143.4	143.18
27.053	143.49	143.46	143.39	143.18
27.347	143.49	143.46	143.39	143.17
27.641	143.49	143.45	143.39	143.17
27.935	143.48	143.45	143.38	143.17
28.229	143.48	143.45	143.38	143.16
28.523	143.48	143.45	143.38	143.16
28.817	143.48	143.45	143.38	143.16
29.111	143.48	143.45	143.38	143.16
29.405	143.48	143.44	143.38	143.16
29.699	143.48	143.44	143.38	143.16
29.994	143.48	143.44	143.38	143.16
30.708	143.47	143.44	143.37	143.15
31.422	143.47	143.43	143.37	143.15
32.136	143.46	143.43	143.36	143.14
32.85	143.44	143.41	143.34	143.12
33.564	143.42	143.38	143.31	143.09
34.278	143.34	143.31	143.24	143.02
34.992	143.03	143	142.93	142.71
35.377	142.87	142.84	142.77	142.56
35.762	142.72	142.68	142.62	142.4
36.146	142.55	142.52	142.46	142.24
36.531	142.39	142.36	142.29	142.08
36.915	142.23	142.19	142.13	141.91
37.3	142.06	142.03	141.96	141.74
37.684	141.89	141.85	141.79	141.57
38.069	141.71	141.68	141.61	141.39
38.453	141.52	141.49	141.42	141.2
38.838	141.33	141.3	141.23	141.01
39.222	141.13	141.1	141.03	140.8
39.607	140.92	140.88	140.81	140.58
39.991	140.69	140.65	140.58	140.34

