A CASE STUDY ON DAM FAILURE ANALYSIS

A DISSERTATION
Submitted in partial fulfillment of the requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
WATER RESOURCES DEVELOPMENT (CIVIL)

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in dissertation entitled **"A CASE STUDY ON DAM FAILURE ANALYSIS"** in partial fulfillment of the requirement for the award of the Degree of Master of Technology in Water Resources Development (Civil) submitted in the Water Resources Development Training Center, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out since July, 2003 till the date of submission under the supervision of Dr. Nayan Sharma, Professor, WRDTC, IIT Roorkee and Dr. M. Perumal, Associate Professor, Dept. of Hydrology, IIT Roorkee India.

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

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Place: Roorkee
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This is to certify that the above statement made by the candidate is correct to the best of our knowledge.

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SYNOPSIS

Dams are unique hydraulic structures designed to conserve and regulate fluctuating river discharges by creating water reservoirs, which enable to satisfy several vital human needs. On the other hand dams pose considerable hazard to life and property downstream in the event of failure. To prevent or minimize these hazards Dam Safety Organizations or cells have been set-up country as well as in other countries. Such cells take all measures for monitoring and maintaining the good condition of the dam to serve their purpose. Despite such measures, the possibility of dam failures can never be ruled out. In addition there is always a risk of damage by enemy action in the event of war. Now-a-days it has become obligatory to carry out a 'Dam Break' study, which primarily includes analysis of the dam breach flood wave and its routing through the downstream valley for floodplain zoning purposes i.e. the identification of areas likely to be submerged and the duration of submergence in the event of a sudden failure of dam. Such a study would help the local authorities to appropriately the plan evacuation and rehabilitation measures in the eventuality of dam failures.

The present study uses the NWS DAMBRK model for studying dam break flow analysis for Hirakud dam in Orrisa for various hypothetical dam failure scenario with the objective of:

1. Comparing the performance of NWS DAMBRK model with that of MIKE11 model using the same input data information as has been used by the Central Water Commission (1999) for studying the hypothetical dma break of the Hirakud dam using MIKE-11 model, and
2. To assess the flood wave characteristics of the hypothetical failure of the Hirakud dam using DAMBRK model with and without considering the PMF inflow hydrograph entering upstream of the reservoir when other physical conditions at the dam and its downstream remaining the same.

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INTRODUCTION

1.1 GENERAL

Dams play a very vital role in the economy of a country by providing essential benefit like irrigation, hydropower, flood control, drinking water, recreation etc. However in the event of their failure, these may cause catastrophic flooding in the downstream area, which may result in huge loss to human life and property. This loss to life and property would vary with extent of inundation area, size of population at risk and the amount time available. Costa(1985) reports that 60% of the more than 11,100 fatalities associated with all dam failures worldwide have occurred in just three failures: Vaiont, Italy 1963 (2,600 dead; overtopping of concrete arch dam by landslide generated wave); Machhu II, India, 1979 (2000+ dead; overtopping of embankment dam); and Johnstown Dam, Pennsylvania, 1889 (2200 dead; overtopping of embankment dam). In each of these cases, large populations were given little warning. In fact, Costa reports that the average number of fatalities per dam failure is 19 times greater than when there is inadequate or no warning. Major causes of failures identified by Costa are overtopping due to inadequate spillway capacity (34 percent) foundation defects (30 percent) and piping and seepage (28 percent).

One of the preventive measures in avoiding dam failure disaster is by issuing flood warning to the public of downstream when there is failure of dam. However, it is quite difficult to conduct analysis and determine the warning time of the dam break flood at the time of disaster. Therefore, pre-determination of the warning time assuming a various hypothetical dam break situations is a needed exercise in dam safety measures. With this view, the hypothetical failure of Hirakud dam is studied herein.

The dam failure study involves the following component steps;

i) Development or identification of the inflow hydrograph to the reservoir at the time of failure.

ii) Routing that hydrograph through the reservoir.

iii) Development of the failure condition of the structure.

iv) Calculating the outflow hydrograph from the failed structure, and

v) Modelling the movement of the flood wave downstream to determine the travel time, maximum water level reached, inundated area etc.

Considering the above steps, there are various mathematical models available in practice to study the hypothetical dam failure problems for developing the flood inundation maps downstream of the failed dams. The well- known models available for dam-break studies are the National Weather Service's DAMBRK model developed by Dr. D.L.Fread (1984) and the Danish Hydraulic Institute's MIKE-11 model.

Generally the field study of dam failure using the mathematical models pose various problems with regard to matching the model assumptions. The difficult problem is concerned with regard to the failure description of the structure as the failure occurred in nature would be different from the failure description adopted in the model. Besides, the dam failure of overtopping generally occur due to severe storm with high inflow into the reservoir. Therefore, this inflow hydrograph is used for dam break analysis. Also due to the failure of the dam, the downstream gauging stations are generally submerged resulting in no information on the downstream hydrographs. Therefore in many cases, the only available information is the maximum water level marks reached at the time of the passing of the flood wave. Therefore, many uncertainties are associated with various aspects of dam failure analysis.

The hypothetical dam break study for Hirakud dam has already been studied using both DAMBRK model Mahapatra thesis (2000) and MIKE-11 model (CWC, 1999). While Mohapatra (2000) has carried out Hirakud dam failure study up to 40 km downstream of the dam, and the CWC has carried out the study of hypothetical dam failure scenario of Hirakud dam and the ensuing flood propagation up to 310 km downstream of the dam, no comparative evaluation of the performance of these models for hypothetical dam failure scenario was made. In addition, none of the studies have made an assessment of dam break flood wave characteristics using the analyses with and without considering inflow into the reservoir. Such an analysis would help to conduct hypothetical dam break studies without giving importance to the inflow hydrograph entering in to the reservoir, but giving importance to the level of water behind the dam at the time of failure of the dam.

1.2 OBJECTIVES OF THE STUDY

Considering the above-discussed aspect of the hypothetical dam failure scenarios of Hirakud, the following are the objectives of the present study:

(i) In order to compare the performance of the NWS DAMBRK model with that of MIKE-11 model, it is proposed to carry out the dam break analysis of Hirakud dam using the same input information as has been used by CWC for studying the dam break study of the Hirakud dam using MIKE- 11 model and

(ii) To assess the dam break flood wave characteristics of the Hirakud dam failure study using the DAMBRK model with and without considering inflow Hydrograph entering upstream of the reservoir when the condition at the downstream of the dam remaining same for both the scenarios.

LITERATURE REVIEW

2.1 INTRODUCTION

The dam break analysis problem is one of the most fascinating hydraulic problems and the concerned literature is extensive. The first study was carried out by Ritter (1892) who used the method of characteristics to obtain a closed form solution for a dam of semi-infinite extent upon a horizontal bed with zero bed resistance. Both experimental and theoretical, however, have shown that the neglect of bed resistance invalidates the Ritter solution in a region that starts near the leading edge of the flood wave. Dressler (1952) used a perturbation procedure to obtain a first order correction for resistance effects. Sakkas and Strelkoff (1973), Chen and Armbruster (1980) have used the method of characteristics to obtain numerical solutions for dam break problem on sloping beds. These solutions were for reservoirs of finite lengths and include the effects of bed resistance. Investigators of dam break flood waves such as Ritter (1892), Re (1946), Dressler (1954), Stoker (1957), Su and Barnes (1969) and Sakkas and Strelkoff (1973) assumed the breach encompasses the entire dam and that it occurs instantaneously. U.S. Army Corps of Engineers (1960) have recognized the need to assume the partial rather than complete breaches; however, they assumed the breach occurred instantaneously. The assumption of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam break flood waves.

Recognising the practical aspect Cristofano (1965), Harris and Wagner (1967) incorporated the partial time dependant breach formation in earthen dams ; however, this procedure requires critical assumptions and specification of unknown critical parameter values. Also Harris and Wagner (1967) used a sediment transport relation to determine the time for breach formation , but this procedure requires specification of breach size and slope in addition to other critical parameters for the sediment transport relation.

2.2 DAM BREAK MODEL

The national weather Service's DAMBRK model developed by Dr. D.L.Fread (1984) is used in this study of Hirakud Dam failure analysis. This model simulates the failure of a dam, computes the resultant outflow hydrograph and simulates movement of the dam break flood wave through the downstream river valley. The model is built around three major capabilities which are reservoir routing, breach simulation and river routing. However, it does no rainfall-runoff analysis and storm inflow hydrographs to the upstream of reservoir must be developed external to the model. A brief description of these model capabilities are given herein and for detailed description may be referred to the user manual of NWS (Fread, 1984).

2.2.1 Reservoir Routing

In this model the reservoir routing may be performed either using storage routing or dynamic routing.

a) Storage routing

The storage routing is based on the law of conservation given as:

$$I\text{-}Q=dS/dt \qquad (2.1)$$

in which I is the reservoir inflow, Q is the total reservoir outflow, and = dS / dt is the time rate of change of reservoir storage volume. The above equation may be expressed in finite difference form as ;

$$(I+I')/2 - (Q+Q')/2 = \Delta S/dt \qquad (2.2)$$

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

$$\Delta S = (A_s + A_s') \, (h-h')/2 \qquad (2.3)$$

in which A_s is the reservoir surface area coincident with the elevation (h) and it is a function of h. The discharge Q which is to be evaluated from equation (2.2) is a function of h and this unknown h is evaluated using the Newton-Raphson iteration technique and, thus, the estimate of discharge corresponding to h.

(b) Dynamic Routing

The hydrologic storage routing technique, expressed by the equation above implies that the water surface elevation within the reservoir is horizontal. This assumption is quite adequate for gradually occurring breaches with no substantial inflow hydrograph. However, when (1) the breach is specified to form almost instantaneously so as to produce a negative wave within the reservoir, and/or (2) the reservoir inflow hydrograph is significant enough to produce a positive wave progressing through the reservoir, a routing option which simulates the negative and /or positive wave occurring within the reservoir may be used in DAMBRK model. Such a technique is referred to as dynamic routing. The routing principle is same as dynamic routing in river reaches and it is performed using Saint Venant's equations which will be described in the section of river routing.

(c) Breach Simulation

Two types of breaching may be simulated using this model:

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

ii) A piping failure in which the breach is simulated as a rectangular orifice that grows with time and is centered at any specified elevation within the dam. Instantaneous flow through the breach is calculated with either orifice or weir equations depending on the relation between the pool elevation and the top of the orifice.

The peak shape of the outflow hydrograph due to dam breach is governed largely by the geometry of the breach and its development with its time. The actual formation of a breach in earth dams is a complex process, depending upon the various hydraulic, hydrological and structural factors, and parameters. This process can be expected to be highly non-linear with time and partial collapse may occur when the downstream face of the dam has suffered considerable erosion.

DAMBRK model defines the breach due to overtopping in five parameters, viz. side slope of the breach section z; the final bottom width of the breach, YBMIN; the time from inception to completion of breach, TF; and, the failure elevation, HF. The model assumes that the breach starts at a point and both the breach width and the depth increases at a linear rate over the failure time. The elevation of the breach bottom

YBMIN, is usually taken to be the channel bottom or the dominant ground elevation of the dam, except when this was not physically justifiable due to the backwater effects. Therefore, cross-sectional information immediately downstream of the dam in order to calculate tail water elevation for any needed correction for partial submergence is required.

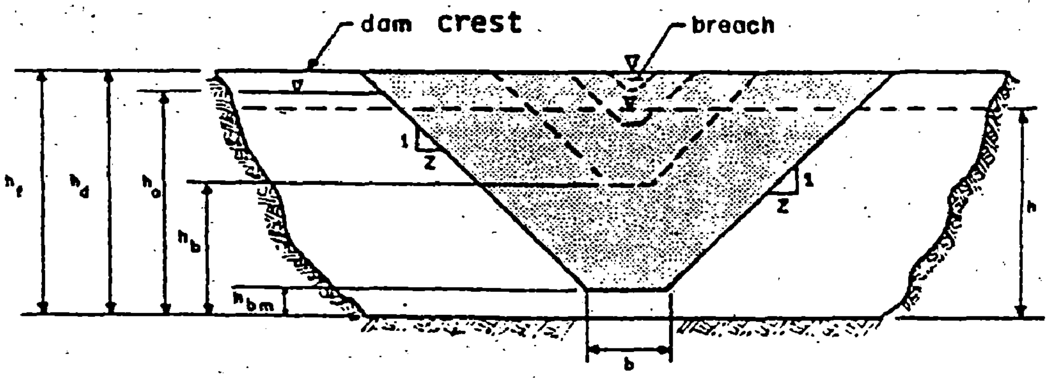


FIG. FRONT VIEW OF DAM SHOWING FORMATION OF BREACH

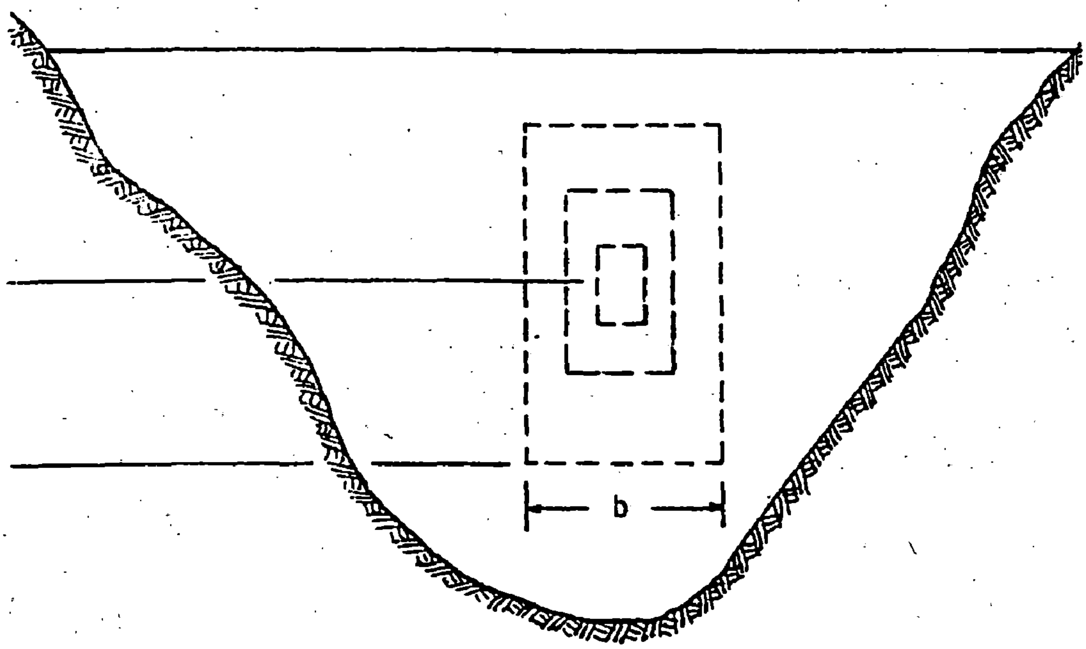


FIG. ORIFICE BREACH

(d) Breach shape

In case of an overtopping failure, DAMBRK model can simulate the breach shape as rectangular, trapezoidal or triangular. But is seen from the case histories of dam failure (Mac-Donald and Monopolis, 1984) that the breach shape occurred during the dam failures were mostly trapezoidal.

(e) River Routing

The movement of the dam break flood wave through the downstream channel is simulated using the complete unsteady flow equations, known as the Saint-Venant equations.

The Saint- Venant unsteady flow equations consists of a conservation of mass equation expressed as,

$$\frac{\partial Q}{\partial x} + \frac{\partial (A + A_0)}{\partial t} = q \qquad (2.4)$$

and the conservation of momentum equation :

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2 / A)}{\partial x} + gA(\frac{\partial h}{\partial x} + S_f + S_e) + L = 0 \qquad (2.5)$$

where,

A= active cross sectional flow area,

A_0 = inactive (off-channel storage) cross-sectional area,

x = distance along the channel,

q = lateral inflow or outflow per unit distance along the channel,

g = acceleration due to gravity,

h = water surface elevation,

S_f = friction slope,

Se = expansion- contraction loss slope,

L= lateral inflow or outflow momentum effect due to assumed flow path of inflow being perpendicular to the main flow.

The friction slope and expansion-contraction loss slope are evaluated by the following equation:

$$S_f = \frac{n^2|Q|Q}{2.21A^2R^{4/3}} \tag{2.6}$$

And,

$$S_e = \frac{K\Delta(Q/A)^2}{2g\Delta x} \tag{2.7}$$

Wherein,

n = Manning's roughness coefficient,

R = A/B where B is the top width of the active portion of the channel,

K = An expansion-contraction coefficient varying from 0.1 to 0.3 for contraction, -0.5 to –1.0 for expansion,

$\Delta(Q/A)^2$ = Difference in $(Q/A)^2$ for cross section at either end of a reach.

The non- linear partial differential equations (2.4) and (2.5) are represented by a corresponding set of non- linear finite difference algebraic equation and they are solved by the Newton- Raphson method using weighted four point implicit scheme to evaluate Q and h. The initial conditions are given by known steady discharge at the dam, for which water surface elevation at each cross section are calculated by solving the steady state non-uniform flow equation. The outflow hydrograph from the reservoir is the upstream boundary condition for the channel routing and the model is capable of dealing with fully supercritical flow or fully subcritical flow. There is a choice of downstream boundary conditions such as internally calculated loop rating curve, user provided single valued rating curve, user provided time dependant water surface elevation, critical depth and dam which may pass flow via spillway, overtopping and/or breaching.

(f) Data Requirement

The DAMBRK model was developed so as to require data that was accessible to the forecaster. The input data requirement are flexible in so far as much of the data may be ignored when a detailed analysis of a dam break flood inundation event is not feasible due to lack of data or insufficient data preparation time. Nonetheless the resulting approximate analysis is accurate and convenient to obtain than that which could be computed by other techniques. The input data can be categorized into two groups.

The first data group pertains to the dam (the breach, spillways, and reservoir storage volume). The breach data consists of the following parameters: (failure time of breach, in hours), b (final bottom width of breach), z (side slope of breach), h_{bm} (final elevation of breach bottom), h_0 (initial elevation of water in reservoir), h_f (elevation of water when breach begins to form), and h_d (elevation of dam). The spillway data consists of the following: hs (elevation of uncontrolled spillway crest), c_s (coefficient of discharge of uncontrolled spillway), h_g (elevation of center of submerged gated spillway), c_g (coefficient of discharge of gated spillway), c_d (coefficient of discharge of crest of dam), Q_t (constant head independent discharge from dam). The storage parameters consists of the following: a table of surface area (A_s) in acres or volume in acre ft. and the corresponding elevation within the reservoir. The forecaster must estimate the values of τ , b , z , h_{bm} and h_f . The remaining values are obtained from the physical description of the dam, spillway, and reservoir. In some cases h_s c_s , h_g , c_g and c_d may be ignored and Q_t used in their place.

The second group pertains to the routing of the outflow hydrograph through the downstream valley. This consists of a description of the cross sections, hydraulic resistance coefficients, and expansion coefficients. Location mileage and tables of top width (active and inactive) and corresponding elevations specify the cross sections. The active top widths may be total widths as for a composite section, or they may be left floodplain, right floodplain, and channel widths. The channel widths are usually not as significant for an accurate analysis as the overbank widths. The number of cross-sections used to describe the downstream valley depends on the variability of the valley widths. They also depend on the availability of cross-section measurements. However, a minimum of two must be used. Additional cross-sections are created by the model via linear interpolation between adjacent cross-sections specified by the forecaster. This features enables only a minimum of cross-sectional data to be input by the forecaster according to such criteria as data availability, variation, preparation time etc. The number of interpolated cross-sections created by the model is controlled by the parameter DXM which is input for each reach between specified cross-sections. The expansions and contraction coefficients (FKC) are specified as non-zero values at sections where significant expansion or contractions occur. But they may be left blank most of the analyses.

2.3 MIKE- 11 MODEL

MIKE 11 is a comprehensive, one-dimensional modeling system for the simulation of flows, sediment transport and water quality in estuaries, rivers, irrigation systems and other water bodies. It is a 4^{th} generation modeling package designed for microcomputers with DOS or UNIX operating systems and provides the user with an efficient interactive menu and graphical support system with logical and systematic layouts and sequencing of the menus.

MIKE 11 hydrodynamic module is an implicit, finite difference model that can describe subcritical as well as supercritical flow conditions through a numerical description which is altered according to the local flow conditions in time and space. The hydrodynamic model is based on the one-dimensional Saint-Venant equations. The two equations representing conservation of mass and momentum as in DAMBRK model, are derived on the basis of the following assumption.

- Water is incompressible and homogeneous; i.e. , without significant variations in density.
- Bottom slope is small.
- Wavelengths are large compared to the water depth.
- Flow is subcritical.

By virtue of its general formulation, this model is suitable for wide range of application as detailed below:

- Flood forecasting and reservoir operation
- Simulation of flood and evaluation of flood control measures
- Operation of irrigation and surface drainage system
- Design of channel system
- Sedimentation studies
- Dam break studies

The dam break model set up consists of a single or several channels, reservoirs, dam break structures, and other auxiliary dam structure such as spillways sluices etc. The river is represented in the model by cross section at regular intervals. However due to the highly unsteady nature of dam break flood propagation, it is advisable that the river course be described as accurately as possible through the use of closely spaced cross sections, particularly where the cross section is changing rapidly. Further the cross sections should extend as far as the highest modelled water level, which normally will be in excess of the highest recorded flood level.

The reservoir is normally modelled as a single 'h' point to describe the storage characteristics by the use of storage area at different levels. This point will often also be the upstream boundary of the model, where inflow hydrograph may be specified. However, in case of very long and wide reservoirs the routing of the inflow floods has to be carried out and hence the reservoir itself will have to be represented by cross sections at regular intervals. The downstream will be the either a discharge water level relation or time series water level as in case of tidal waves etc.

The manner in which the failure is to commence can be specified as one of the following:

- Given numbers of hours after the start of the simulation
- At a specified time
- At a specified reservoir level

The breach may be specified as a rectangular, triangular or trapezoidal in shape. The initial and final breach width levels along with the side slopes of the breach are required to be specified. The model has the option to select either the linear failure mechanism or an erosion based formulation. The linear failure mode assumes a linear increase in breach dimension with time, between specified limits. Erosion based breach formulations are based on sediment continuity equation for the breach, i.e., the height of the breach varies according to the equation;

$$dHb/dt = S / W_b (1 - P) \qquad (2.8)$$

where, W_b= Width of the breach in the flow direction

S= Sediment transport capacity

P= Soil porosity

Hb= Height of breach

The increase in the breach depth is calculated from classical sediment transport formula being those of Meyer-Peter and Muuller and Engelund-Hansen. The increase in breach width is calculated as the increase in breach depth multiplied by the side erosion index. For the erosion based failure additional data such as slopes of the upstream and downstream face of dam, width of dam crest and density, grain size, porosity and critical shear stress of dam materials are required.