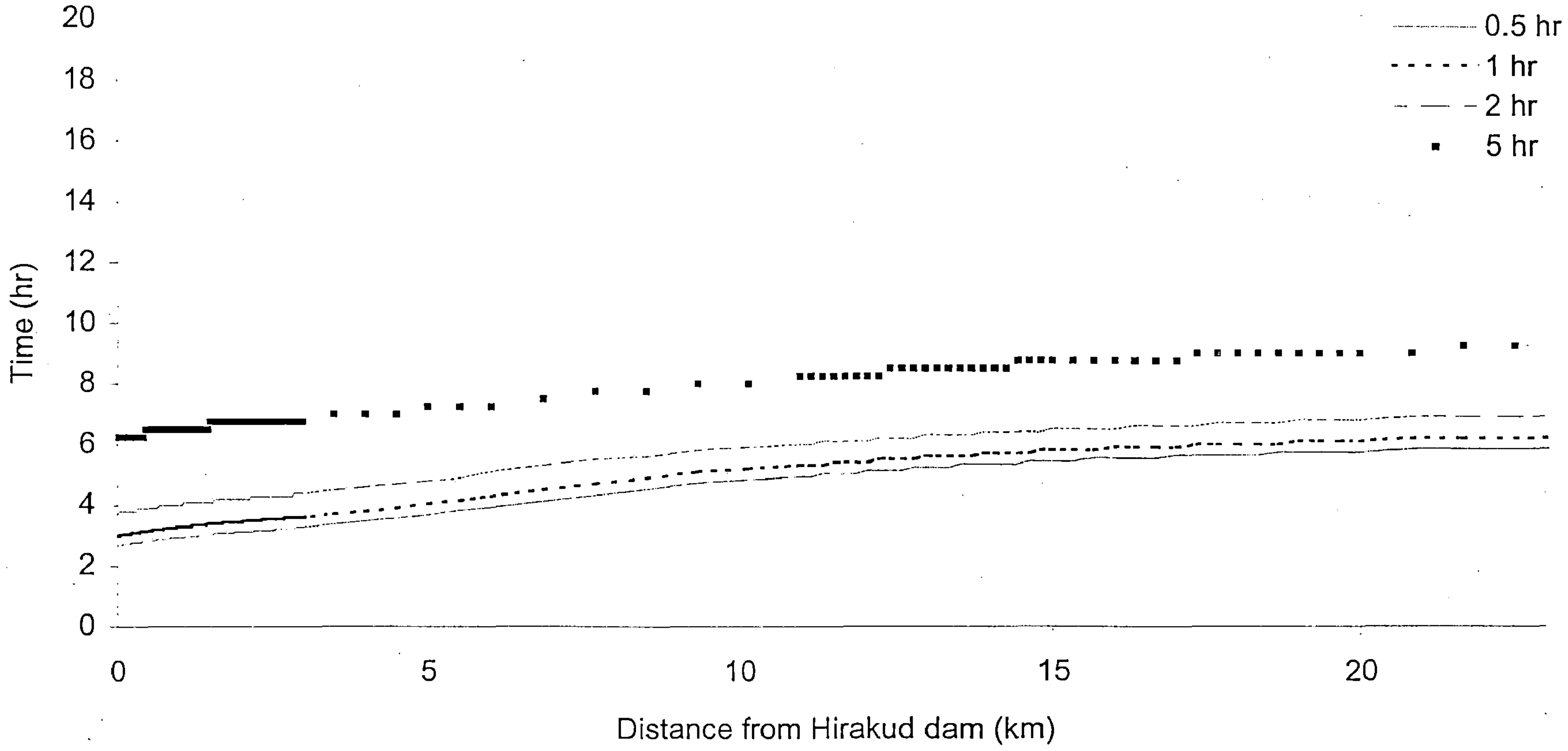


Figure 15: Effect of breach time on time to reach maximum water elevation at different locations

Thus, in the case of dam break flow of Hirakud dam, time to breach has a nominal effect on maximum water levels and however, an increase in time to breach results in a higher value of time to attain the maximum water levels.

4.5.3 Effect of Bed Roughness

In an open channel flow, the bed friction is the most important characteristics governing the flow. In the sensitivity analysis of bed roughness, different cases are considered as given in Table 9.

Table 9: Cases of studies for sensitivity analysis of bed roughness

Cases	Bed roughness	
	River (n_r)	Flood Plain (n_f)
1	0.05	0.05
2	0.075	0.075
3	0.050	0.075
4	0.035	0.05

In Fig. 16 (a), maximum water levels for cases 1 & 2 are presented for different locations and in Fig. 16(b), the corresponding values are shown for cases 3 and 4. It is observed that an increase in roughness value increases the maximum water levels. This is due to more resistance to the propagated flow. Though not presented here, the numerical results show a decrease in peak discharges due to increase in roughness. From Figure 16, it is also inferred that the roughness values of the flood plain have a greater bearing on the results.

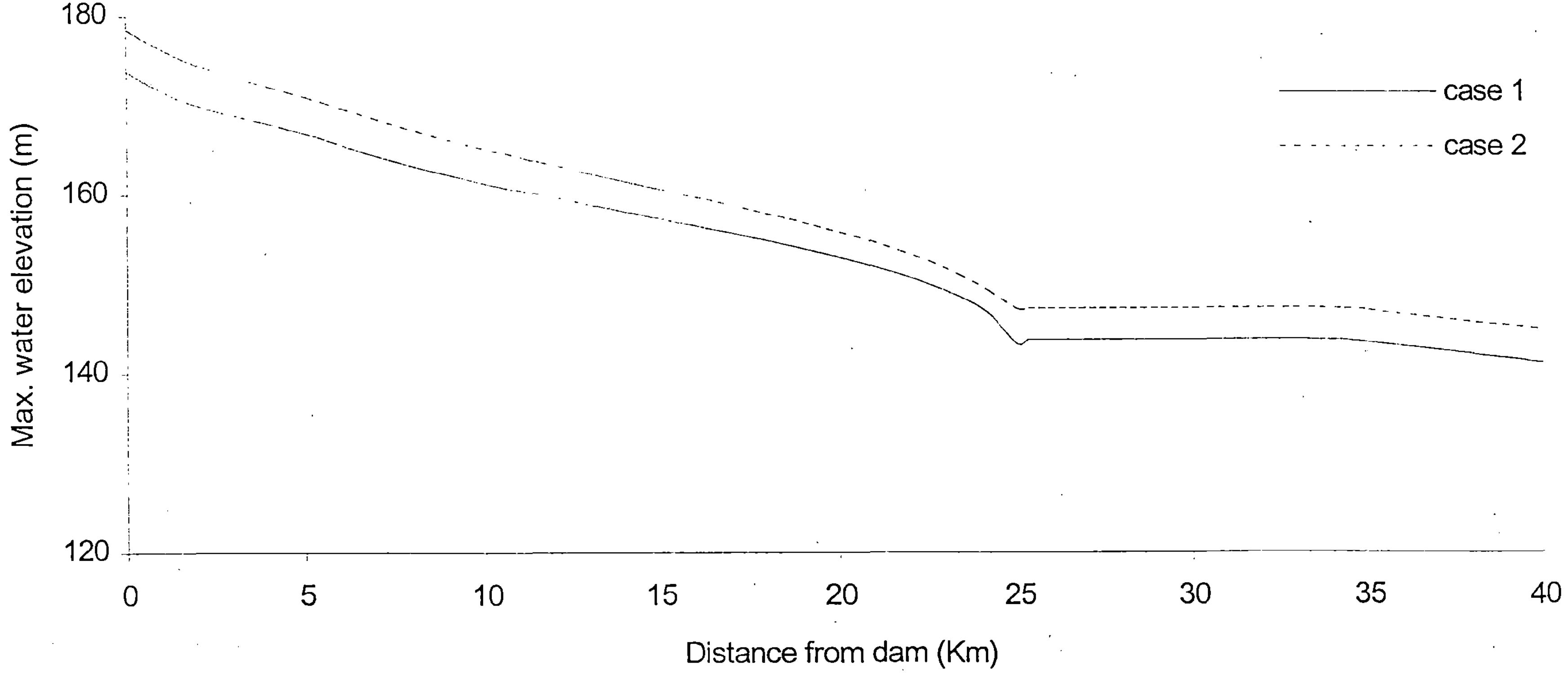


Figure 16 (a): Effect of bed roughness on maximum water level elevation (case 1 and 2)

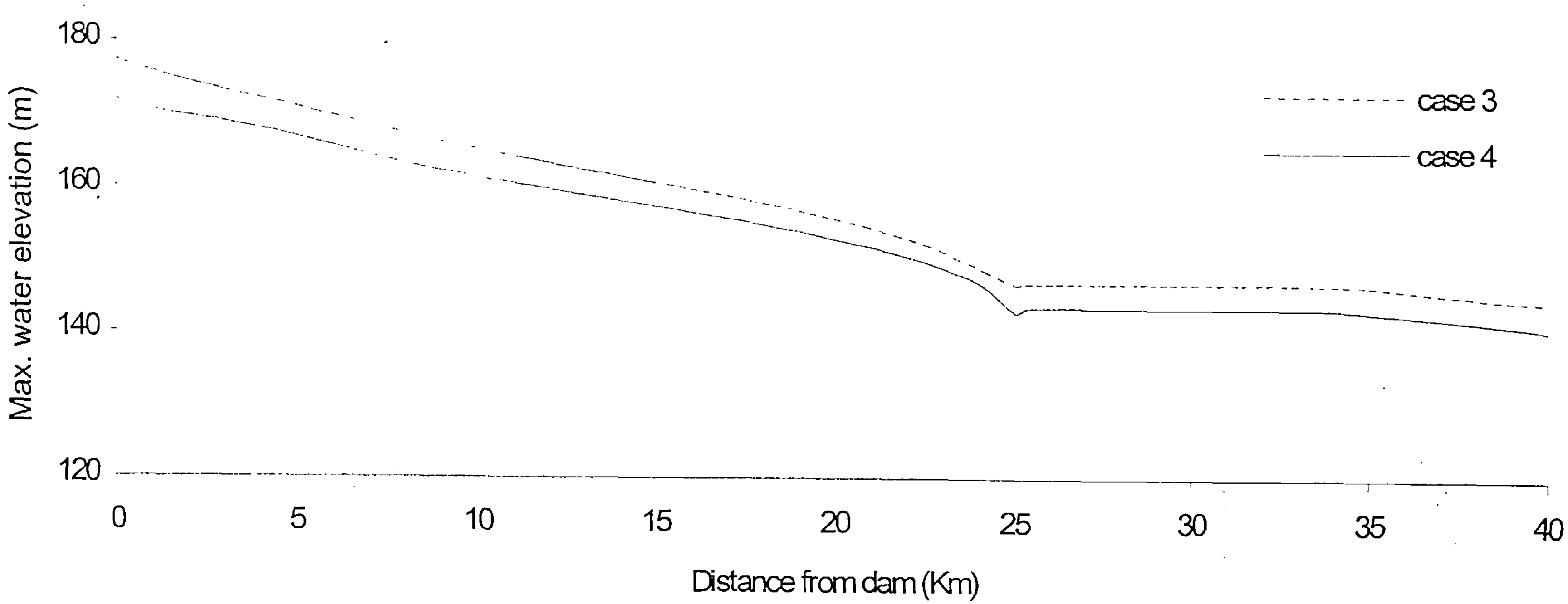


Figure 16 (b): Effect of bed roughness on maximum water level elevation (case 3 and 4)