Data required for the dam break studies are:

- River and reservoir details
- Cross section of the river downstream of dam at suitable intervals
- Upstream boundary condition which is usually the design flood hydrograph
- Downstream boundary condition that can be a rating curve.
- Salient features of all hydraulic structures
- Details of inflow / outflow etc.
- Manning's roughness coefficient
- Properties of construction material in case of earth and rockfill dams such as grain diameter, porosity, density, critical shear stress etc.

2.4 SALIENT FEATURES OF THE PAST HYPOTHETICAL DAM FAILURE STUDIES FOR HIRAKUD DAM

2.4.1 Study Using DAMBRK Model

The salient features of the study conducted by Mahpatra (2001) on the hypothetical failure of the Hirakud dam are given below:

- The dam was assumed to fail by overtopping failure
- Breach parameters were assumed as; breach width=250 m, time to breach=1hr and side slope of breach=0, the final level of breach =152.4m
- The bed roughness coefficient assumed were 0.035 and 0.050 for main rivers and flood plains respectively
- The peak discharge at dam site was estimated as 144565 cumecs
- Maximum water level at dam site was estimated as171.83
- The channel routing was done up to 40 km of the downstream of the dam

- The maximum water level reached at 40km downstream of dam was 140.65m
- A sensitivity analysis breach time shows nominal increase in Q_{peak} and H_{max} due to decrease of breach time.

2.4.2 Study Using MIKE 11 Model

The Central Water Commission (1999) carried out dam break study for the hypothetical failure study of the Hirakud dam and the following recommendations and conclusions were arrived at:

The banks were inundated at all the locations up to 310km due to the dam breach, the banks up to 180 km. downstream of the dam got inundated even for the release of discharge at full capacity from the spillway.

The study showed that the river was overflowing in most of the cross sections. In MIKE11 model the cross sections are assumed as vertical beyond the extreme bank levels specified and hence the water levels obtained above the bank levels may not be realistic. As such it was pointed out that all these cross sections were to be extended beyond the present bank levels up to the maximum water level in order to describe the topography correctly in the model.

2.5 CONCLUSIONS

Though the study by Mahaptra (2000) for the hypothetical dam failure analysis of the Hirakud dam was carried out using DAMBRK model at later date than the CWC study (1999), due to unawareness of the CWC report, comparison of the performance of the models aimed to serve the same purpose was not made. While there is no doubt that MIKE11 is becoming an authorized model for dam failure analysis in our country, there is no reason why the NWS DAMBEK model can also not used as an alternative model for dam failure analysis. One of the probable reasons that why the MIKE 11 is preferred for dam failure analysis may be due to its capabilities to deal with various other hydraulic and hydrological problems, unlike that of DAMBRK model which only deals with dam failure analysis. But that is not a scientific reason. So the preference of MIKE 11 over NWS DAMBRK model should be based on their performance and the evaluation of their performances is possible when the same data base is used.

STATEMENT OF THE PROBLEM

The preference of one dam break analysis model over the other has not been addressed properly in our country. In fact the selection based on the criterion of performance is difficult to establish as both these models are applied for hypothetical dam failure scenarios only which have not been recorded. Therefore, in the absence of real life event simulations by these models, one cannot categorically argue about the superiority of one model over the other. However some insight about the relative performance of these models may be achieved, when these models attempt to simulate the same hypothetical dam break event using the same input data for these models.

Another aspect of dam break analysis is the use of PMF hydrograph as the inflow hydrograph during dam break analysis. It is argued that at the time of the failure of dam, the volume of water behind the dam would be so large, which is sufficient to produce an outflow hydrograph from the breached dam with high peak flood values without being influenced by the entering PMF hydrograph.

Considering these aspects, the following the problems are taken up in this study:

i) In order to compare the performance of NWS DAMBRK model with that of MIKE-11 model, it is proposed to carry out the dam break analysis of Hirakud dam using the same input information as has been used by the Central Water Commission (1999) for studying the dam break study of the Hirakud dam using MIKE- 11 model, and

ii) To assess the flood wave characteristics of the hypothetical failure of the Hirakud dam study using DAMBRK model with and without considering the inflow hydrograph entering upstream of the reservoir when the other physical conditions at the dam and its downstream remaining the same.

DESCRIPTION OF THE STUDY AREA

HIRAKUD DAM

4.1. INTRODUCTION

Hirakud Dam Project is built across river Mahanadi at about 15 Kms. upstream of Sambalpur town in State of Orissa. This happens to be the first post independence major multipurpose river valley project in India. The dam is 6 Kms from NH 6 . The nearest railhead is Hirakud railway station (S.E.R), which is 8 Kms from the dam site.

The project provides 1,55,635 Hects of Kharif and 1,08,385 Hects of Rabi irrigation in industries of Sambalpur, Bargarh, Bolangir, and Subampur. The water released through powerhouse irrigates further 436000 Hects of CCA in Mahanadi delta. Installed capacity for power generation in 307.5 MW through its two power houses at Burla , at the right bank to and Chiplima , at 22 Kms down stream of dam . Besides the project provides flood protection to 9500 sq Kms of delta area in district of Cuttack and Puri.

4.2 BRIEF HISTORY OF THE PROJECT

After high floods of 1937, Sir M. Visveswararya gave proposal for detailed investigation for storage reservoirs in Mahanadi basin to tackle problem of floods in Mahanadi delta, In 1945, it was decided under the chairmanship of Dr. B.R.Ambedkar, the then Member of Labor in Govt. of India that the potentialities of river Mahanadi should be fruitfully and expeditiously investigated for multipurpose use. Central Water- ways Irrigation and Navigation Commission took up the work. The foundation stone of Hirakud Dam was laid by Sir Hawthorns Lewis, the then Governor of Orissa on 15th March 1946. The project report was submitted to Government in June 1947. The first batch of concrete was laid by Pandit Jawaharalal Neheru on 12th April 1948. The project was formally inaugurated by Prime Minister Pt. Jawaharlal Nehru on 13th, January 1957. Power generation along with supply for irrigation started progressively from 1956 and full potential was achieved by 1966.

4.3. PROJECT FEATURES

Hirakud dam is a composite structure of Earth, Concrete and Masonry. The main dam having an overall length of 4.8 K.m. spans between hills Lamdungri on left hand Chandili Dunguri on the right. The Dam is flanked by 21 K.Ms. long earthen dykes both on left and right sites to close the low saddles beyond the abutment hills. It has the distinction of being the longest dam in the world, being 25.8 K.Ms. long with dam and dykes taken together. It has also the rare distinct of forming the biggest artificial lake in Asia with reservoir spread of 743 sq. Kms at full reservoir level. An index plan and general lay out plan of Hirakud dam have been given in Fig.4.1 and 4.2. The spillway on either side are solid gravity type with ogee shaped crest and having the crest level at EL 185.93m(610ft). There are 34 nos. of radial gates, 21nos. in left spillway and 13 nos. in right spillway of size (15.54m*6.10m) each. Both spillway contains 64 nos. under sluices, 40 nos. in the left side and 24 nos. in the right with sill level at EL155.43m (510 ft). The sizes of under sluices is 3.56*6.20m each. The combined discharge capacity of spillway and sluices at FRL is 41609 cumecs against the peak of revised PMF of 69632 cumecs. This has necessitated a dam breach analysis to study the damages that could occur at the down stream of dam in case of failure of dam. The spillway section is given in Fig.4.3.

Hirakud dam intercepts 83400-sq. km (32200 sq. miles) of Mahanadi catchments. The reservoir has storage of 5818 M. Cum with gross of 8136 M Cum.

Cost: The Completed Cost of the Project was Rs.100.02 crores (in 1957)

(A) HYDROLOGICAL

(a) Catchment - 83400 Sq. Kms (32200 sq. miles)

(b) <u>Rain fall(mm)</u>	-	<u>Original</u>	<u>Revised</u>
Mean annual	-	1381mm(1900 – 45)	1088mm
Maximum annual	-	1809mm (in 1919)	2518mm
Minimum annual	-	940mm (in 1902)	607mm
75% dependable Annual		1020mm	816mm

(c) Run Off (M Hect. M) -	Original	Post Construction
Average annual	6.17M.Hect.M	3.36 M.Hect.M
	(1926-46)	(1958-92)
Maximum annual	8.62 M.Hect.M	9.09 M.Hect.M
	(1919)	(1961)
Minimum annual	2.54 M.Hect.M	1.14 M.Hect.M
	(1902)	(1979)

(B) DAM AND RESERVOIR

Top dam level	R.L 195.680M.	(R.L.642 ft.)
F.R.L/ M. W. L	R.L 192.024M	(R.L 630 ft.)
Dead storage level	R.L 197.830 M	(R.L 590 Ft)

Storage capacity In M Cum (M. Ac. Ft.) (1988)	Original	Revised
Gross	8136 (6.60)	7189 (5.83)
Live	5818 (4.72)	5375 (4.36)
Dead	2318 (1.88)	1814 (1.47)

Water spread area at F.R.L	743 Sq. Km.
At D.S.L	274 Sq. Km.
Maximum fetch At F.R.L	83.2 Km.

MAIN DAM

Total length of Dam	4840 m.(15,741ft)
Length of concrete and masonry dam	1148.5m (3768ft)
On left side	499.9m(1640ft)
On right side	648.6m (2128ft)
Length of Earth dam	3651.5m(11980ft)
Left earth dam	1353.3m(4440ft)
Right Earth Dam	2298.2m (7450ft)
Length of dyke, Left Dyke	9337M in five gaps.
Right Dyke	10759 M in one stretch.

Total quantity of earthwork in Dam 18.1 M Cum.

Total quantity of concrete and 1.07 M. Cum

(C) SPILLWAY

Spillway capacity	42450 cumecs (15 lakhs cusecs)
Crest level	- R.L. 185.928 M (R.L. 610 ft.)
Size of sluices	- 3.658 x 6.20 M (12x 2034 ft)
No. of sluices	- 64 (40 on left and 24 on right)
Sill of sluices	- R.L. 155.448 M (R.L. 510 ft)
No. of crest gates	- 34 (21 on left and 13 on right)
Size of crest gates	- 15.54 M x 6.10 M. (51x 20 ft.)

Types – Solid gravity with ogee profile and skijump bucket.

(D) POWER GENERATION

Installed Capacity

At Burla	- 5 x 37.5	
	- 2x24.0	= 235.50 MW
At Chiplima	- 3x24	= 72.00 MW
Total		= 307.50 MW

Length of the power channel- 22.40 Km.

Full supply discharge of power - 22.40 Km

channel (beyond escape)

Full supply discharge of Chiplima P.H. – 333.4 cumecs.

Bed width of power channel;

Upto escape -75.5 M

Fig.4.1 Index Plan of Hirakud Dam

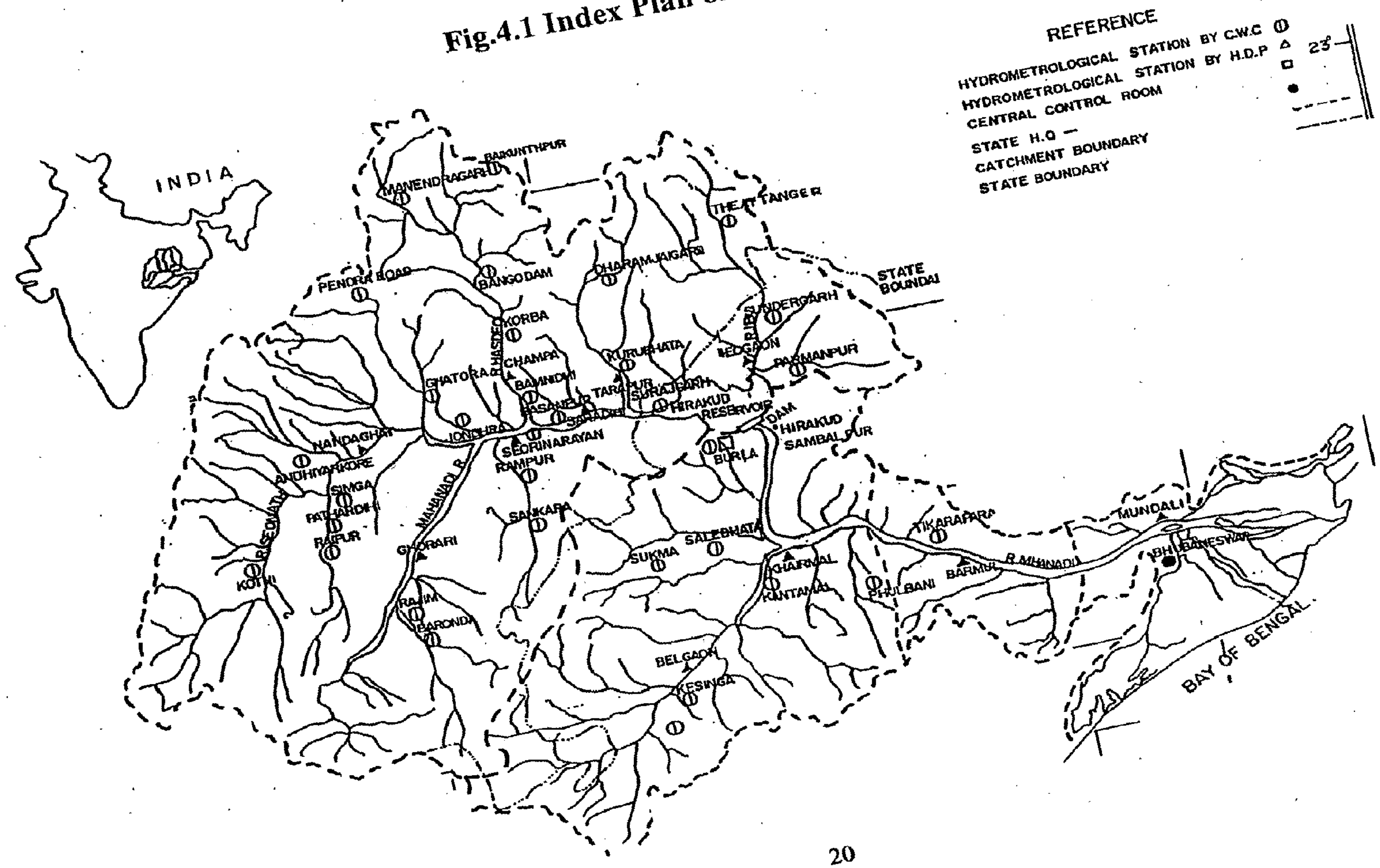
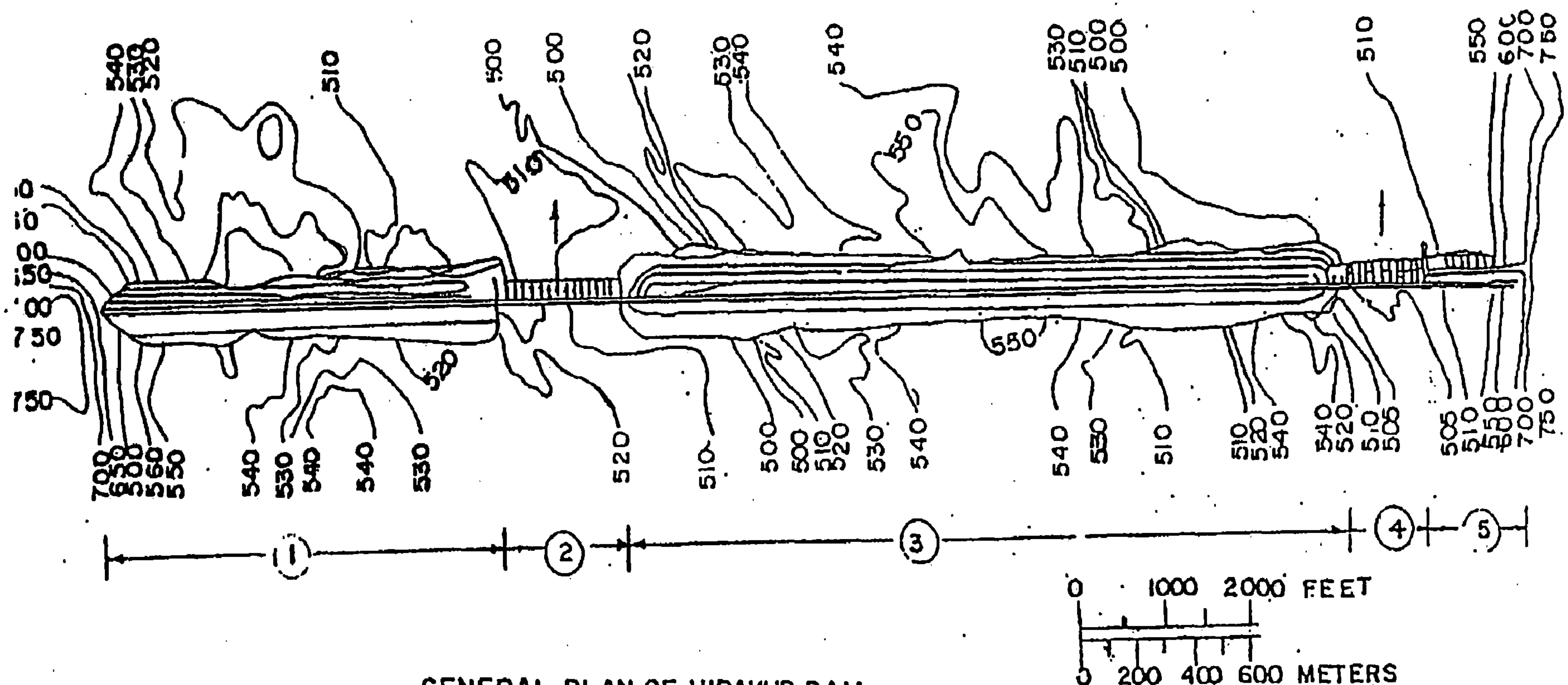


Fig.4.2 Layout Plan of Hirakud Dam



GENERAL PLAN OF HIRAKUD DAM

1- LEFT EARTH DAM.

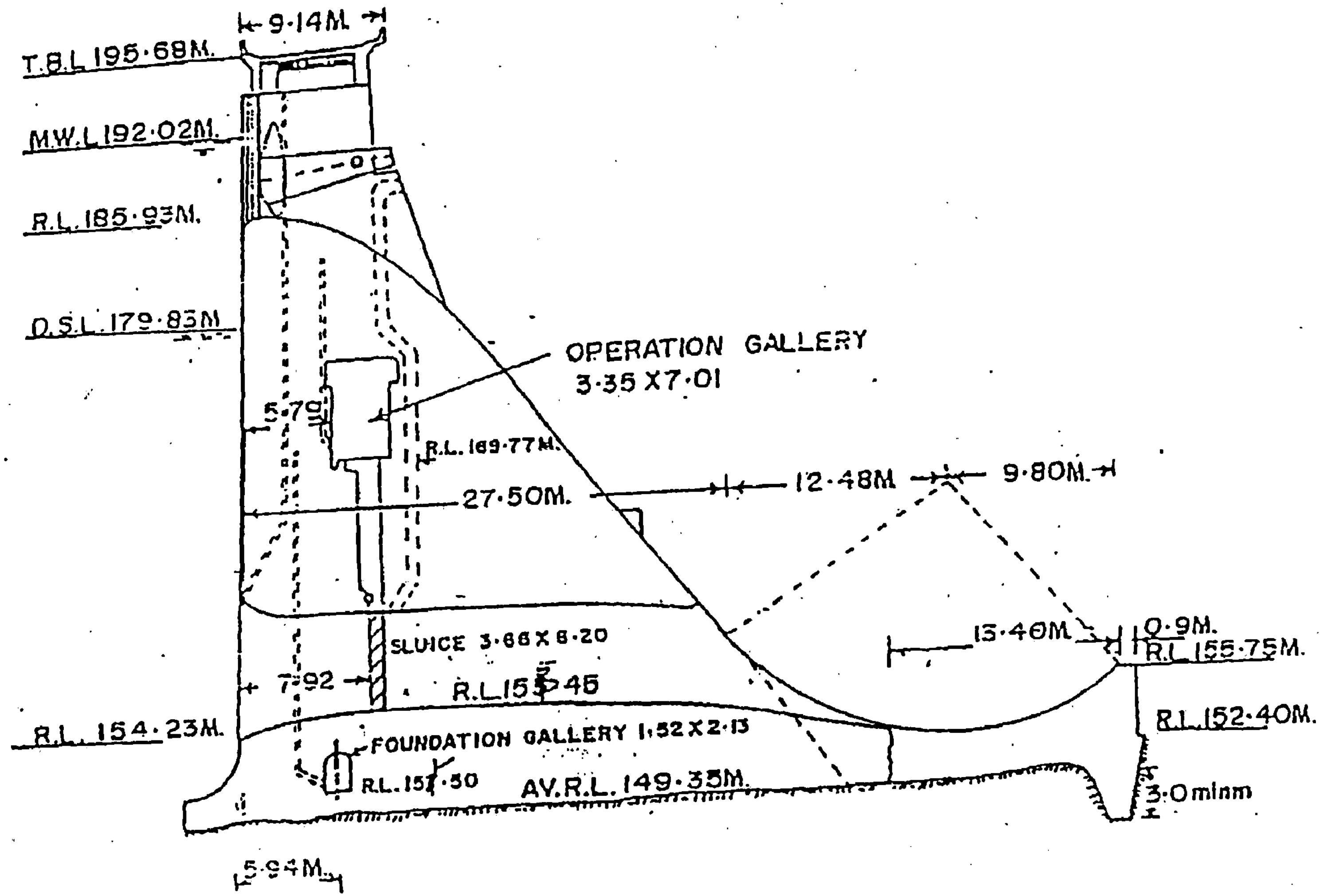
2 - LEFT SPILLWAY CONCRETE DAM.

3- CENTRAL EARTH DAM.