4.6 FLOOD INUNDATION MAP

A flood inundation map is presented in Fig. 17, considering the worst case of dam break flow. This case corresponds to the input values are: (i) breach width = 250 m (ii) time to breach = 1 hr. (iii) $n_r = 0.05$ and (iv) $n_f = 0.075$. The computational model uses one-dimensional equation. Therefore, the maximum water level at a location represents the water level throughout the width. From the results of the model, maximum water elevations are determined at different location. These levels are superimposed on the contour map of the study area to prepare the inundation map. In this map, the times to attain these levels are indicated within brackets. Important villages and towns are shown in the figure. Inundation area corresponding to the design discharge from the spillway is also marked in the same figure.This inundation map may be helpful to designers and planners working in the field of flood disaster management, Provided analysis is carried out using elaborate and reliable data as present study involves many simplifying assumptions.

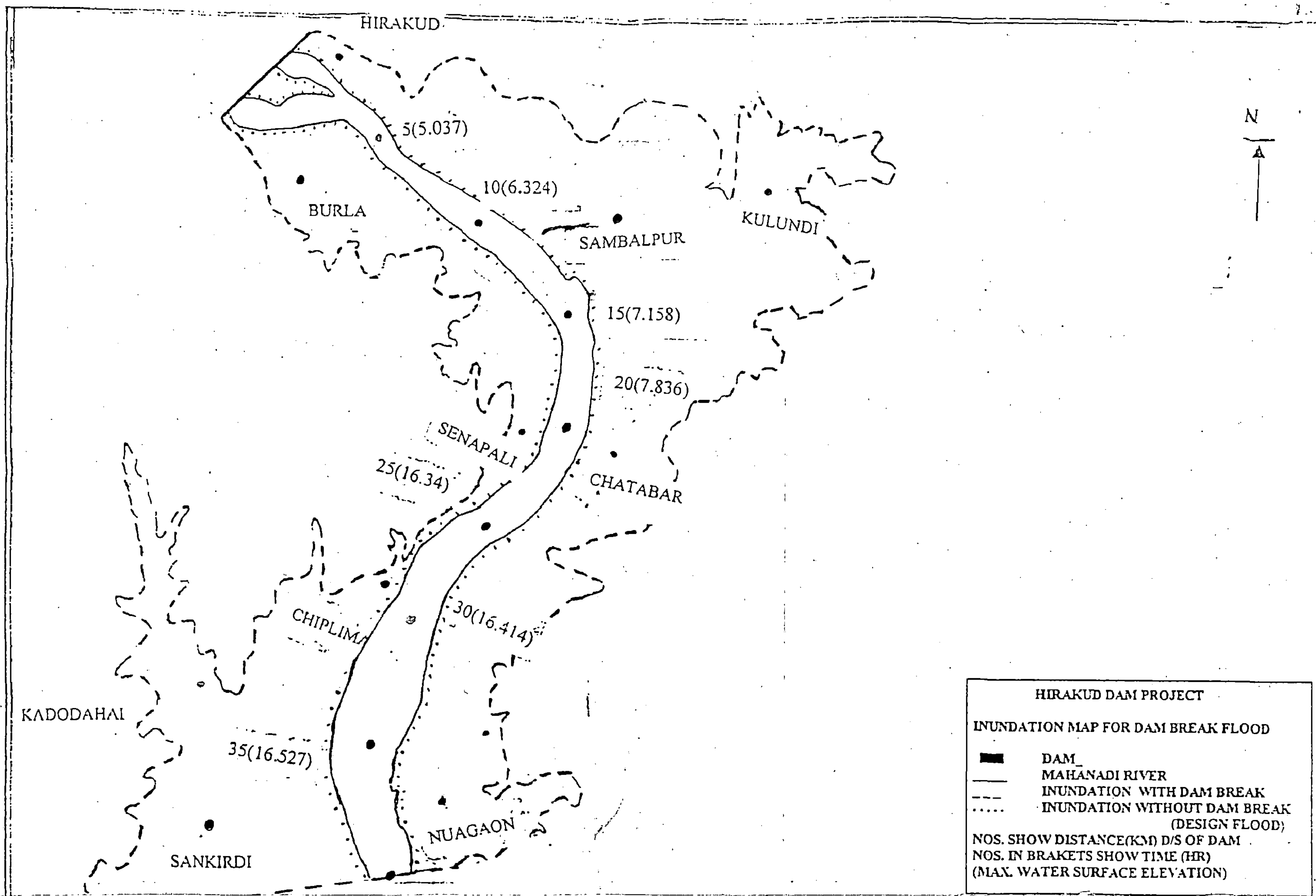


Fig. 17 : Inundation Map for the Worst Dam Break Scenario

CONCLUSIONS

In this dissertation work, dam break flow analysis for Hirakud dam was performed assuming a hypothetical dam failure case. A mathematical model 'DAMBRK' was used for this purpose. This model employs one-dimensional St. Venant equations and four Point Preismann scheme for channel routing. Data required for the above study was obtained from various sources. As it was a case of hypothetical dam failure, breach parameters were assumed. Conclusions derived from the present study are given belows.