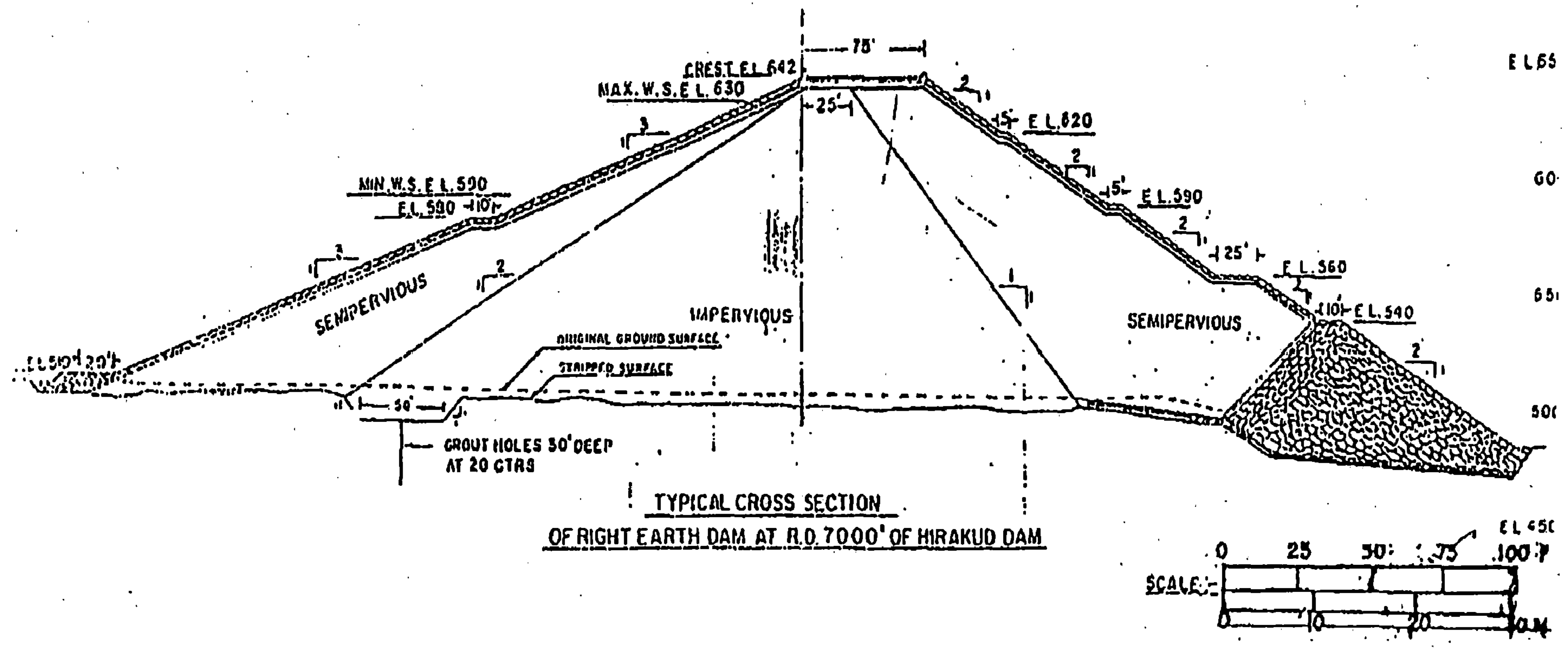
4 - RIGHT SPILLWAY CONCRETE DAM AFFECTED BY CRACKING

5 - POWER DAM.

Fig.4.3 Spillway Section of Hirakud Dam



MAXIMUM SPILLWAY SECTION



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

DAM BREAK ANALYSIS

5.1 AVAILABILITY OF DATA

The input data required for the NWS DAMBRK model can be categorized into two groups: The first data group pertains to the dam and the inflow hydrograph into the reservoir, and the second group pertains to the routing of the outflow hydrograph from the breached dam through the downstream valley.

5.1.1 First Data Group

With respect to the data group pertaining to the dam , the information on reservoir elevation- surface area relationship, spillway details, elevation of bottom and top of dam , elevation of water surface in the reservoir at the beginning of analysis and at the time of failure and breach description are required. The particulars of the data availability under each of the above mentioned categories are given below:

(a) Reservoir Elevation – Surface Relationship

The reservoir elevation-surface area relationship of Hirakud dam as input to the DAMBRK model is given below:

Table 5.1

AREA OF RESERVOIR AT DIFFERENT LEVEL

LEVEL (M)	AREA (M^2)
152.40	0
167.64	42970000
179.83	27766000
184.40	416490000
185.90	466460000
187.45	525480000
190.50	651910000
192.02	727310000
195.07	835680000
200.00	1460250000

(b) Spillway details

The spillway related information are required for the development of spillway rating table. Also under this category of data, information on the coefficient of uncontrolled weir flow is needed for computing the discharge due to overtopping of the dam . In addition the discharge through the sluices is also considered as part of the spillway discharge. When discharge through the sluices are considered the minimum head available corresponds to the Dead Storage Level i.e., EL. 179.88m The spillway rating table established considering discharge through different outlets corresponding to different water levels is given in Table 5.2.

Table 5.2

SPILLWAY AND SLUICES DISCHARGE AT DIFFERENT WATER LEVEL

WATER LEVEL (M)	DISCHARGE (CUMECS)
179.88	23745
181.40	24578
182.62	25230
184.45	26154
185.37	26625
185.98	26914
186.59	27349
187.20	28149
188.41	30463
189.02	31897
190.24	35272
190.85	37308
191.46	39410
192.07	41609
193.29	48294
194.51	51315
195.73	53841
196.04	54422
196.65	55535

(c) **Elevation Details**

Elevation of top of dam-	195.68m
Elevation of bottom of dam-	152.40m
Elevation of initial water surface level in the reservoir when the Computation begins -	193.45m

In this analysis, the water surface elevation in the reservoir corresponding to the beginning of computation and the elevation corresponding to the beginning of breach is considered the same.

(d) **Breach Description**

The profile of the breach used in the earlier analysis of the Hirakud dam using MIKE-11 is also considered as the required breach for this analysis. It is shown in Figure 5.1.

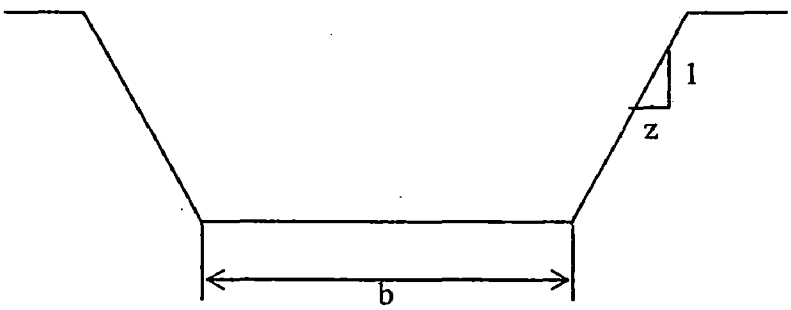


Figure 5.1 : Trapezoidal breach

The breach description for all the cases considered in the analysis is given in Table 5.3.

TABLE 5.3

Failure Modes and Parameters Adopted in Simulation for linear failure mode

CASE NO	Breach Level(m)		Breach Width(m)		Slope	Breach Development Time(Hr)
	Initial	Final	Initial	Final		
1	195.68	155.48	1	200	0.75	1
2	195.68	155.48	1	200	0.75	2
3	195.68	155.48	1	200	0.75	3
4	195.68	155.48	1	200	0.75	4
5	195.68	155.48	1	250	0.75	1
6	195.68	155.48	1	250	0.50	2
7	195.68	155.48	1	250	0.75	3
8	195.68	155.48	1	250	0.75	2

(e) Inflow Hydrograph

Inflow hydrograph used in the earlier dam break analysis of Hirakud dam is also used as the inflow in the present analysis and the same is given in Fig. 5.2.

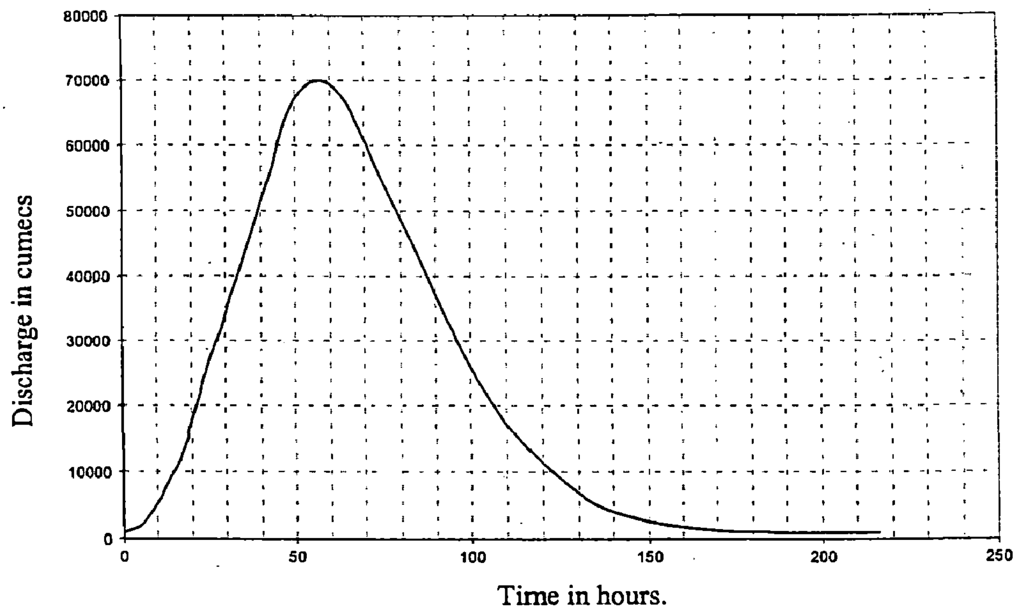


Fig. 5.2. Flood hydrograph (PMF) for Hirakud dam

5.1.2 Second Group of Data

The second group of data pertaining to the routing of outflow hydrograph through the downstream of the valley consists of the description of the cross section , hydraulic resistance coefficient and contraction-expansion coefficient of the reach, steady state flow in the river at the beginning of the simulation and downstream boundary condition . The cross sections are specified by its location downstream of the dam and using the tables of top width and corresponding elevation. In this study 39 numbers of cross sections are available at different location downstream of the dam which are depicted in Fig. 5.5. The Manning's roughness coefficient for the entire river channel reach is taken as 0.033 for all elevations.

5.2 ANALYSIS

This section describes the failure analysis of dam carried out using the DAMBRK model. Before analyzing the data using the DAMBRK model, some preliminary analysis for the formulation of input data required by the model was made. This analysis deals with the breach description of the dam. Also the assumptions involved in channel routing analysis of this dam break flood wave have been explained.

5.2.1 Comparison of DAMBRK Results with MIKE 11 Results

Following the guidelines of U.S. Federal Energy Regulatory Commission, the CWC (1999) in its analysis of Hirakud dam using MIKE-11 model has adopted a final breach bottom width as 200 m. The shape of the breach assumed is trapezoidal having a side slope of 0.75, and different breach development times of 1 hr, 2 hr, 3 hr and 4hr were used .Another trapezoidal breach size with the final bottom width 250 m having a side slope of 0.75was also used for the analysis. For this breach size, the breach development times of 1 hr,2hr and 3hr. have been used. In order to assess the sensitivity of the change in breach size, the side slope of the breach has been changed from 0.75 to 0.50 corresponding to the case breach development time of 2 hr and final width of 250m. In all the CWC has used eight cases of different breaches and these details are given in Table 5.3. The CWC has surmised that the breach case with the final breach width of 200 m, with a breach slope of 0.75 and the breach development time of 4 hr. corresponding to case 4 given in the breach description Table 5.3 is considered to give critical condition for the dam failure.

For this critical condition, MIKE-11 model has computed the maximum water level reached in the reservoir due to the application of the PMF hydrograph as 193.49m. The F.R.L. value of 192.07 m is considered as the initial water surface elevation when dam break computation begins using MIKE-II model. However, in the present study using DAMBRK model, it is considered that the computation as well as dam breach starts occurring at the level of 193.45m behind the reservoir and it is at the elevation the MIKE11model assumes the breach development. Fig.5.4 shows the maximum discharge computed from the breached dam at different locations and their time of occurrence respectively estimated by the DAMBRK model since the beginning of breach. The corresponding peak discharge values and its time of peak as simulated by the MIKE-II model are also shown in the respective figures for comparative purpose.

In the absence of any calculated flood from Ong and Tel rivers, the flows in Ong and Tel river have been arbitrarily assessed, by CWC, as 10% of the PMF of river Mahanadi upstream of Hirakud dam. This analysis has adopted the same lateral flow for Ong and Tel rivers to be added with DAMBRK estimated hydrograph.

The maximum water levels computed at different locations downstream of the dam as estimated by the DAMBRK model are shown in Fig 5.3. Corresponding maximum water levels at these locations due to MIKE11 model are also shown in Figure 5.3. Similarly the maximum discharges estimated at different along the channel due to passage of dam break flood are shown in Fig. 5.4.

5.2.2 Assessment of Dam Break Flood Wave Characteristics With and Without considering PMF Inflow

In the present study, dam break analysis is made with and without consideration of the probable maximum flood hydrograph entering upstream of the Hirakud dam in order to assess the effect of inflow hydrograph on the downstream flood wave characteristics from the breached dam. The inflow hydrograph used for the analysis was described in Section 5.1.1. The flood wave characteristics estimated in the form of peak water level elevations at different locations and down stream of the dam, with and without considering inflows are given in Table 5.4.

Table-5.4.
Maximum Water level computed using DAMBRK model at different locations downstream of the dam with and without considering PMF hydrograph

Chainage From Dam	Maximum Water Level for Non Impingement of PMF	Maximum Water Level for Impingement of PMF
0	168.52	168.54
3	166.56	166.58
6	164.29	164.32
11	161.22	161.25
15	159.22	159.24
20	149.33	149.34
25	147.21	147.24
30	142.87	142.90
35	142.03	142.06
40	140.48	140.51
45	138.71	138.88
60	131.90	131.93
76	125.75	125.78
80.75	122.35	122.39
85	120.56	120.60
92.68	116.88	116.91
95	115.28	115.32
105	109.10	109.14
113	106.95	106.99
125	101.62	101.66
137	95.79	95.87
145	94.83	94.91
155	93.22	93.30
163	91.55	91.69
170	90.08	90.17
180	86.92	87.02
190	77.39	77.53
200	74.53	74.66
214	65.23	65.27
220	65.61	65.67
225	63.62	63.69
226	63.15	63.23
238	58.05	58.12
250	53.05	53.14
260	50.07	50.20
269	47.53	47.70
284	44.53	44.77
300	41.08	41.39
310	36.49	37.35

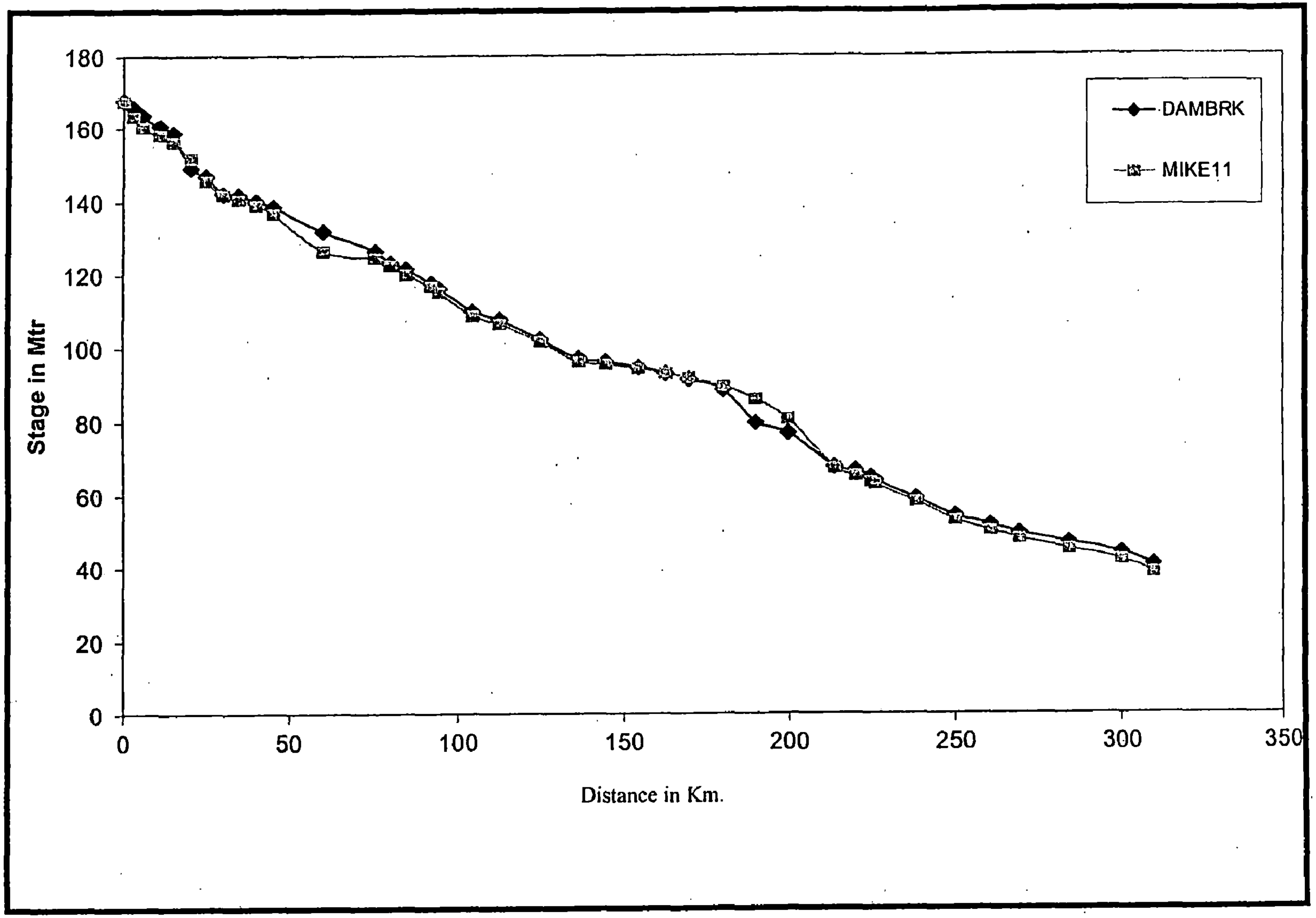


FIG. 5.3. PEAK FLOOD ELEVATION PROFILE ESTIMATED FOR HIRAKUD HYPOTHETICAL DAM FAILURE (BREACH CASE NO.4)

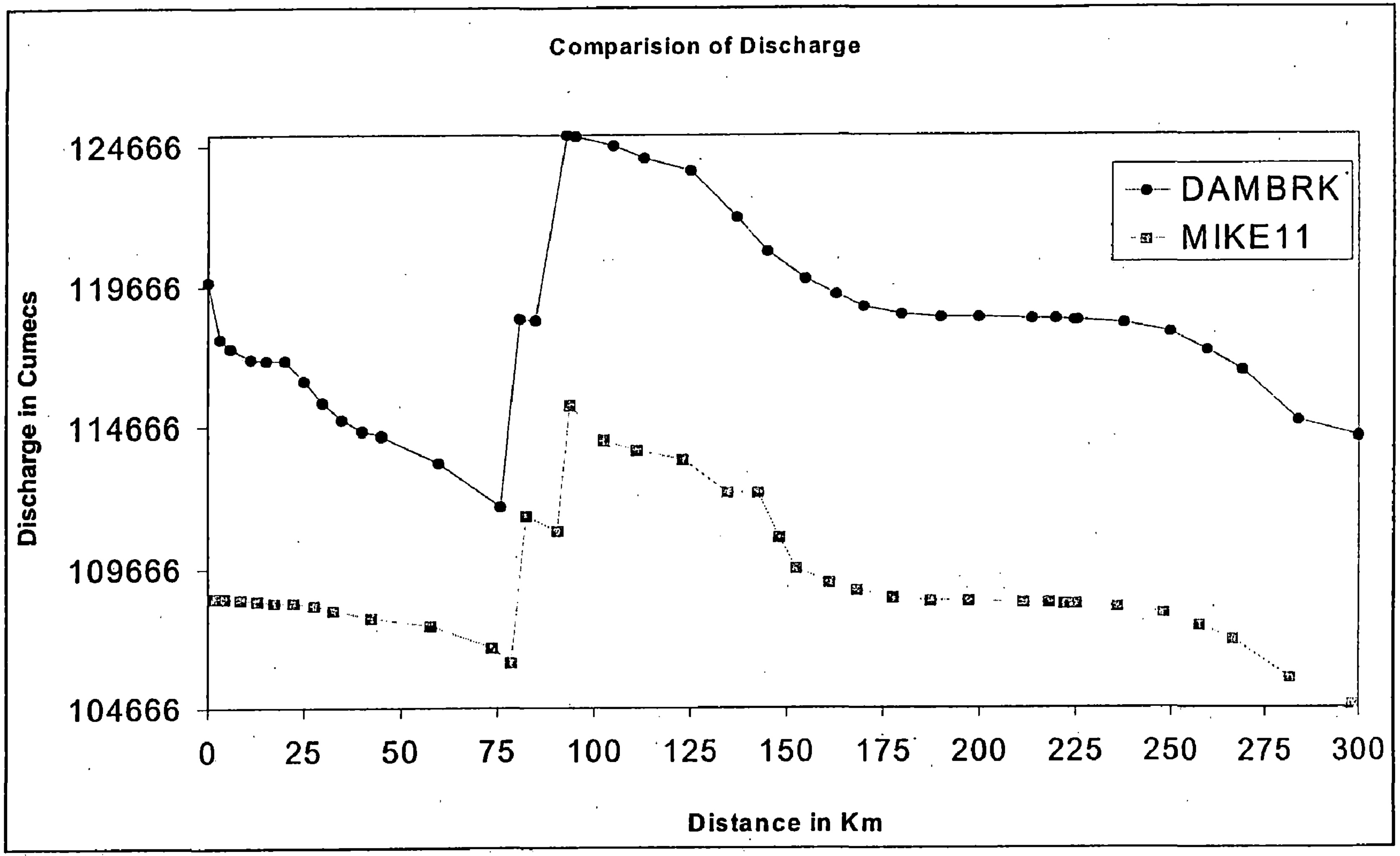


FIG. 5.4. PEAK FLOOD DISCHARGE PROFILE ESTIMATED FOR HIRAKUD DAM FAILURE (BREACH CASE NO.4)

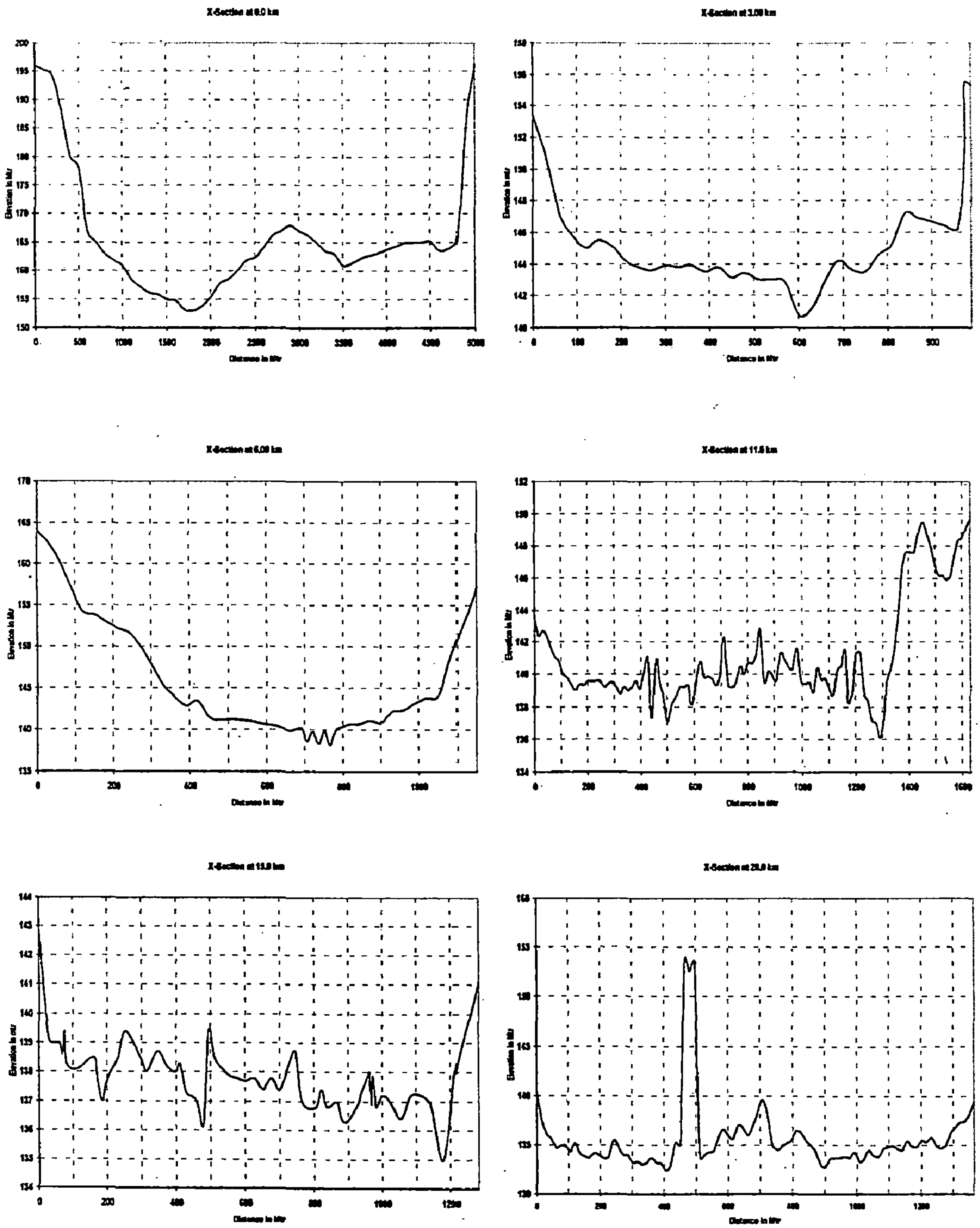


Fig. 5.5.Plate No-2.
River Cross Section at 0.0 km
3km,6km,11km,15km,20km

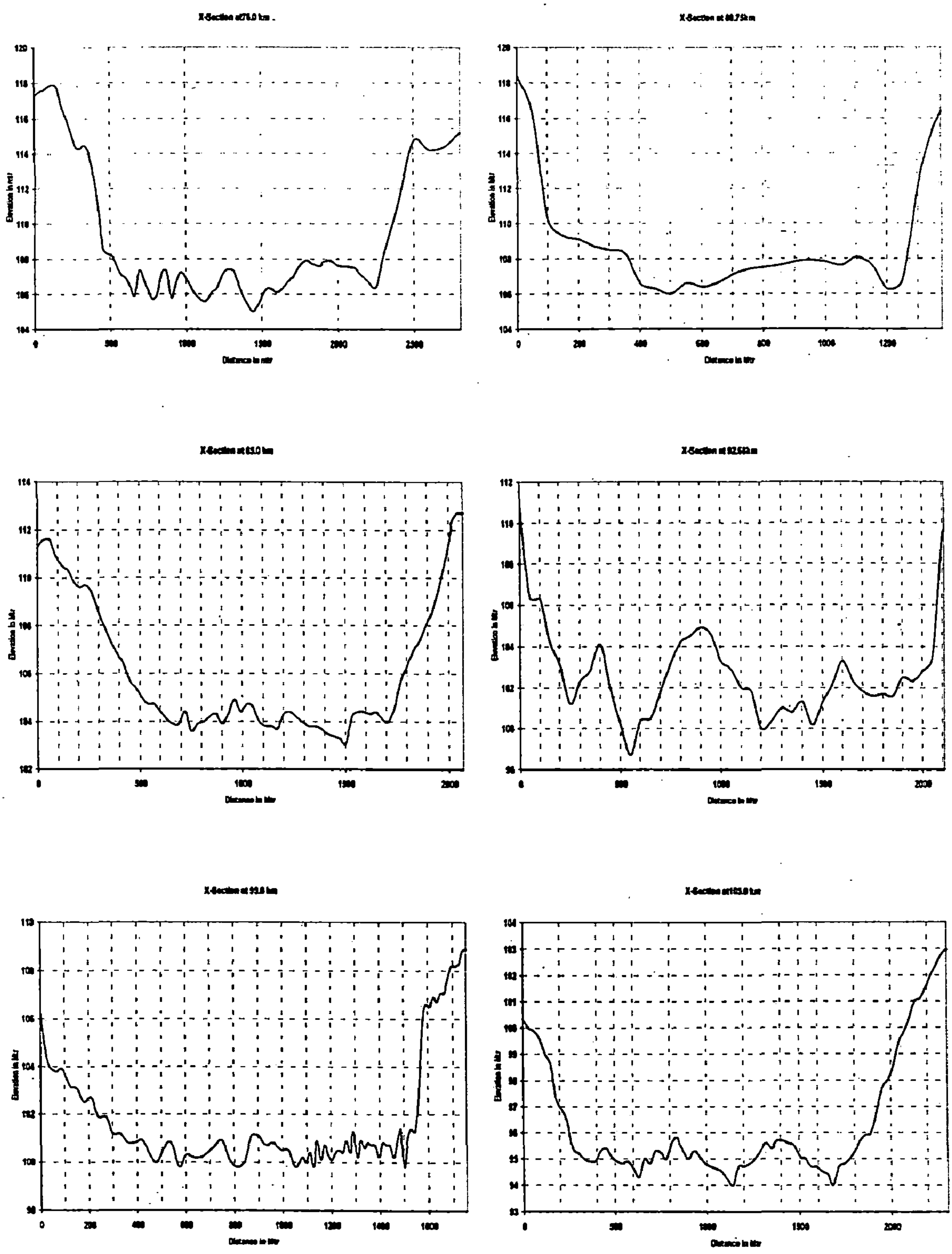


Fig. 5.5. Plate No-2
River Cross section at
25.0km,30.0km,35.0km
40.0km,45.0km,60.0km.

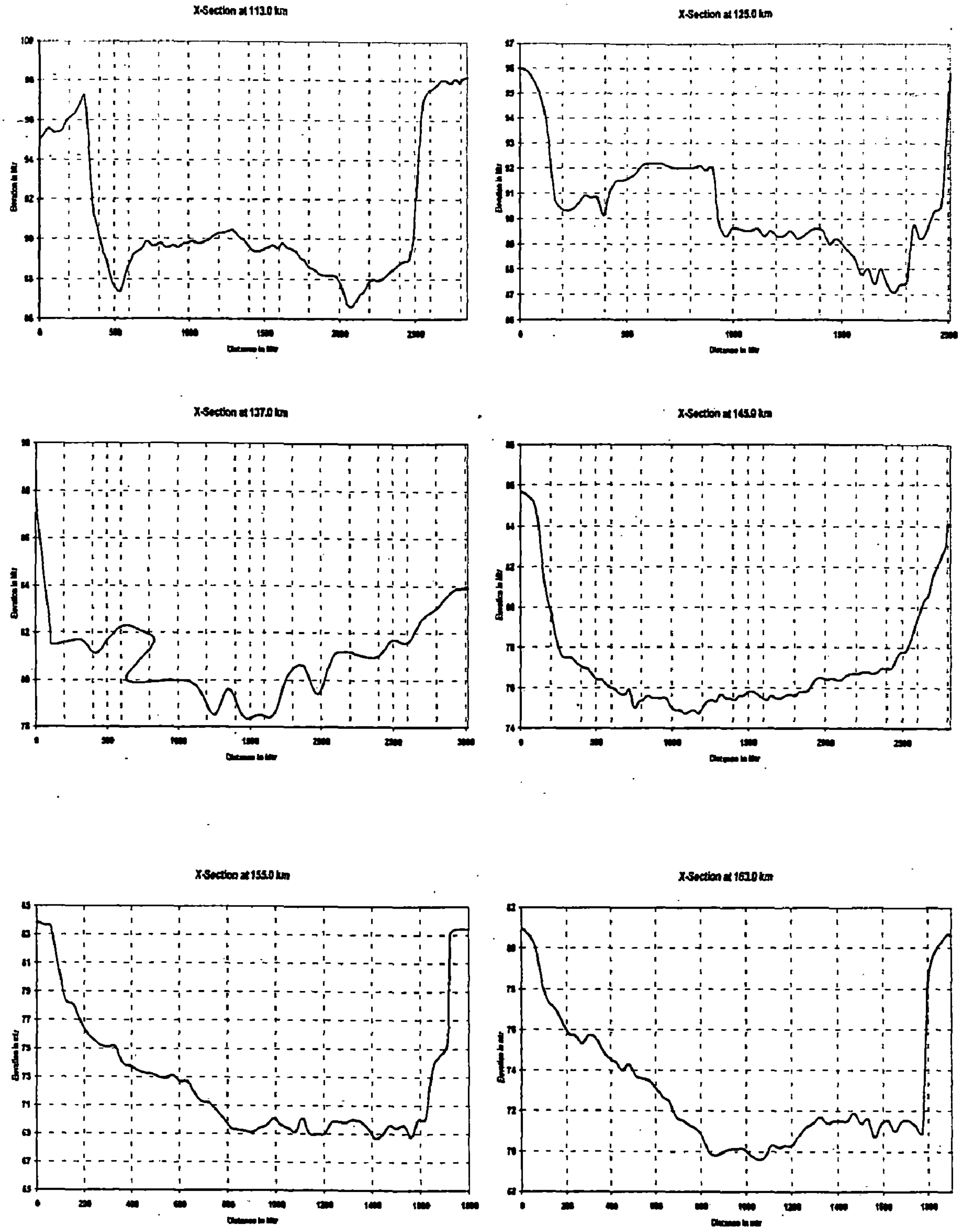


Fig. 5.5.Plate No-2
River Cross Section at
113km,125km,137km,
145km,155km,163km

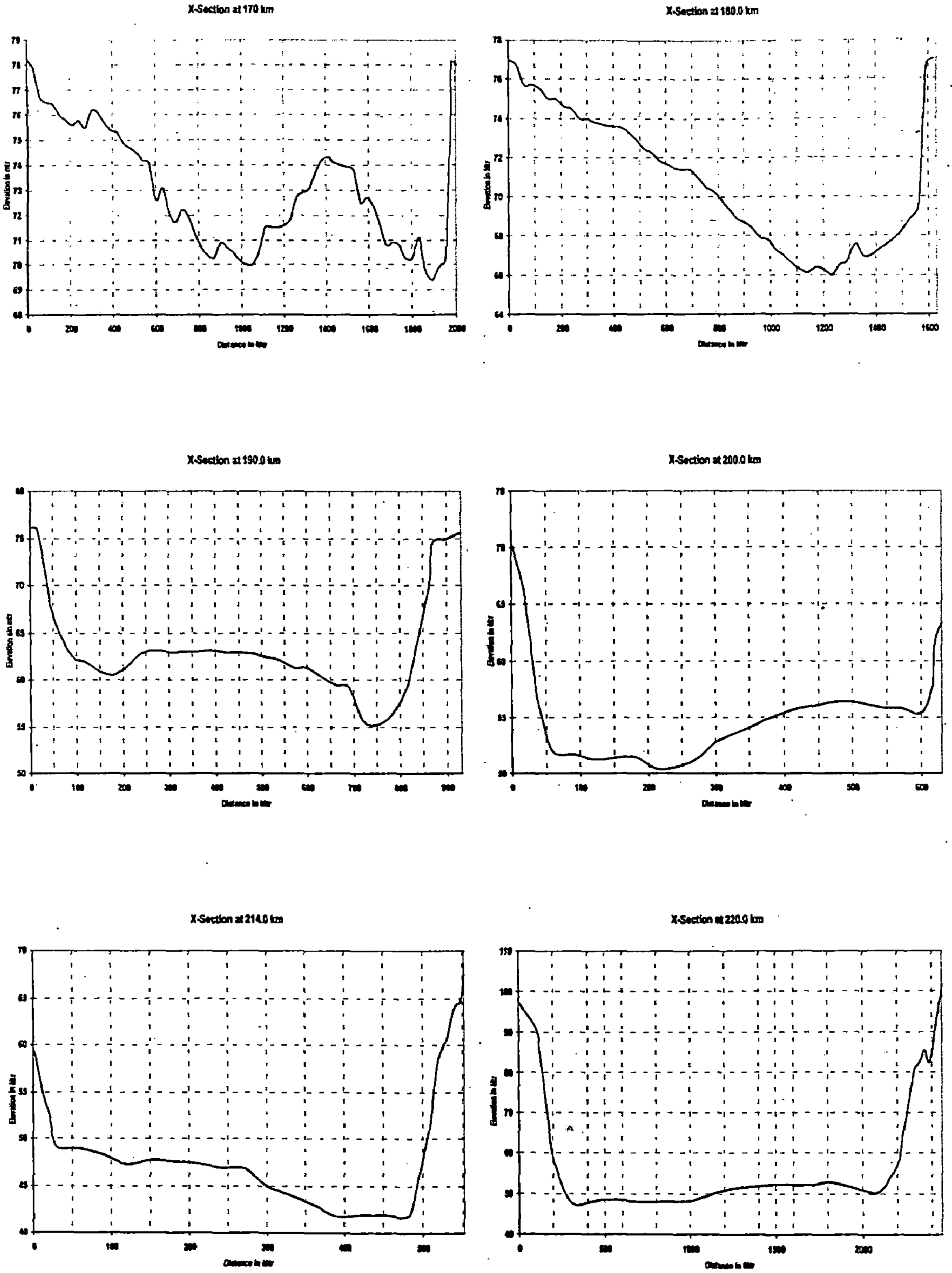


Fig. 5.5. Plate No-2
River Cross section at170km,180km,190km,
200km,214km,230km

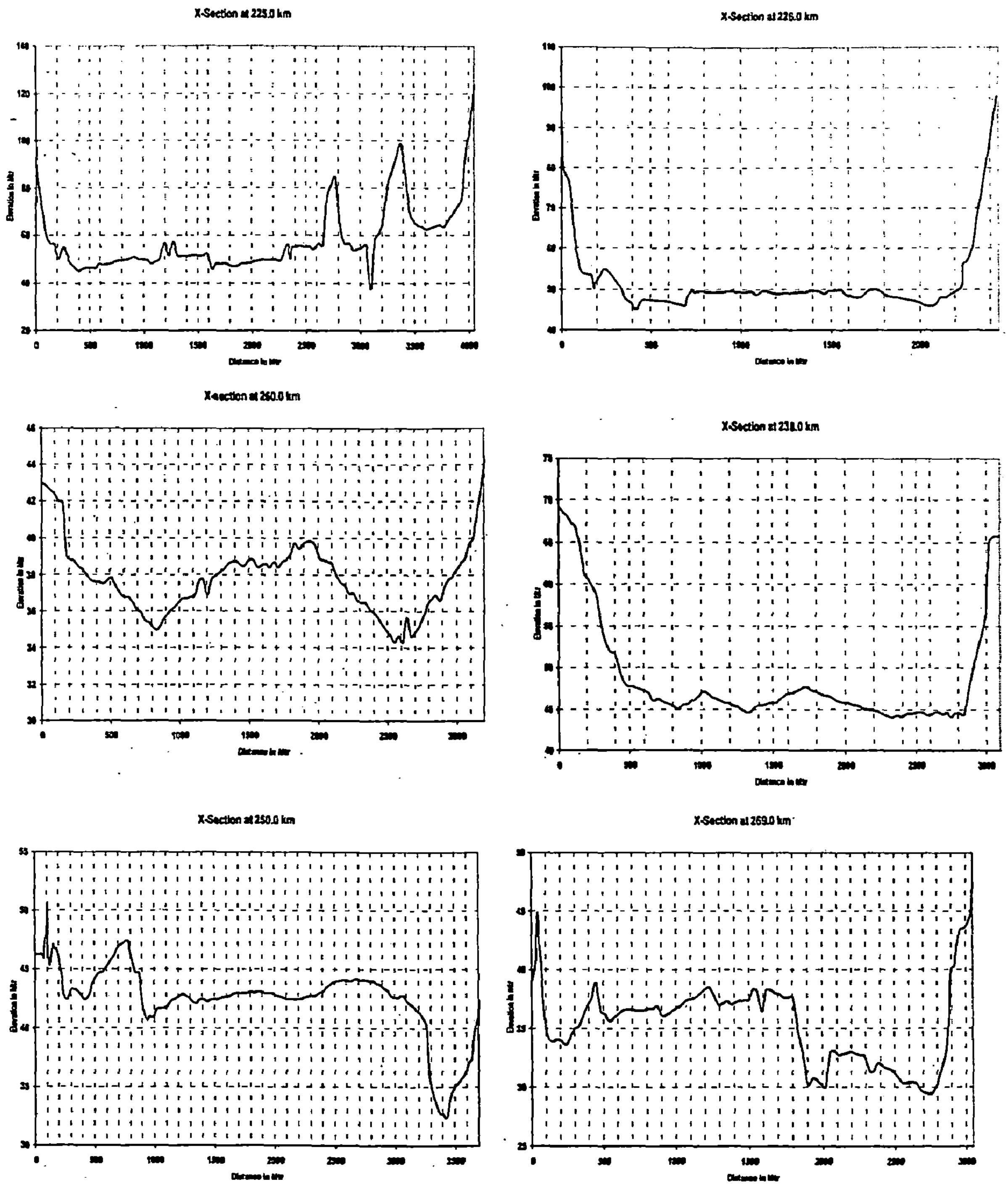
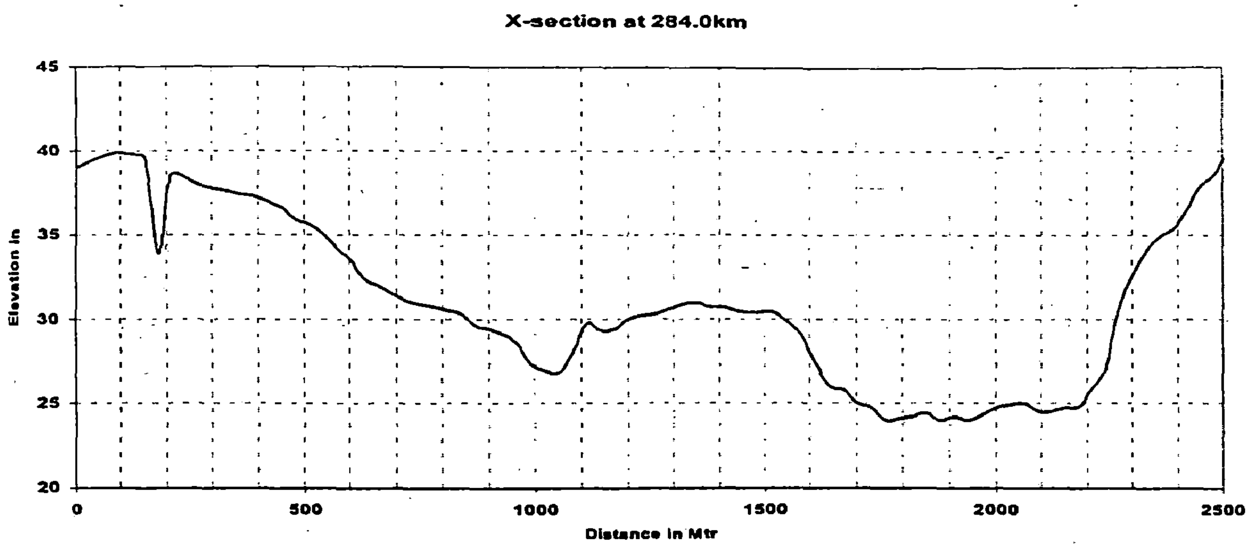


Fig. 5.5. Plate No-2
River Cross Section at
235km,236km,238km,
250km,260km,268km



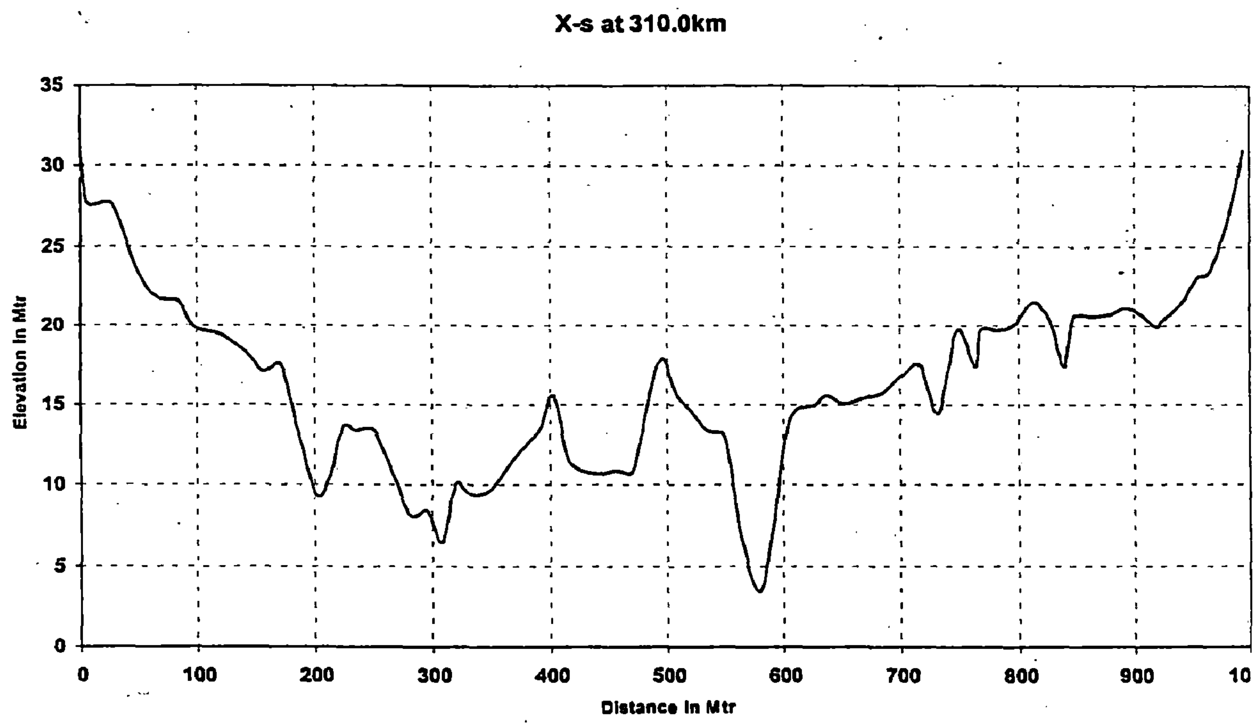


Fig. 5.5. Plate No-2
River Cross Section at
284km,310km

DISCUSSION OF RESULTS

6.1 INTRODUCTION

In this chapter, the results of the present study of hypothetical dam failure analysis of Hirakud dam using DAMBRK model are discussed. The study was taken up to compare the relative performance of the NWS DAMBRK model for the hypothetical failure analysis of Hirakud dam with the corresponding result of the MIKE-11 model using the same input information. The study was also taken up to assess the flood wave characteristics of the hypothetical failure of the Hirakud dam using DAMBRK model with and without considering the designated inflow hydrograph entering upstream of the reservoir, when the other physical conditions at the dam and its downstream remaining the same.