(i) The peak discharge at dam site is144565 cumecs against the design value of 42450 cumecs.

(ii) The maximum water levels at the dam site is 171.83 m and at 40 km distance is 140.65 m. A sensitivity analysis for breach width shows that as the breach width increases, both Q_{peak} and H_{max}. increase.

(iii) A sensitivity analysis for breach time shows nominal increase in Q_{peak} and H_{max}, due to decrease of breach time.

(iv) Effect of roughness shows that the greater the roughness value, higher the H_{max} attained.

(v) Flood inundation map showing important places affected, for the worst scenario of dam break, has been presented.

The following recommendations are suggested for further studies.

(i) A dam break flow model with two-dimensional equations may be used.

(ii) A dam break flow model taking sediment transport into account may be used.

(iii) An accurate calibration procedure to determine the Manning's roughness (n) using optimization technique may be developed.

(iv) A model considering the breach mechanism may be used to determine the breach parameters.

(v) The present study may be extended to evaluate different losses as a function of water levels, of the study area.

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Appendix- 1

Salient Features of Hirakud dam Project

1.0 Location:

(i) Longitude	83^0 –52' E
(ii) Latitude	21^0 -32' N
(ii) District and Tahasil	Sambalpur
(iv) River	Mahanadi
(v) Location of dam	15 km upstream of Sambalpur town.

2.0 Main Works:

(i) Total length of dam	4800 m.
(ii) Earth-cum-rock fill dam	3651.5 m.
(iii) Left Earth dam	1353.3 m.
(iv) Right Earth dam	2298.2 m.
(v) Length of concrete & masonry dam	1148.5 m.
(vi) On left side	499.9 m.
(vii) On right side	648.6 m.

3.0 Hydrology:

(i) Catchment area at dam site	83400 sq, km.
(ii) Maximum annual rainfall	1808.73 mm.
(iii) Minimum annual rainfall	940.31 mm.
(iv) Mean annual rainfall	1381.25 mm.
(v) Maximum annual runoff	9.09 M.Hect.M.
(vi) Minimum annual runoff	1.14 M.Hect.M.
(vii) Average annual runoff	3.36 M.Hect.M.

4.0 Dam and Appurtenant works:

4.1 Water levels

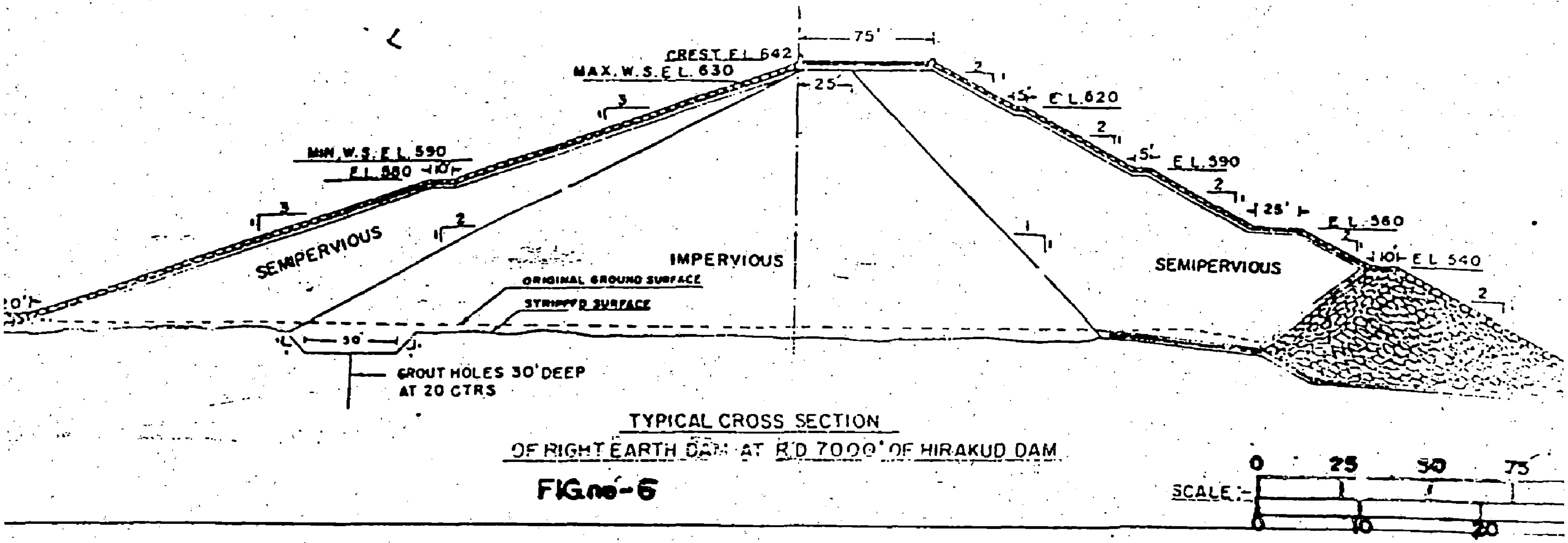
(i) Top of dam	195.68 m.
(ii) F.R.L./M.W.L.	192.024m.
(iii) Dead storage level	179.83 m.