6.2 COMPARISON OF PERFORMANCES OF DAMBRK AND MIKE11 MODELS

6.2.1 Sensitivity Analysis

While evaluating the performance of MIKE11 model, Central Water Commission carried out sensitivity analysis by changing one parameter at a time to know its impact on the estimated dam break flood hydrograph at the dam site and at various downstream locations. It is obvious that dam break flood wave formation is due to formation of breach in the dam. Only linear failure formation breach formation in MIKE11 and DAMBRK model have the same logic behind their development and therefore, for comparative purposes of both models performances the option for the same breach formation technique should be used. Accordingly, eight different cases breach sizes as described in Table 5.3. were used for dam break analysis using DAMBRK model.

It is seen from Table 6.1. showing the maximum water levels reached at different locations downstream of the dam that for shorter breach time relatively lower maximum water level is reached at any location in comparison with the longer breach time, and this level is reached in a relatively shorter time than in the case of longer breach time. This comparison is based on the condition that breach size remains same and the breach time to attain this size increases.

However, when breach size increases from 200 m (corresponding to cases 1 – 4) final bottom breach width to 250 m (corresponding to cases 5 – 8) final bottom breach width, the maximum water level reached at these same locations is higher than that reached in the case of breach width having 200 m final bottom breach width. The time to reach these maximum water levels are also comparatively lesser. Table 6.1 clearly shows that the maximum water levels reached at different locations are not significantly affected by the breach time variations. However, there affected by breach size variation.

A comparison of the results of sensitivity analysis with DAMBRK model and MIKE11 model reveal that the former model estimates slightly higher maximum water levels at different locations for all the eight cases of breach sizes studied.

6.2.2 Discussion of Results of DAMBRK Floods due to Critical Condition Breach

It is seen from Fig 5.4showing the maximum discharges estimated at different locations downstream of Hirakud dam, corresponding to the dam break outflow hydrograph due to breach formation leading to critical condition, that DAMBRK model estimates higher discharges all along the 310 km stretch of the river, by about 10,000 m^3/sec.In comparison with the corresponding estimates of MIKE 11 model (also shown in Fig. 5.4). However, the pattern of variation of discharge profile is nearly identically similar for both DAMBRK and MIKE 11 results.

Fig 5.3 shows the profile of the maximum water levels reached all along the 310 km stretch of the river, corresponding to the outflow hydrograph due to breach formation leading to critical condition. Fig 5.3 also shows the corresponding results of MIKE11 model as reported by the CWC (1999). Though both these results are overlapping in the figure, it is seen that maximum water levels reached due to DAMBRK model is slightly higher than the corresponding MIKE II results.

All these results suggest that given the identical breach conditions and other physical conditions at dam and in the river reach remaining the same, the DAMBRK model has a tendency to estimate discharges and stages slightly on the higher side in comparison with the corresponding estimates of MIKE II.

6.3. DISCUSSION OF RESULTS FOR ASSESSMENT OF FLOOD WAVE CHARACTERISTICS WITH AND WITHOUT CONSIDERATION OF PMF INFLOWS

The study carried out with the objective of assessing the sensitivity of the dam break flows due to dam break analysis with and without consideration of the PMF inflows resulted in table 5.4 showing the maximum water levels estimated at different locations due to the application of both models. It is seen that there is no significant difference in the values of maximum stage computed with and without the PMF inflows consideration, and similarly for the values of peak flow, at dam locations and at specific sites downstream of the dam. Though significant volume of flow has entered into reservoir, it has not contributed much for the increase of maximum stages recorded at downstream locations significantly. The possible reason which can be contributed for this small difference in level may be that the released water behind the dam due to failure has occupied a larger area of floodplain downstream of the dam, and the routed inflow hydrograph volume through the breached dam has spread over and above the vast floodplain already submerged, thus causing a very small increase in the maximum stage estimated over and above the maximum stage estimated due to the release of stored water behind the dam.

6.4. CONCLUSIONS

Comparative evaluations of the performance of the DAMBRK model and the MIKE11 model for the identical input conditions reveal that the DMBRK model estimates slightly higher discharge than the MIKE11 model.

The envelope curves of the maximum discharges estimated at different locations downstream of the dam by both models show identical similarity and the DAMBRK model estimates a slightly higher maximum discharge in a consistent manner. In the case of envelope curves of the maximum water level elevations estimated at different locations downstream of the dam by both models show no significant difference in the estimated values. Under these circumstances, the preference of MIKEII model for dam break analysis over the DAMBRK model is purely based on personal preferences as there is no technical difference between the performances of these two models.

In a similar way, it may be concluded that there is no significant difference in the maximum stages estimated in the channel reach downstream of the dam by the DAMBRK model for the two cases of with and without considering the designated PMF inflow for dam break analysis. Therefore, one may consider the case of dam break analysis when the reservoir water level is at the top of the dam at the time of failure and without using the PMF hydrograph, as inflow into the reservoir are adequate for preparing flood inundation map required for the Emergency Action Plan.

Table-6.1.

(HIRAKUD DAM – DAM BREAK STUDIES (SENSITIVITY ANALYSIS)

Maximum water level and its time of occurrence after the impingement of the flood hydrograph due to dam breach at selected chainage for different cases

	MAXIMUM WATER LEVEL IN "M" AT CHAINAGE					TIME OF OCCURRENCE IN Hrs- Min AT CHAINAGE				
Case no	3.00	15.00	35.00	45.00	60.00	3.00	15.00	35.00	45.00	60.00
1	165.9	158.5	141.6	138.4	131.8	52.55	52.9	56.57	57.25	59.28
2	166.6	158.6	141.7	138.4	131.9	53.4	53.7	57.32	58.01	60.08
3	166.0	158.6	141.7	138.5	131.9	54.25	54.55	57.98	58.64	60.74
4	166.0	158.7	141.80	138.5	132.0	55.1	55.5	58.79	59.48	61.32
5	167.8	160.9	143.3	139.8	133.0	52.72	53.02	56.10	56.55	58.54
6	167.6	160.6	143.1	139.7	132.9	53.48	53.28	56.95	57.39	59.36
7	168.0	161.2	143.4	139.9	133.1	54.4	54.55	57.62	58.07	60.10
8	168.1	161.2	143.5	139.9	133.1	55.3	55.3	58.36	58.82	60.89

Table-6.1.

HIRAKUD DAM- DAM BREAK STUDIES (SENSITIVITY ANALYSIS)

Maximum water level and its time of occurrence after the impingement of the flood hydrograph due to dam breach at selected chainage for different cases

	MAXIMUM WATER LEVEL IN "M" AT CHAINAGE					TIME OF OCCURRENCE IN Hrs- Min AT CHAINAGE				
Case no	85.00	105.00	125.00	145.00	180.00	85.00	105.00	125.00	145.00	180.00
1	121.40	109.9	102.4	96.2	88.5	61.07	63.32	65.75	70.10	72.33
2	121.5	109.9	102.4	96.2	88.5	61.92	63.91	66.44	70.78	72.99
3	121.5	109.9	102.5	96.2	88.5	62.59	64.69	67.16	71.96	73.16
4	121.5	110.0	102.5	96.3	88.6	63.16	65.45	67.95	72.36	74.12
5	122.6	110.9	103.3	97.3	89.8	60.53	62.53	64.99	68.97	71.16
6	122.5	110.8	103.2	97.2	89.7	61.33	63.29	66.02	70.11	71.80
7	122.7	111.0	103.4	97.4	89.9	61.9	63.39	66.28	70.41	72.07
8	122.7	111.0	103.4	97.5	89.9	62.73	64.57	66.94	71.02	73.21

Table-6.1.

HIRAKUD DAM- DAM BREAK STUDIES (SENSITIVITY ANALYSIS)

Maximum water level and its time of occurrence after the impingement of the flood hydrograph due to dam breach at selected chainage for different cases

	MAXIMUM WATER LEVEL IN "M" AT CHAINAGE					TIME OF OCCURRENCE IN Hrs- Min AT CHAINAGE				
Case no	200.00	220.00	260.00	300.00	310.00	200.00	220.00	260.00	300.00	310.00
1	76.50	66.6	51.4	43.9	40.6	72.93	73.53	79.23	82.42	82.42
2	76.6	66.6	51.5	44.0	40.6	73.59	74.18	79.9	83.11	83.11
3	76.6	66.6	51.5	44.0	40.7	74.36	74.96	80.73	83.95	83.97
4	76.7	66.7	51.5	44.1	40.7	74.71	75.30	81.61	84.15	84.15
5	78.3	67.4	52.5	45.9	42.9	71.71	72.25	78.83	81.25	81.25
6	78.1	67.4	52.4	45.8	42.7	72.36	73.48	79.55	81.97	81.97
7	78.4	67.5	52.6	46.1	43.1	73.17	73.73	80.37	82.81	82.81
8	78.5	67.5	52.6	46.1	43.1	73.75	74.3	80.93	83.37	83.37

CONCLUSIONS

In this study dam break analysis for Hirakud dam was performed using NWS DAMBRK model for various hypothetical dam failure scenarios with the objective of

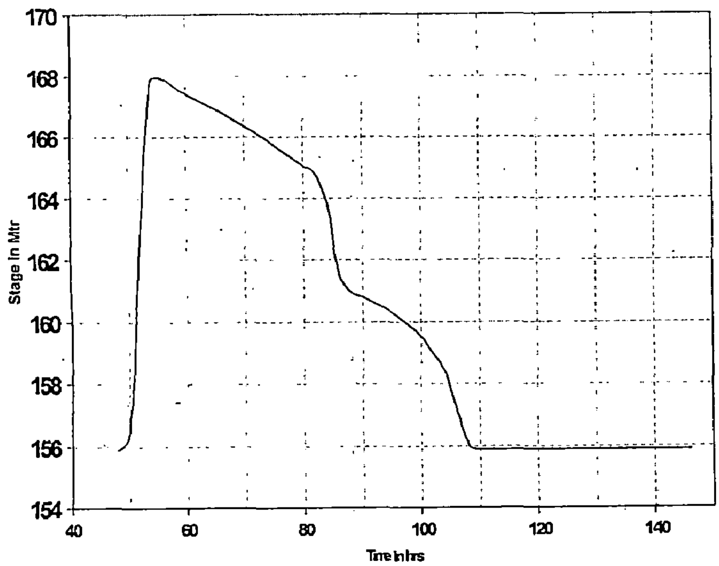
1. Comparing the performance of NWS DAMBRK model with that of MIKE11 model using the same input data information as has been used by the Central Water Commission (1999) for studying the hypothetical dam break of the Hirakud dam using MIKE-11 model, and
2. To assess the flood wave characteristics of the hypothetical failure of the Hirakud dam using DAMBRK model with and without considering the PMF inflow hydrograph entering upstream of the reservoir, when other physical conditions at the dam and its downstream remaining the same.

The studies carried out in this work with these objectives reveal that the DAMBRK model consistently estimates slightly higher discharge than the MIKE11 model from the breached dam and this is reflected on the maximum discharges and maximum water level elevations estimated at different locations downstream of the breached dam.

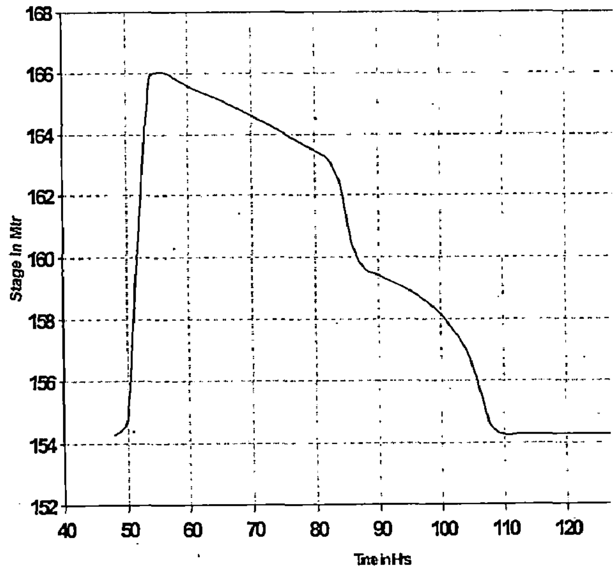
It may be concluded from the study carried out with the second objective that there is no significant difference in the estimates of maximum stages arrived at various locations downstream of the breached dam with and without using the designated PMF hydrograph. Therefore, one may consider the results of the case of dam break analysis, when the reservoir water level is at the top of the dam at the time of failure and without considering the PMF hydrograph as inflow into the reservoir, are adequate for preparing flood inundation map required for the Emergency Action Plan.

Stage Hydrograph of Different Chainages Down Stream of Hirakud Dam of Case No. 4

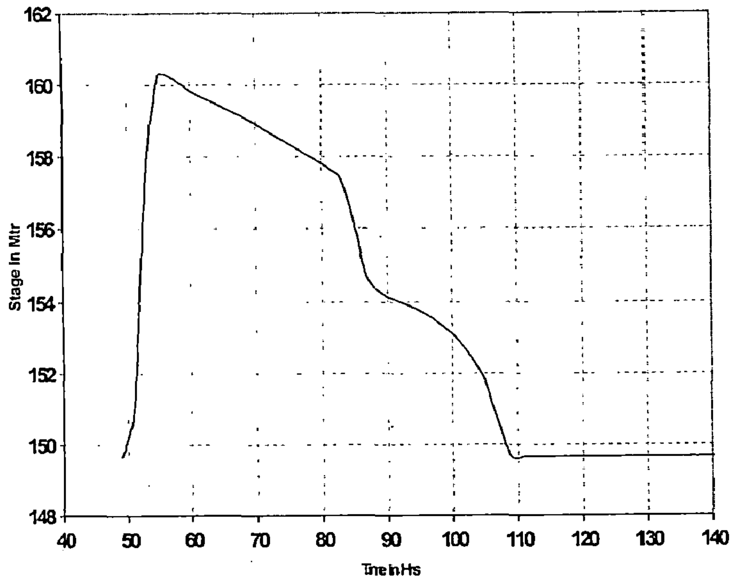
x-section at 0.0km



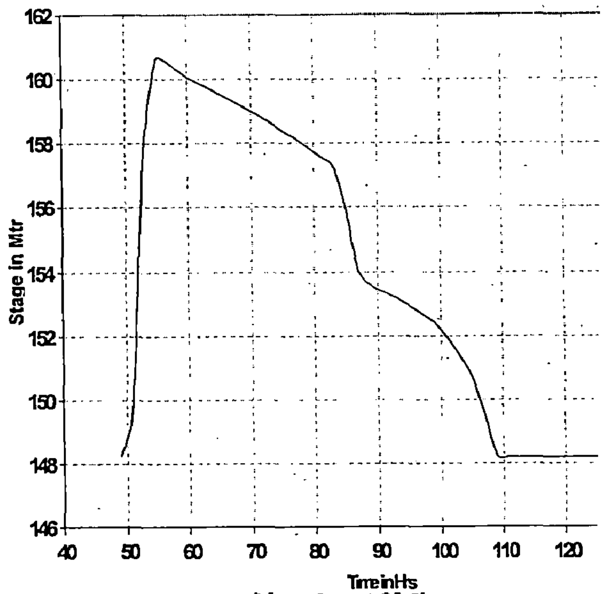
X-section at 3.0 km



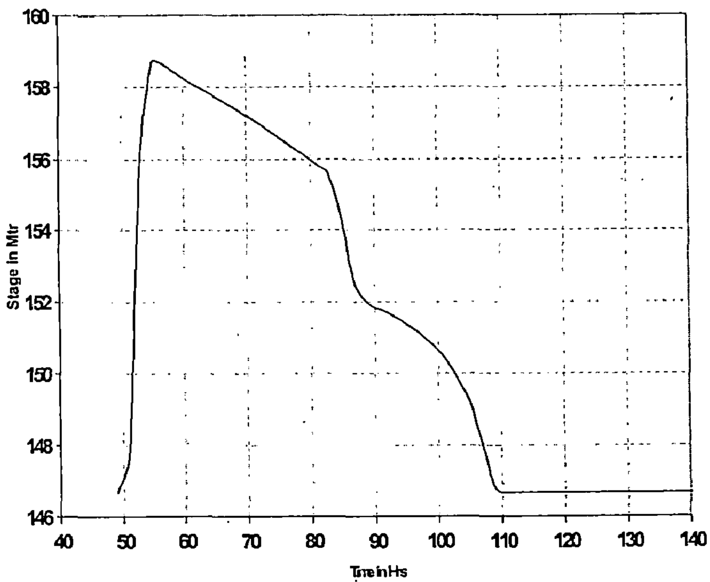
X-section at 8.00km



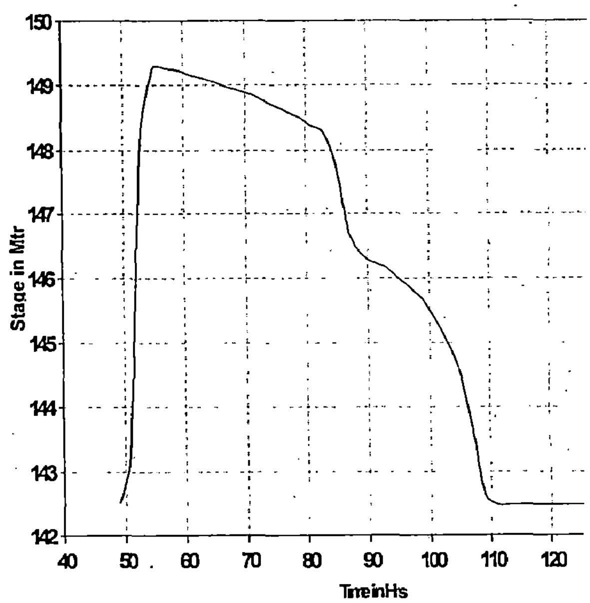
X-section at 11.0 km



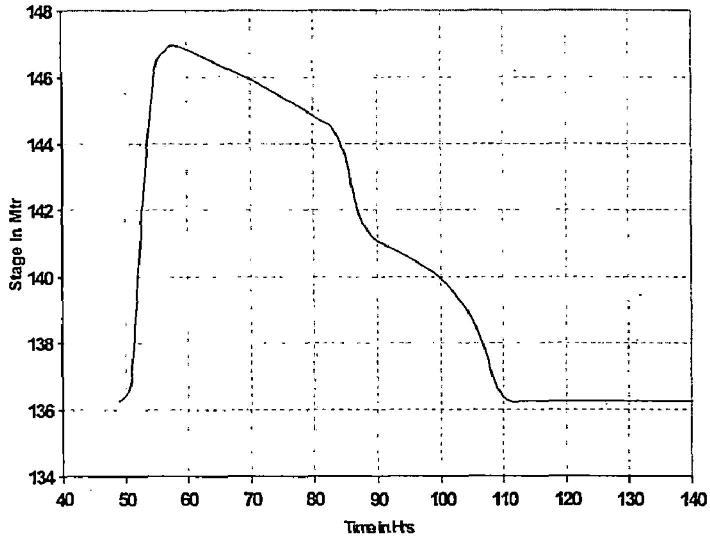
X-section at 15.0km



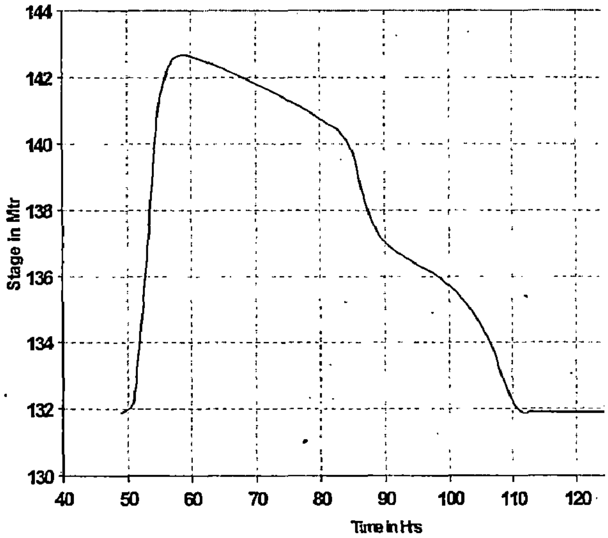
X-section at 20.0km



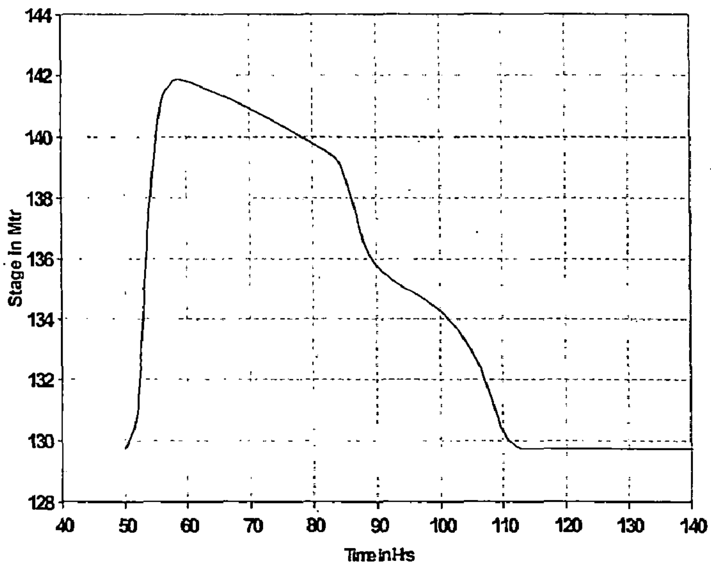
X-section at 25.0 Km



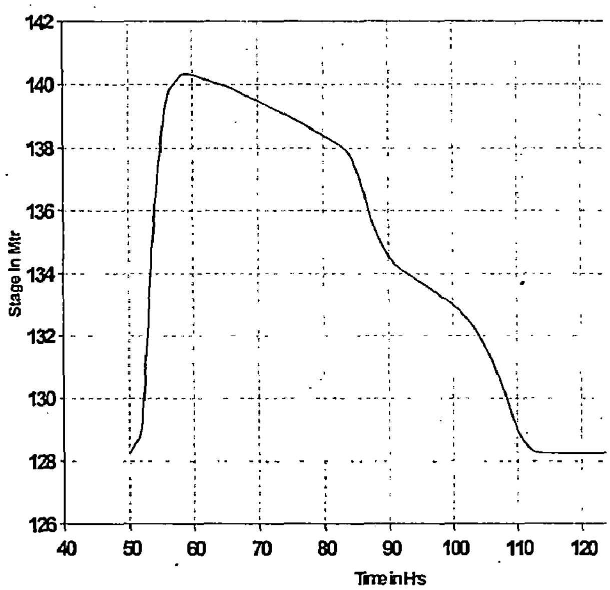
X-section at 30.0 km



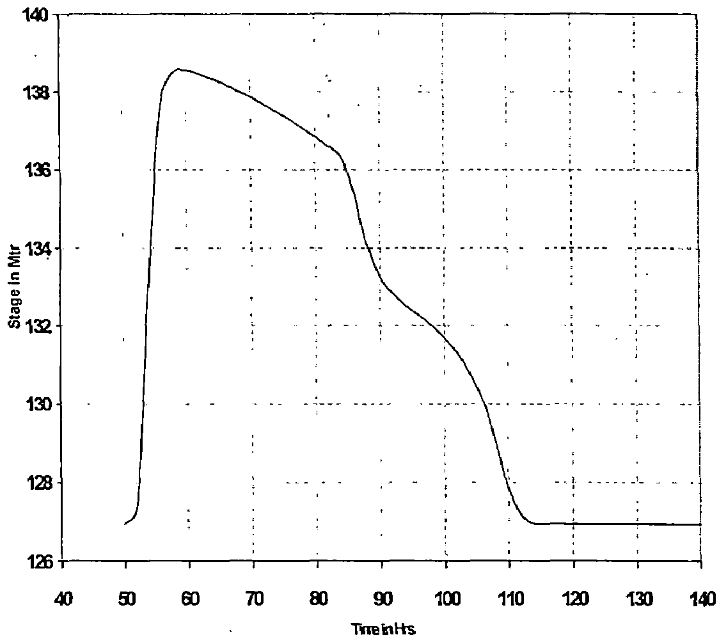
X-section at 35.0 km



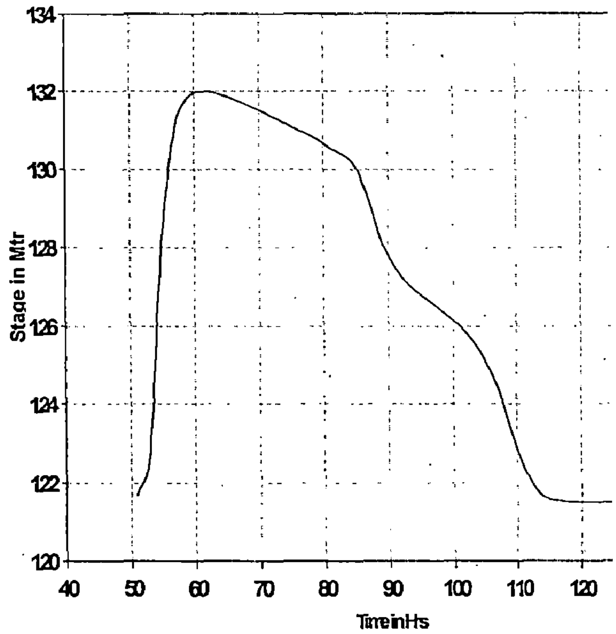
X-section at 40.0 km



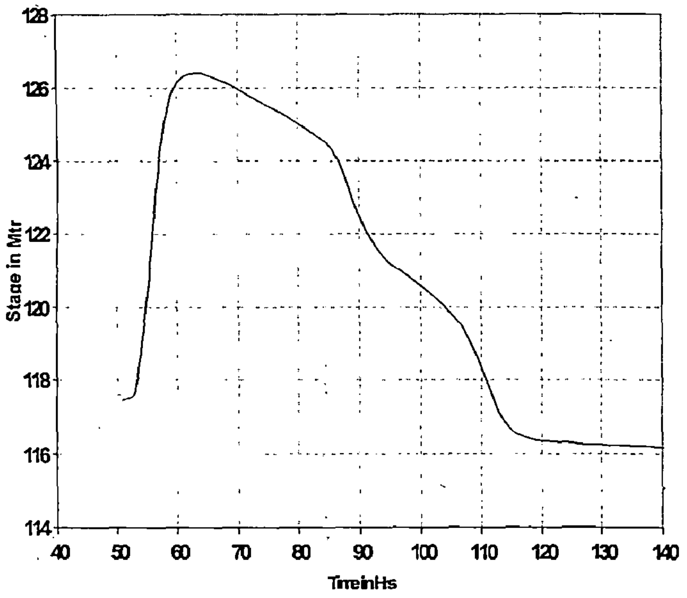
X-section at 45.0km



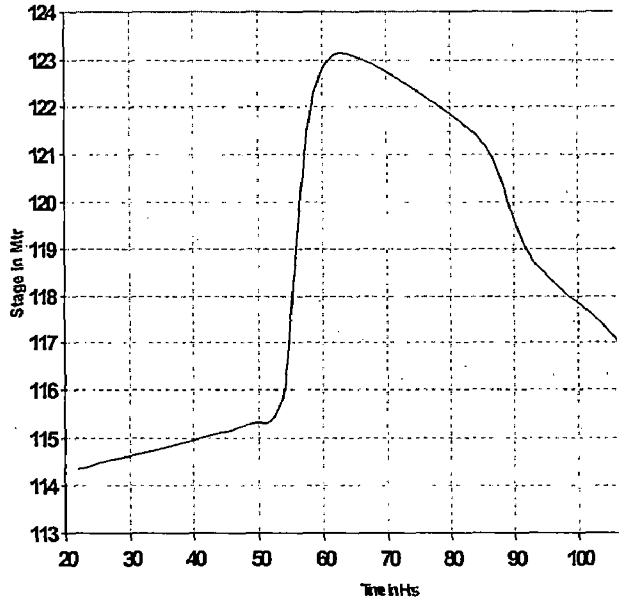
X-section at 60.0 km



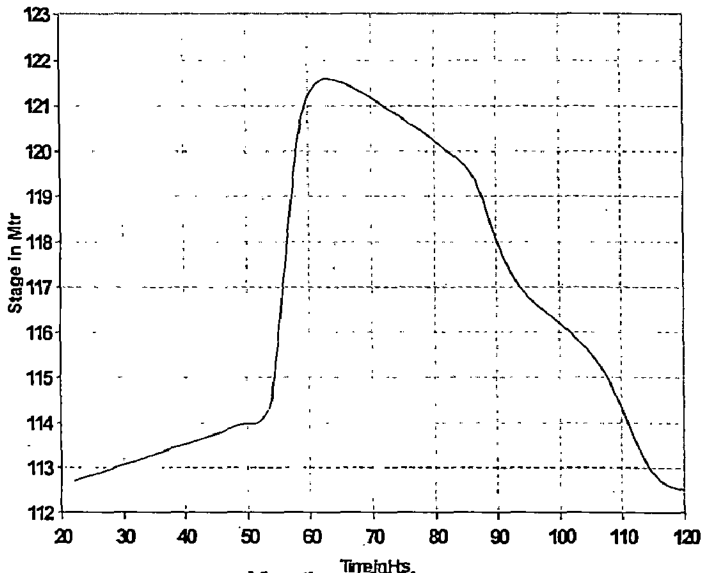
X-section at 76.0 km



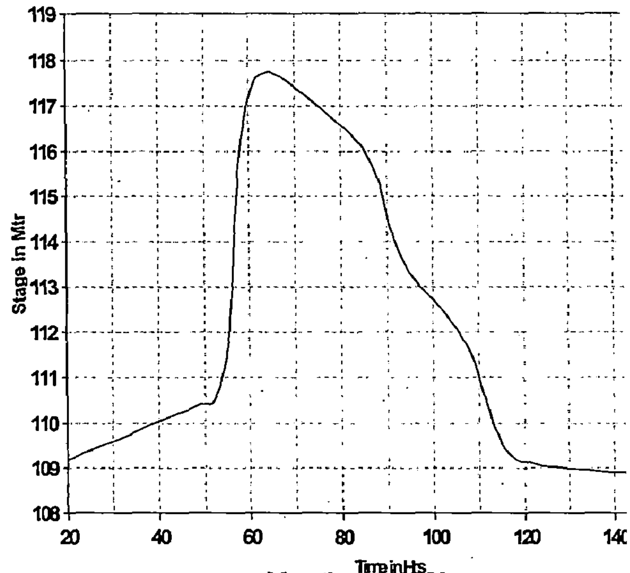
X-section at 80.75 km



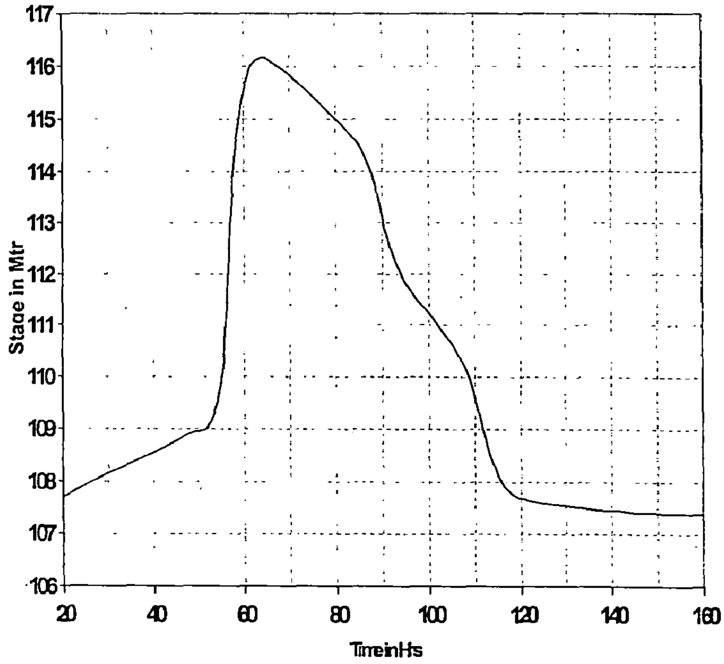
X-section at 85.0 km



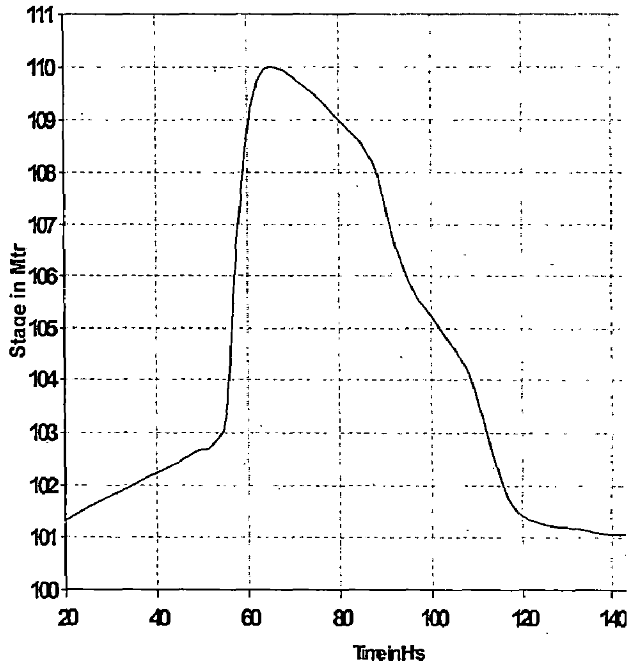
X-section at 92.68 km



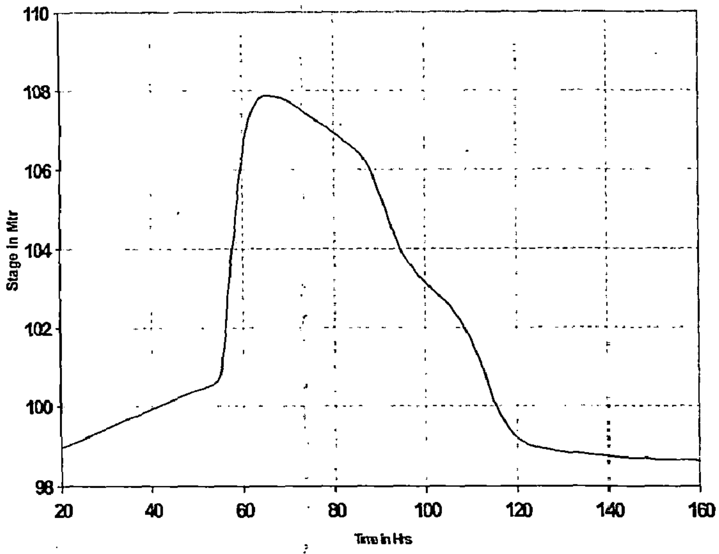
X-section at 95.0 km



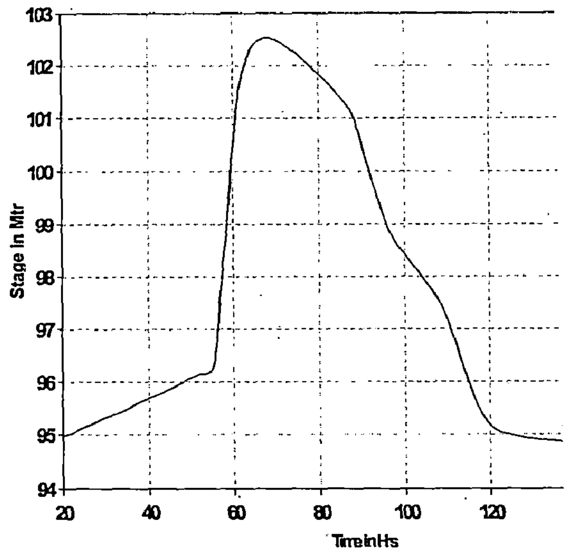
X-section at 105.0 km



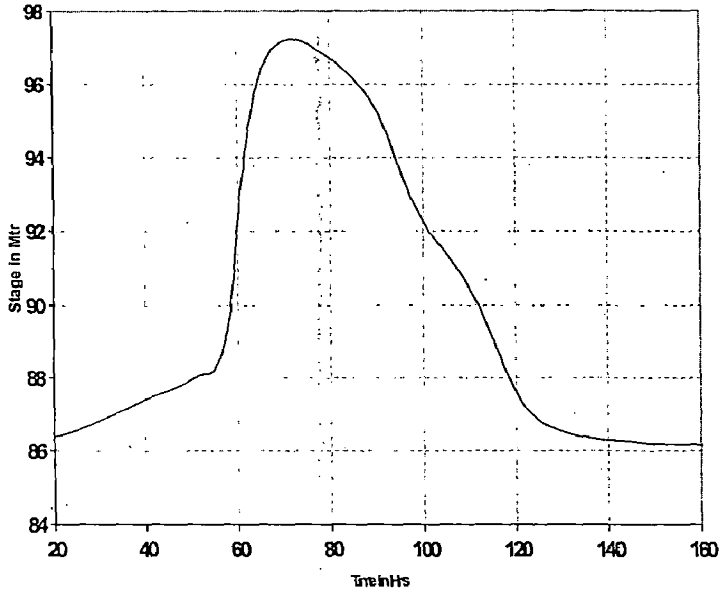
X-section at 113.0 km



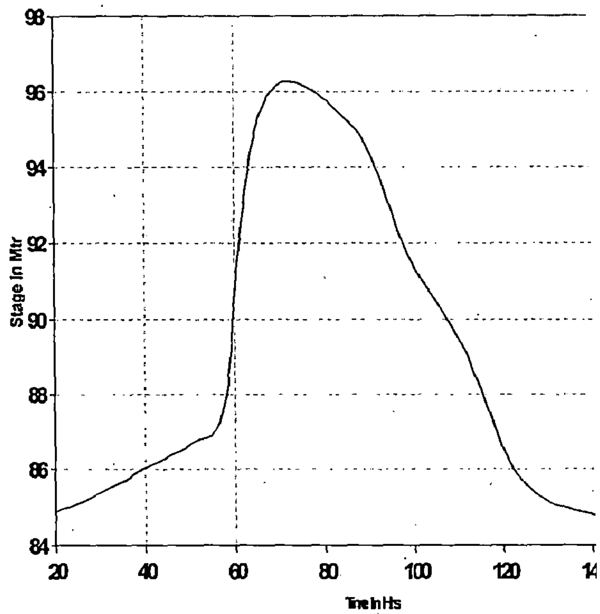
X-section at 125.0 km