4.2 Reservoir:

(i) Gross storage capacity	8136 M.Cum.
(ii) Dead storage capacity	2318 M. Cum.
(iii) Live storage capacity	5818 M.Cum.
(iv) Water spread area at F.R.L.	743 sq.km.
(v) Water spread area at D.S.L.	274 sq. km.
(vi) Maximum fetch at F.R.L.	83.2 sq.km.

4.3 Spillway:

(i) Number of under sluices	64
(ii) Size of under sluices	3.658 × 6.2 m each
(iii) Sill of under sluices	155.448 m.
(iv) No of crest gates	34
(v) Size of crest gates	15.54 × 6.10 m.
(vi) Crest level of spillway dam	185.928 m.
(vii) Spillway capacity	42450 Cumecs.



TYPICAL CROSS SECTION
OF RIGHT EARTH DAM AT R.D 7000' OF HIRAKUD DAM

FIG.no-6

0	35	376	778	1215	1230	2400	3000
0	0	0	0	0	0	600	6800
20.0							
133.04	134.00	135.00	136.00	137.00	138.00	150.00	160.00
0	415	910	1050	1250	1285	2800	3200
0	0	0	0	0	0	2200	4300
25.0							
121.34	125.00	127.50	130.00	132.50	140.00	150.00	160.00
0	60	65	640	1150	1800	3100	4000
0	0	0	0	0	200	2900	11000
30.0							
119.02	120.00	122.50	125.00	127.50	140.00	150.0	160.0
0	60	210	810	1065	2250	3500	4300
0	0	0	0	0	1800	8000	10000
35.0							
118.16	120.00	122.00	124.00	130.00	140.00	150.0	160.0
0	120	510	960	2020	2800	3800	4750
0	0	0	0	0	2800	9800	12000
40.0							
116.25	120.00	122.50	125.00	132.50	140.00	150.0	160.0
0	600	1050	1260	1800	3050	4250	5000
0	0	0	0	0	1700	10000	14750
45.0							
115.455	118.00	120.00	122.00	124.00	126.00	140.00	150.00
0	780	1020	1140	1160	1190	3250	4550
0	0	0	0	0	0	1500	4250
0.035	0.035	0.035	0.035	0.035	0.035	0.035	0.050
0.035	0.035	0.035	0.035	0.035	0.035	0.050	0.050
0.035	0.035	0.035	0.035	0.035	0.035	0.050	0.050
0.035	0.035	0.035	0.035	0.035	0.035	0.050	0.050
0.035	0.035	0.035	0.035	0.035	0.035	0.050	0.050
0.035	0.035	0.035	0.035	0.035	0.045	0.050	0.050
0.035	0.035	0.035	0.035	0.035	0.050	0.050	0.050
0.035	0.035	0.035	0.035	0.035	0.050	0.050	0.050
0.035	0.035	0.035	0.035	0.035	0.050	0.050	0.050
0.035	0.035	0.035	0.035	0.035	0.045	0.050	0.050

0.030	.5	.8	.18	.325	.8	.28	.7
.375	.8						
0	0	0	0	0	0	0	0
0	0						
0	0	0	.00082				