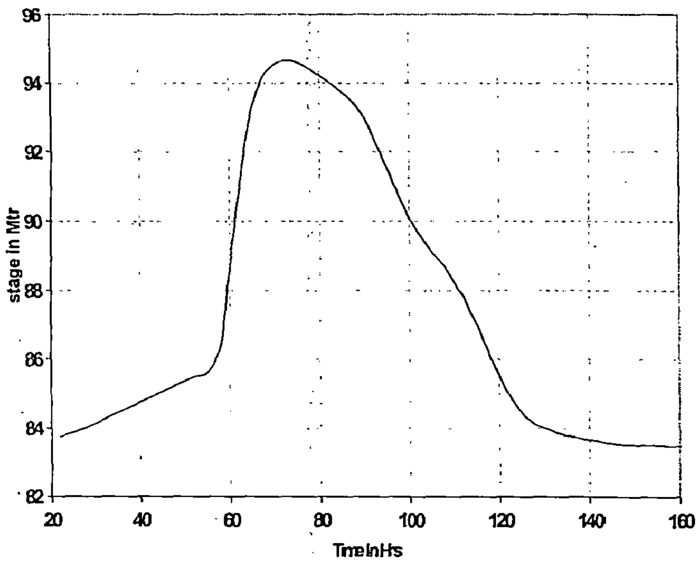
X-section at 137.0 km



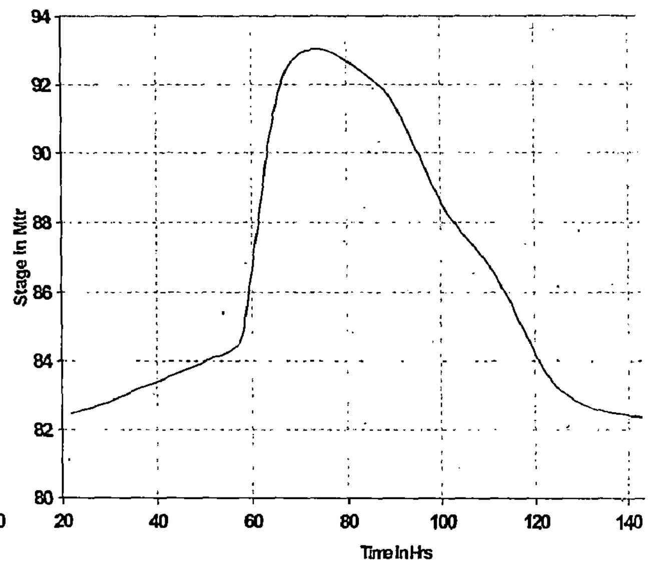
X-section at 145.0 km



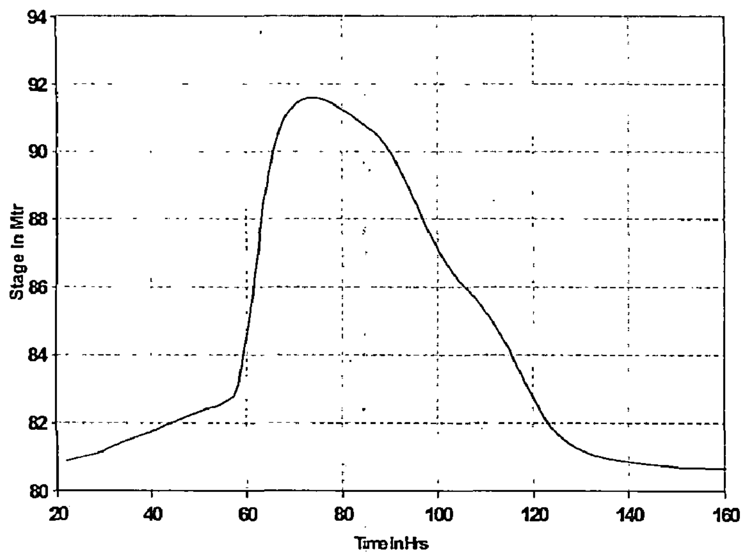
X-section at 155.0 km



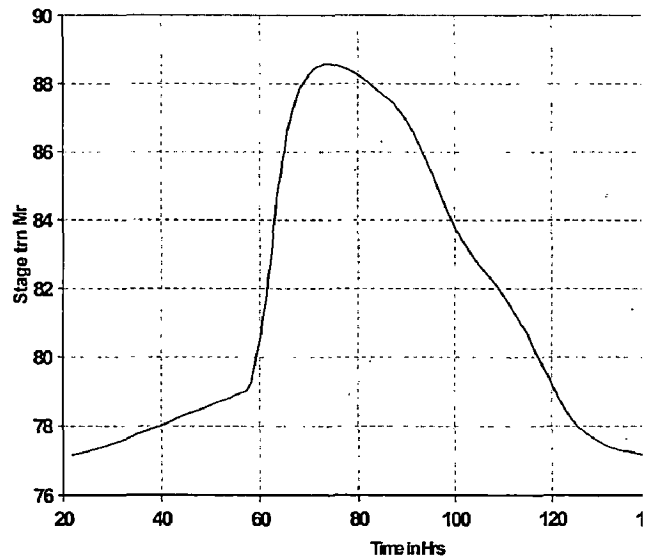
X-section at 163.0 km



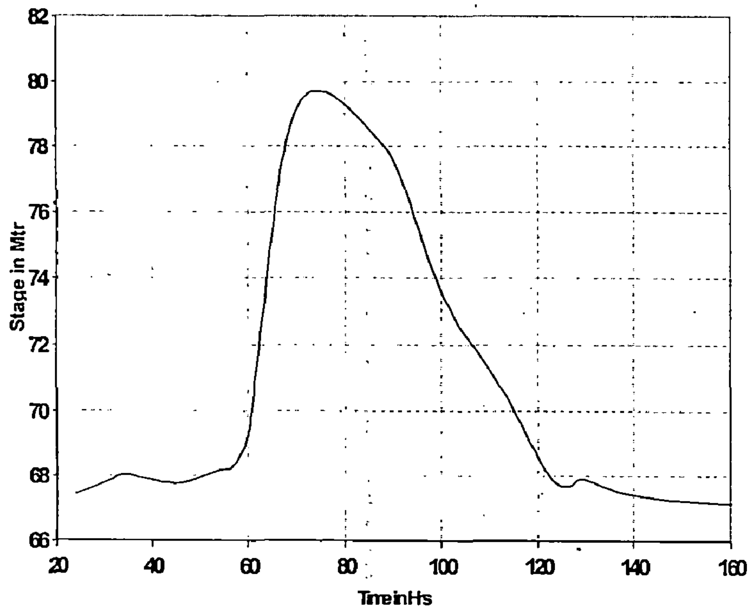
X-section at 170.0 km



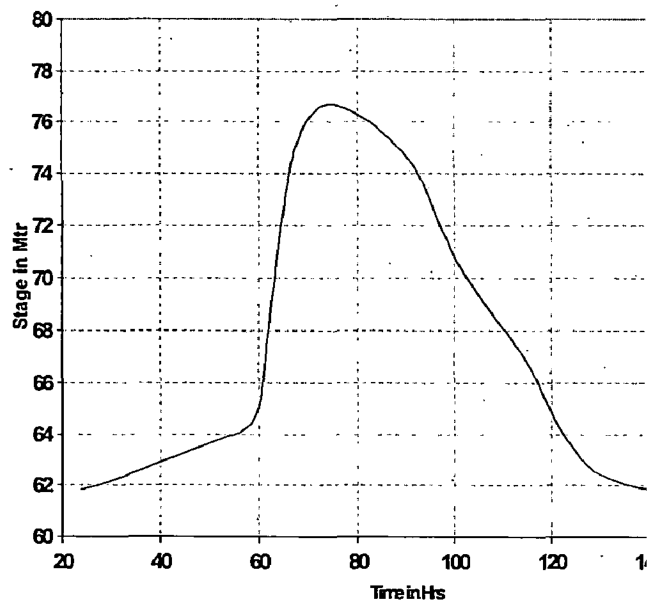
X-section at 180.0km



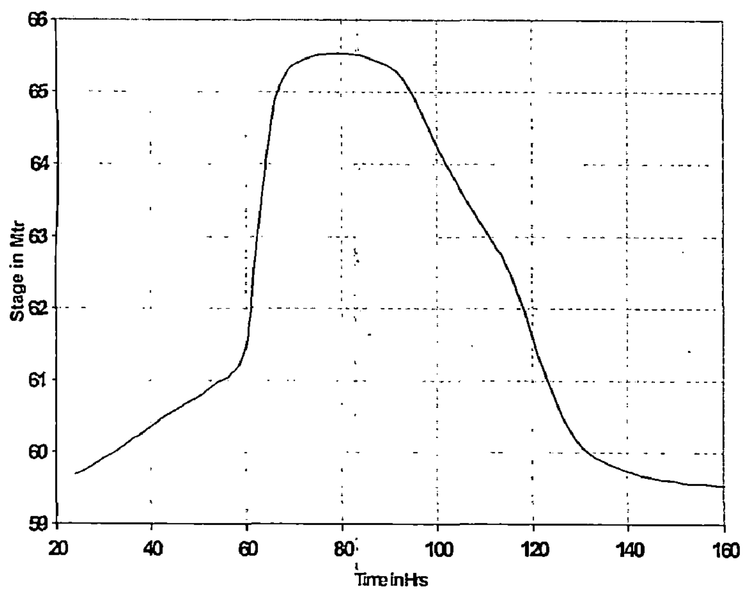
X-section at 190. km



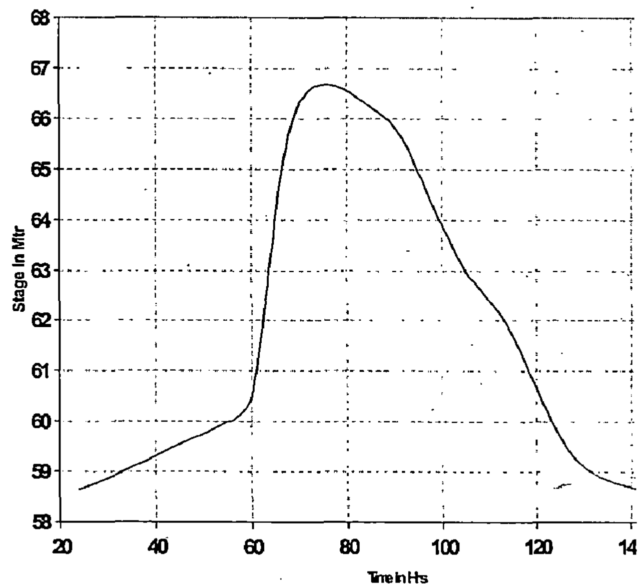
X-section at 200.km



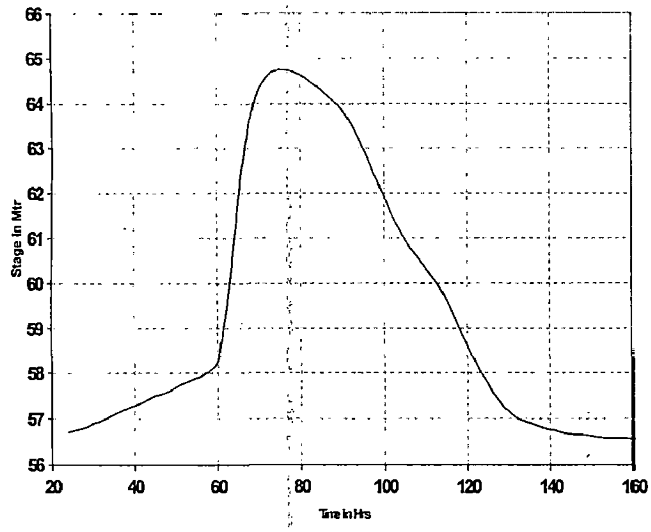
X-section at 214.0 km



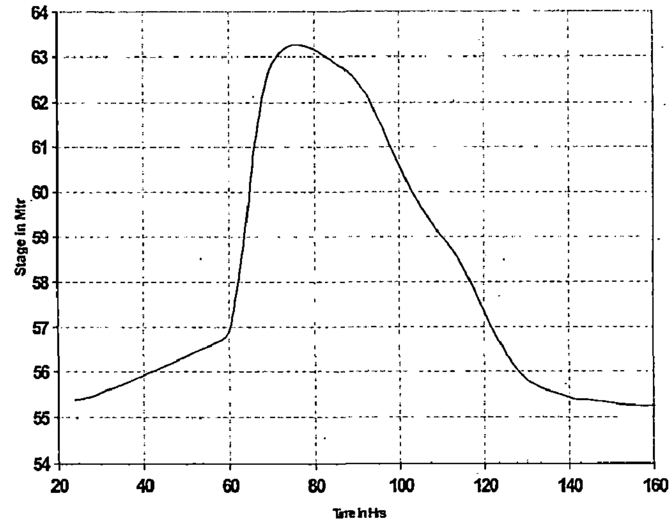
X-section at 220.0 km



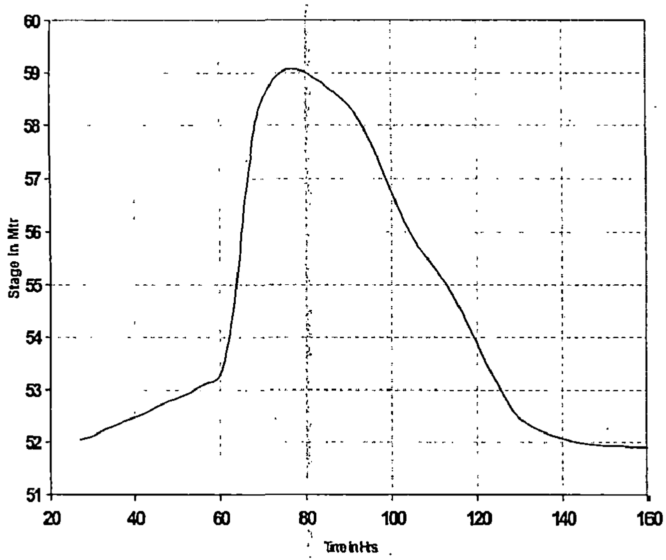
X-section at 225. km



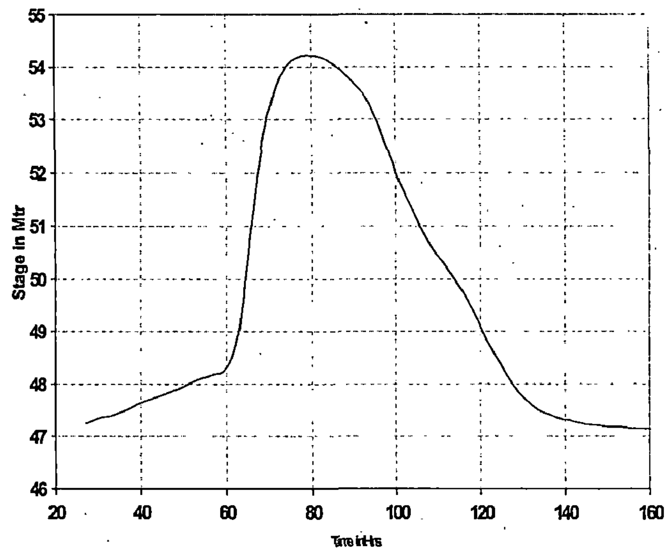
X-section at 226.0 km



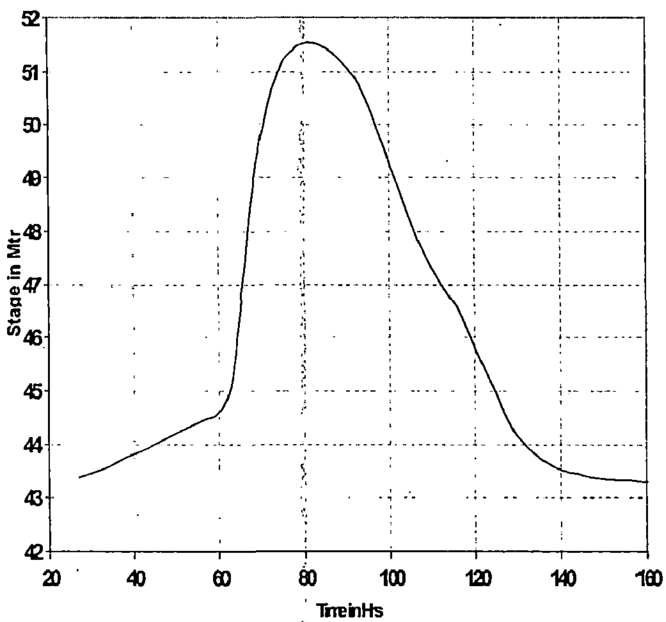
X-section at 238.0 km



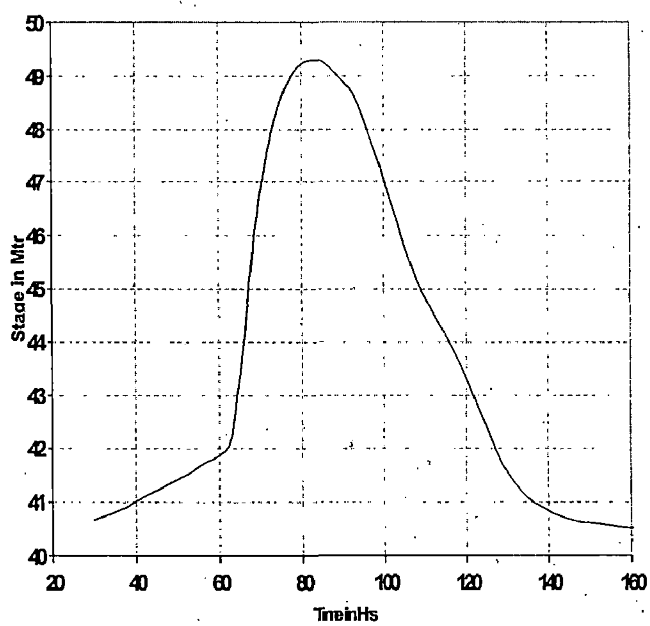
X-section at 250.0 km



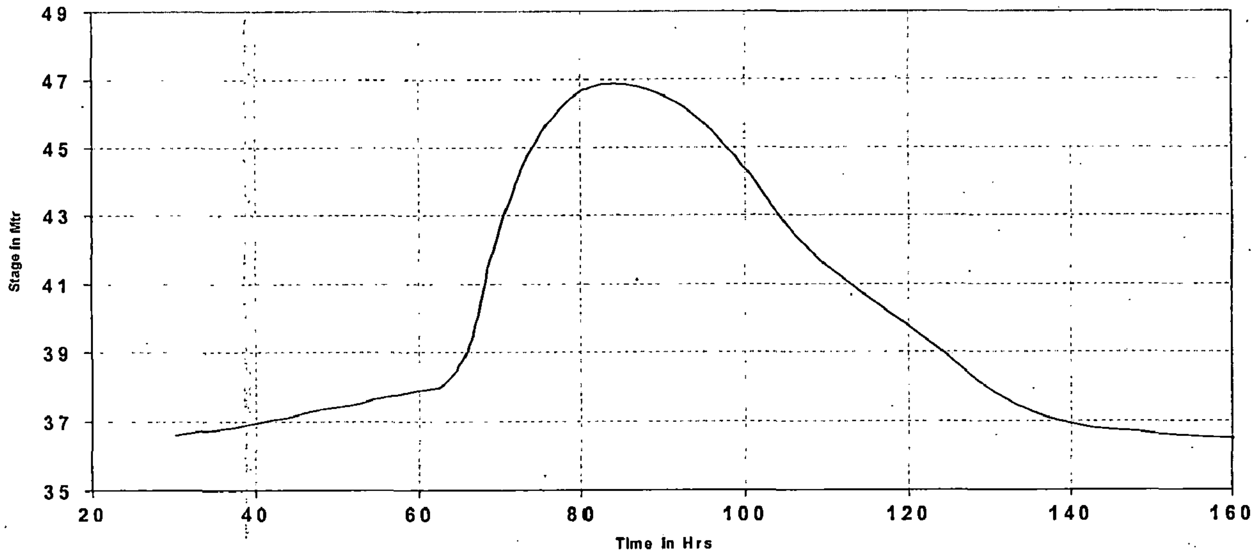
X-section at 260.0 km



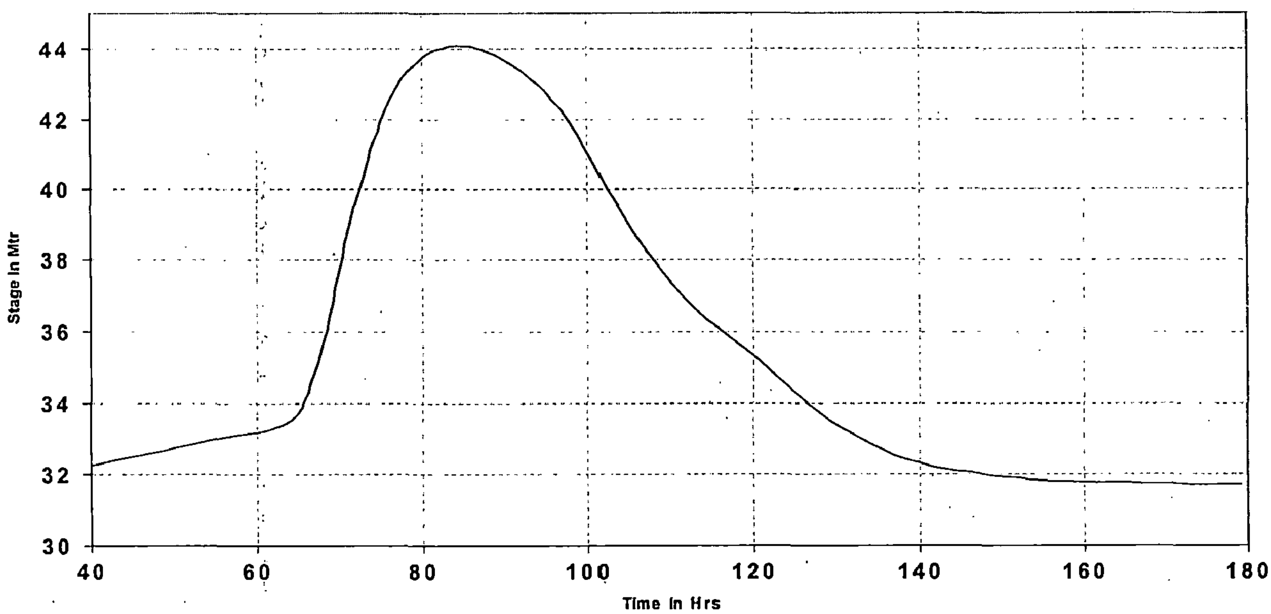
X-section at 269.0 km



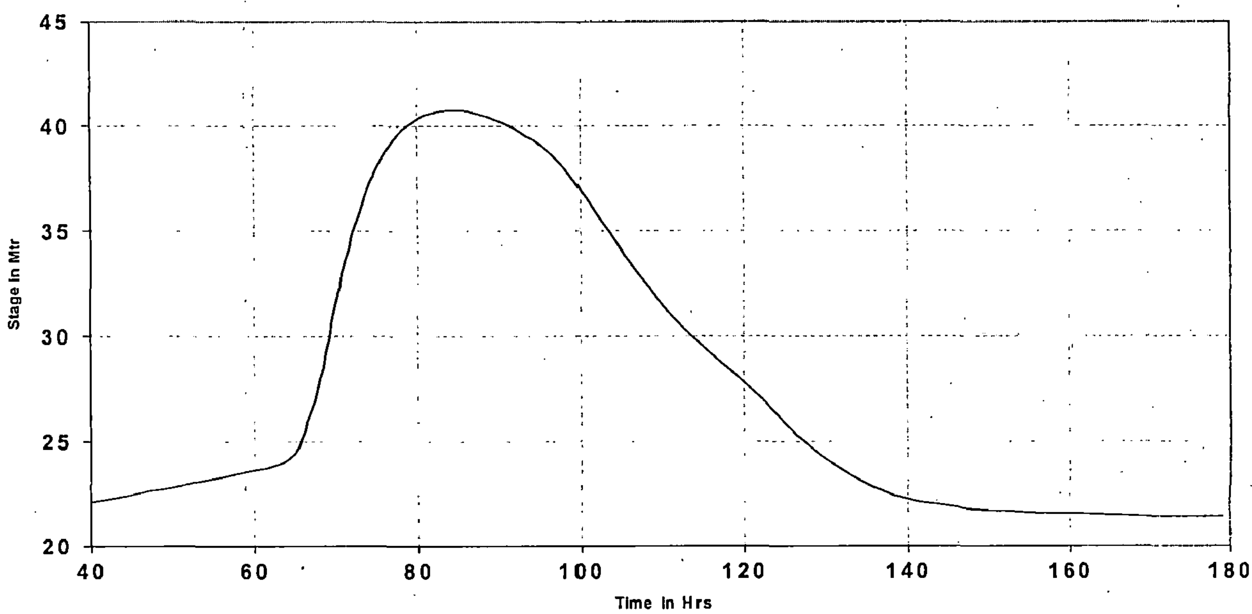
X-section at 284.0 km



X-section at 300.0 km



X-section at 310.0 km



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APPENDIX

ANALYSIS OF THE DOWNSTREAM FLOOD HYDROGRAPH

PRODUCED BY THE DAM BREAK OF

HIRAKUD DAM

ON

MAHANADI RIVER

ANALYSIS BY

DAM BREAK ANALYSIS BY WRDTC
ROORKEE,UTTARANCHAL,247667.

BASED ON PROCEDURE DEVELOPED BY
DANNY L. FREAD, PH.D., SR. RESEARCH HYDROLOGIST

QUALITY CONTROL TESTING AND OTHER SUPPORT

BY

JANICE M. LEWIS, RESEARCH HYDROLOGIST

HYDROLOGIC RESEARCH LABORATORY
W23, OFFICE OF HYDROLOGY
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SILVER SPRING, MARYLAND 20910

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***                         ***
***  SUMMARY OF INPUT DATA  ***
***                         ***
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INPUT CONTROL PARAMETERS FOR HIRAKUD DAM

PARAMETER	VARIABLE	VALUE
NUMBER OF DYNAMIC ROUTING REACHES	KKN	1
TYPE OF RESERVOIR ROUTING	KUI	0
MULTIPLE DAM INDICATOR	MULDAM	0
PRINTING INSTRUCTIONS FOR INPUT SUMMARY	KDMP	3
NO. OF RESERVOIR INFLOW HYDROGRAPH POINTS	ITEH	37
INTERVAL OF CROSS-SECTION INFO PRINTED OUT WHEN JNK=9	NPRT	0
FLOOD-PLAIN MODEL PARAMETER	KFLP	0
METRIC INPUT/OUTPUT OPTION	METRIC	0

HIRAKUD DAM RESERVOIR

TABLE OF ELEVATION VS SURFACE AREA

SURFACE AREA (ACRES) SA(K)	ELEVATION (FT) HSA(K)
360682.0	656.00
179646.0	630.00
161021.0	625.00
129794.0	615.00
102873.0	605.00
6858.0	590.00
10614.0	550.00
.0	500.00

1

HIRAKUD DAM RESERVOIR AND BREACH PARAMETERS

PARAMETER	UNITS	VARIABLE	VALUE
LENGTH OF RESERVOIR	MILE	RLM	51.66

ELEVATION OF WATER SURFACE	FEET	YO	634.00
SIDE SLOPE OF BREACH		Z	.75
ELEVATION OF BOTTOM OF BREACH	FEET	YBMIN	509.00
WIDTH OF BASE OF BREACH	FEET	BB	656.00
TIME TO MAXIMUM BREACH SIZE	HOUR	TFH	4.00
ELEVATION (MSL) OF BOTTOM OF DAM	FEET	DATUM	500.00
VOLUME-SURFACE AREA PARAMETER		VOL	.00
ELEVATION OF WATER WHEN BREACHED	FEET	HF	634.05
ELEVATION OF TOP OF DAM	FEET	HD	642.00
ELEVATION OF UNCONTROLLED SPILLWAY CREST	FEET	HSP	610.00
ELEVATION OF CENTER OF GATE OPENINGS	FEET	HGT	.00
DISCHARGE COEF. FOR UNCONTROLLED SPILLWAY		CS	.00
DISCHARGE COEF. FOR GATE FLOW		CG	.00
DISCHARGE COEF. FOR UNCONTROLLED WEIR FLOW		CDO	38336.00
DISCHARGE THRU TURBINES	CFS	QT	.00

QSPILL(K,1) CFS	HEAD(K,1) FEET
***********	*********
838673.	.0
923759.	15.0
950602.	20.0
*******	28.0
*******	34.0
*******	38.0
*******	40.0
*******	52.0

DHF(INTERVAL BETWEEN INPUT HYDROGRAPH ORDINATES) = 6.00 HRS.
TEH(TIME AT WHICH COMPUTATIONS TERMINATE)= 216.0000 HRS.
BREX(BREACH EXPONENT) = .000
MUD(MUD FLOW OPTION) = 0
IWF(TYPE OF WAVE FRONT TRACKING) = 0
KPRES(WETTED PERIMETER OPTION) = 0
KSL(LANDSLIDE PARAMETER) = 0
DFR(WINDOW FOR CRITICAL FROUDE NO. IN MIX FLOW ALGORITHM)= .050

INFLOW HYDROGRAPH TO HIRAKUD DAM
****

30728.00 87947.00 265677.00 487769.00
882011.001208226.001568031.001923739.00
2312859.002459402.002447605.002296294.002045275.001804746.001565559.001
305003.00
1052995.00 841182.00 659354.00 517261.00 395090.00 301880.00 213792.00
153889.00
116096.00 89713.00 70004.00 54534.00 45316.00 40018.00 36627.00
34366.00
33024.00 32212.00 31647.00 31258.00 31011.00

TIME OF INFLOW HYDROGRAPH ORDINATES

.0000 6.0000 12.0000 18.0000 24.0000 30.0000 36.0000
42.0000
48.0000 54.0000 60.0000 66.0000 72.0000 78.0000 84.0000
90.0000
96.0000 102.0000 108.0000 114.0000 120.0000 126.0000 132.0000
138.0000
144.0000 150.0000 156.0000 162.0000 168.0000 174.0000 180.0000
186.0000
192.0000 198.0000 204.0000 210.0000 216.0000
1

CROSS-SECTIONAL PARAMETERS FOR MAHANADI RIVER
BELOW HIRAKUD DAM

PARAMETER	VARIABLE	VALUE
NUMBER OF CROSS-SECTIONS	NS	39
MAXIMUM NUMBER OF TOP WIDTHS	NCS	8
NUMBER OF CROSS-SECTIONAL HYDROGRAPHS TO PLOT	NTT	6
TYPE OF OUTPUT OTHER THAN HYDROGRAPH PLOTS	JNK	4
CROSS-SECTIONAL SMOOTHING PARAMETER	KSA	0
DOWNSTREAM SUPERCRITICAL OR NOT	KSUPC	0
NO. OF LATERAL INFLOW HYDROGRAPHS	LQ	2
NO. OF POINTS IN GATE CONTROL CURVE	KCG	0

NUMBER OF CROSS-SECTION WHERE HYDROGRAPH DESIRED
(MAX NUMBER OF HYDROGRAPHS = 6)
****

1 5 10 15 20 25

CROSS-SECTIONAL VARIABLES FOR MAHANADI RIVER
BELOW HIRAKUD DAM

```
          PARAMETER                                             UNITS   VARIABLE
         *****************************************  *******  ******

          LOCATION OF CROSS-SECTION                              MILE     XS(I)
          ELEVATION(MSL) OF FLOODING AT CROSS-SECTION            FEET     FSTG(I)
          ELEV CORRESPONDING TO EACH TOP WIDTH                   FEET     HS(K,I)
          TOP WIDTH CORRESPONDING TO EACH ELEV                   FEET     BS(K,I)
               (ACTIVE FLOW PORTION)
          TOP WIDTH CORRESPONDING TO EACH ELEV                   FEET     BSS(K,I)
               (OFF-CHANNEL PORTION)
          NUMBER OF CROSS-SECTION                                         I
          NUMBER OF ELEVATION LEVEL                                       K
1
CROSS-SECTION NUMBER  1
*************************
XS(I) =      .000   FSTG(I) =      .00
HS   ...    494.0    541.2    557.6    574.0    590.4    606.8    623.2
640.0
BS   ...   4380.0  13448.0  14104.0  14432.0  14694.0  14826.0  15301.0
15744.0
BSS  ...       .0       .0       .0       .0       .0       .0       .0
.0

          CROSS-SECTION NUMBER  2
         *************************

XS(I) =     1.863   FSTG(I) =      .00
HS   ...    462.5    472.3    478.9    485.4    492.0    498.6    505.1
511.7
BS   ...    385.0   1902.4   2427.2   2886.4   3017.6   3148.8   3247.2
3247.2
BSS  ...       .0       .0       .0       .0       .0       .0       .0
.0

          CROSS-SECTION NUMBER  3
         *************************

          XS(I) =      4.968   FSTG(I) =      .00

HS   ...    452.6    459.2    475.6    492.0    508.4    518.2    524.8
537.9
BS   ...    103.0    246.0   2460.0   2689.6   3017.6   3214.4   3312.8
3772.0
BSS  ...       .0       .0       .0       .0       .0       .0       .0
.0

         CROSS-SECTION NUMBER  4
         *************************

          XS(I) =      6.831   FSTG(I) =      .00

HS   ...    444.4    452.6    459.2    465.8    475.6    478.4    485.4
492.0
BS   ...    188.0    328.0   2624.0   4428.0   4470.0   4490.0   4920.0
5412.0
```

```
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
1

          CROSS-SECTION NUMBER   5
          ************************

           XS(I) =      9.315   FSTG(I) =      .00

HS   ...     442.8    449.4    452.6    455.9    459.2    462.5    465.8
468.2
BS   ...     453.0    656.0   2853.0   3936.0   3989.0   4018.0   4182.0
4182.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0

           CROSS-SECTION NUMBER   6
          ************************

XS(I) =      12.420    FSTG(I) =      .00
HS   ...     433.0    442.8    446.1    449.4    452.6    455.9    459.2
460.2
BS   ...     938.0   1344.8   1705.0   2132.0   2175.0   2230.0   2296.0
2296.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0

          CROSS-SECTION NUMBER   7
          ************************

XS(I) =      15.525    FSTG(I) =      .00
HS   ...     421.5    429.7    433.0    436.3    439.5    442.8    446.1
447.8
BS   ...    1380.0   3608.0   3772.0   3790.0   3830.0   3910.0   4132.0
4428.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
           CROSS-SECTION NUMBER   8
          ************************

XS(I) =      18.680    FSTG(I) =      .00
HS   ...     390.3    400.2    406.7    413.3    419.8    426.4    433.0
438.9
BS   ...     205.0    328.0   2460.0   3280.0   3476.0   3575.0   3673.0
3739.2
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
1
          CROSS-SECTION NUMBER   9
          ************************

XS(I) =      21.735    FSTG(I) =      .00
HS   ...     390.0    406.7    413.3    419.8    426.4    433.0    439.5
444.9
BS   ...    2234.0   3608.0   3772.0   5904.0   6232.0   6560.0   6724.0
6888.0
```

```
BSS ...        .0        .0        .0        .0        .0        .0        .0
.0

          CROSS-SECTION NUMBER 10
          ************************

XS(I) =    24.840   FSTG(I) =     .00
HS   ...   378.3    393.6    400.2    406.7    413.3    419.8    426.4
433.0
BS   ...  1986.0   3280.0   3608.0   4100.0   4220.0   4262.0   4262.0
4300.0
BSS ...        .0        .0        .0        .0        .0        .0        .0
.0

          CROSS-SECTION NUMBER 11
          ************************

XS(I) =    27.945   FSTG(I) =     .00
HS   ...   378.2    393.6    400.2    406.7    413.3    419.8    426.4
433.0
BS   ...  2032.0   3280.0   3936.0   4001.6   4010.0   4010.0   4015.0
4020.0
BSS ...        .0        .0        .0        .0        .0        .0        .0
.0
          CROSS-SECTION NUMBER 12
          ************************

XS(I) =    37.260   FSTG(I) =     .00
HS   ...   360.8    373.9    380.5    387.0    393.6    400.2    406.7
409.3
BS   ...  1985.0   2460.0   2952.0   3444.0   4264.0   4428.0   4451.0
5215.2
BSS ...        .0        .0        .0        .0        .0        .0        .0
.0
1
          CROSS-SECTION NUMBER 13
          ************************

XS(I) =    47.196   FSTG(I) =     .00
HS   ...   344.4    347.7    354.2    360.8    367.4    373.9    380.5
387.0
BS   ...  1905.0   2296.0   5740.0   5904.0   6396.0   6724.0   8364.0
9676.0
BSS ...        .0        .0        .0        .0        .0        .0        .0
.0

          CROSS-SECTION NUMBER 14
          ************************

XS(I) =    50.150   FSTG(I) =     .00
HS   ...   347.7    354.2    360.8    367.4    373.9    380.5    387.0
389.3
BS   ...  2815.0   3116.0   3936.0   4001.6   4040.0   4264.0   4428.0
4592.0
BSS ...        .0        .0        .0        .0        .0        .0        .0
.0
```

```
          CROSS-SECTION NUMBER 15
          ************************

XS(I) =    52.790   FSTG(I) =     .00
HS  ...    337.8    344.4    351.0    357.5    360.8    364.1    367.4
369.8
BS  ...   3365.0   4100.0   4920.0   5412.0   5838.0   6068.0   6560.0
6724.0
BSS ...       .0       .0       .0       .0       .0       .0       .0
.0
           CROSS-SECTION NUMBER 16
          ************************

XS(I) =    57.550   FSTG(I) =     .00

HS  ...    323.4    328.0    334.6    341.1    347.7    354.2    360.8
364.1

BS  ...    120.0    330.0   2296.0   5084.0   6494.0   6724.0   6888.0
6900.0
BSS ...       .0       .0       .0       .0       .0       .0       .0
.0
1

              OSS-SECTION NUMBER 17
          ************************

XS(I) =    58.955   FSTG(I) =     .00
HS  ...    327.3    334.6    341.1    344.4    347.7    351.0    354.2
356.9
BS  ...   2981.0   4428.0   4985.0   5084.0   5284.0   5510.0   5576.0
5740.0
BSS ...       .0       .0       .0       .0       .0       .0       .0
.0
           CROSS-SECTION NUMBER 18
          ************************

XS(I) =    65.210   FSTG(I) =     .00
HS  ...    308.3    314.9    321.4    324.7    328.0    331.3    334.6
337.8
BS  ...   3985.0   5412.0   5904.0   6166.0   6724.0   7052.0   7282.0
7544.0
BSS ...       .0       .0       .0       .0       .0       .0       .0
.0
           CROSS-SECTION NUMBER 19
          ************************

XS(I) =    70.173   FSTG(I) =     .00
HS  ...    283.7    288.6    295.2    301.8    308.3    314.9    321.4
322.4
BS  ...    692.0    984.0   6150.0   7052.0   7281.0   7380.0   9118.4
9348.0
BSS ...       .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 20
****

```
XS(I) =     77.625   FSTG(I) =      .00
HS   ...    285.4    295.2   298.5    301.8    305.0    308.3    311.6
314.9
BS   ...   2840.0   3280.0  4100.0   4838.0   5904.0   5970.0   6068.0
6560.0
BSS ...        .0       .0      .0       .0       .0       .0       .0
.0
1
```

CROSS-SECTION NUMBER 21
****

```
XS(I) =     85.077   FSTG(I) =      .00
HS   ...    251.7    265.7   272.2    275.5    278.8    282.1    285.4
287.0
BS   ...   2735.0   3854.0  8856.0   9430.0   9840.0 10004.0 10168.0
10168.0
BSS ...        .0       .0      .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 22
****

```
XS(I) =     90.045   FSTG(I) =      .00
HS   ...    245.3    249.3   255.8    262.4    269.0    273.0    277.2
282.0
BS   ...   2872.0   4264.0  7544.0   7806.4   8528.0   8610.0   8700.0
9184.0
BSS ...        .0       .0      .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 23
****

```
XS(I) =     96.255   FSTG(I) =      .00
HS   ...    229.6    236.2   242.7    249.3    255.8    262.4    269.0
274.9
BS   ...   2296.0   3034.0  4264.0   4756.0   5084.0   5166.0   5313.6
5904.0
BSS ...        .0       .0      .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 24
****

```
XS(I) =    101.220   FSTG(I) =      .00
HS   ...    228.3    229.6   236.2    242.7    249.3    255.8    262.4
265.7
BS   ...    310.0    492.0  3526.0   4100.0   5248.0   5510.0   5576.0
6232.0
BSS ...        .0       .0      .0       .0       .0       .0       .0
.0
1
```

```
          CROSS-SECTION NUMBER 25
          *************************

XS(I) =    105.570   FSTG(I) =     .00
HS  ...     228.0    239.4    242.7    246.0    249.3    252.6    255.8
256.8
BS  ...    2953.0   3444.0   4002.0   4920.0   5166.0   6150.0   6280.0
6396.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
          CROSS-SECTION NUMBER 26
          *************************

XS(I) =    111.780   FSTG(I) =     .00
HS  ...     216.5    223.0    229.6    236.2    242.7    246.0    249.3
252.6
BS  ...    1234.0   1640.0   2624.0   3280.0   4264.0   4592.0   4920.0
5248.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0

          CROSS-SECTION NUMBER 27
          *************************

XS(I) =    117.990   FSTG(I) =     .00
HS  ...     180.4    196.8    206.6    213.2    229.6    239.4    246.0
249.3
BS  ...     310.0    492.0   2460.0   2624.0   2788.0   2880.0   2886.0
3017.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0

          CROSS-SECTION NUMBER 28
          *************************

XS(I) =    124.200   FSTG(I) =     .00
HS  ...     164.0    180.4    187.0    196.8    213.2    219.8    229.6
239.4
BS  ...     982.0   1115.0  21853.2   1935.2   1968.0   2032.0   2080.0
2132.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
1

          CROSS-SECTION NUMBER 29
          *************************

XS(I) =    132.890   FSTG(I) =     .00
HS  ...     137.8    147.6    154.2    164.0    180.4    196.8    213.2
221.4
BS  ...     780.0    951.2   1312.0   1541.6   1640.0   1738.4   1968.0
1968.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
```

```
          CROSS-SECTION NUMBER 30
         ************************

XS(I) =    136.620   FSTG(I) =      .00
HS   ...     157.4    164.0    196.8    229.6    262.4    295.2    328.0
332.9
BS   ...    2338.0   2788.0   6724.0   7216.0   7478.4   7609.6   7739.0
7872.0
BSS  ...        .0       .0       .0       .0       .0       .0       .0
.0

         CROSS-SECTION NUMBER 31
         ************************

XS(I) =    139.725   FSTG(I) =      .00
HS   ...     147.6    164.0    196.8    229.6    262.4    295.2    328.0
393.6
BS   ...    2210.0   2952.0   9348.0   9840.0  11152.0  13120.0  13300.0
13448.0
BSS  ...        .0       .0       .0       .0       .0       .0       .0
.0

         CROSS-SECTION NUMBER 32
         ************************

XS(I) =    141.588   FSTG(I) =      .00
HS   ...     147.6    154.2    164.0    196.8    229.6    262.4    295.2
318.2
BS   ...     770.0    984.0   6232.0   7216.0   7478.0   7675.2   8200.0
8200.0
BSS  ...        .0       .0       .0       .0       .0       .0       .0
.0
1
          CROSS-SECTION NUMBER 33
         ************************

XS(I) =    147.798   FSTG(I) =      .00
HS   ...     144.3    147.6    154.2    164.0    180.4    196.8    213.2
228.0
BS   ...    1672.0   1968.0   7708.0   8200.0   8856.0   9020.0   9282.0
9840.0
BSS  ...        .0       .0       .0       .0       .0       .0       .0
.0
          CROSS-SECTION NUMBER 34
         ************************

XS(I) =    155.250   FSTG(I) =      .00
HS   ...     111.5    131.2    137.8    144.3    150.9    157.4    164.0
167.3
BS   ...     988.0   1213.6   1640.0   8528.0  10496.0  11808.0  12136.0
12136.0
BSS  ...        .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 35
****

```
XS(I) =    161.460    FSTG(I) =      .00
HS   ...     112.2    118.1    124.6    127.9    131.2    137.8    144.3
147.6
BS   ...    1846.0   2296.0   5576.0   8856.0   9512.0   9840.0  10332.0
10496.0
BSS ...         .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 36
****

```
XS(I) =    167.050    FSTG(I) =      .00
HS   ...      98.4    111.5    124.6    131.2    137.8    144.3    150.9
155.8
BS   ...    2128.0   2952.0   7872.0   9184.0   9280.0   9630.0   9800.0
9840.0
BSS ...         .0       .0       .0       .0       .0       .0       .0
.0
1
```

CROSS-SECTION NUMBER 37
****

```
XS(I) =    176.364    FSTG(I) =      .00
HS   ...      81.3     85.3     91.8     98.4    105.0    111.5    118.1
124.6
BS   ...    1123.0   1312.0   3608.0   4510.0   8856.0   9184.0  11316.0
11316.0
BSS ...         .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 38
****

```
XS(I) =    186.300    FSTG(I) =      .00
HS   ...      56.4     65.6     75.4     82.0     98.4    106.6    114.8
121.4
BS   ...     712.0    902.0    984.0   1968.0   4756.0   6232.0   6560.0
6724.0
BSS ...         .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 39
****

```
XS(I) =    192.510    FSTG(I) =      .00
HS   ...      11.5     24.6     41.0     57.4     73.8     83.6    490.2
100.0
BS   ...     274.0    328.0    984.0   1886.0   2952.0   3116.0   3280.0
3280.0
BSS ...         .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 30
****

```
XS(I) =    136.620    FSTG(I) =      .00
HS   ...    157.4    164.0    196.8    229.6    262.4    295.2    328.0
332.9
BS   ...   2338.0   2788.0   6724.0   7216.0   7478.4   7609.6   7739.0
7872.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 31
****

```
XS(I) =    139.725    FSTG(I) =      .00
HS   ...    147.6    164.0    196.8    229.6    262.4    295.2    328.0
393.6
BS   ...   2210.0   2952.0   9348.0   9840.0  11152.0  13120.0  13300.0
13448.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 32
****

```
XS(I) =    141.588    FSTG(I) =      .00
HS   ...    147.6    154.2    164.0    196.8    229.6    262.4    295.2
318.2
BS   ...    770.0    984.0   6232.0   7216.0   7478.0   7675.2   8200.0
8200.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
1
```

CROSS-SECTION NUMBER 33
****

```
XS(I) =    147.798    FSTG(I) =      .00
HS   ...    144.3    147.6    154.2    164.0    180.4    196.8    213.2
228.0
BS   ...   1672.0   1968.0   7708.0   8200.0   8856.0   9020.0   9282.0
9840.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 34
****

```
XS(I) =    155.250    FSTG(I) =      .00
HS   ...    111.5    131.2    137.8    144.3    150.9    157.4    164.0
167.3
BS   ...    988.0   1213.6   1640.0   8528.0  10496.0  11808.0  12136.0
12136.0
BSS ...        .0       .0       .0       .0       .0       .0       .0
.0
```

CROSS-SECTION NUMBER 35
****

```
XS(I) =    161.460   FSTG(I) =     .00
HS  ...     112.2   118.1   124.6   127.9   131.2   137.8   144.3
147.6
BS  ...    1846.0  2296.0  5576.0  8856.0  9512.0  9840.0 10332.0
10496.0
BSS ...        .0      .0      .0      .0      .0      .0      .0
.0
```

CROSS-SECTION NUMBER 36
****

```
XS(I) =    167.050   FSTG(I) =     .00
HS  ...      98.4   111.5   124.6   131.2   137.8   144.3   150.9
155.8
BS  ...    2128.0  2952.0  7872.0  9184.0  9280.0  9630.0  9800.0
9840.0
BSS ...        .0      .0      .0      .0      .0      .0      .0
.0
1
```

CROSS-SECTION NUMBER 37
****

```
XS(I) =    176.364   FSTG(I) =     .00
HS  ...      81.3    85.3    91.8    98.4   105.0   111.5   118.1
124.6
BS  ...    1123.0  1312.0  3608.0  4510.0  8856.0  9184.0 11316.0
11316.0
BSS ...        .0      .0      .0      .0      .0      .0      .0
.0
```

CROSS-SECTION NUMBER 38
****

```
XS(I) =    186.300   FSTG(I) =     .00
HS  ...      56.4    65.6    75.4    82.0    98.4   106.6   114.8
121.4
BS  ...     712.0   902.0   984.0  1968.0  4756.0  6232.0  6560.0
6724.0
BSS ...        .0      .0      .0      .0      .0      .0      .0
.0
```

CROSS-SECTION NUMBER 39
****

```
XS(I) =    192.510   FSTG(I) =     .00
HS  ...      11.5    24.6    41.0    57.4    73.8    83.6   490.2
100.0
BS  ...     274.0   328.0   984.0  1886.0  2952.0  3116.0  3280.0
3280.0
BSS ...        .0      .0      .0      .0      .0      .0      .0
.0
```

MANNING N ROUGHNESS COEFFICIENTS FOR THE GIVEN REACHES
(CM(K,I),K=1,NCS) WHERE I = REACH NUMBER
****

REACH 1 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 2 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 3 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 4 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 5 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 6 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 7 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 8 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 9 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 10 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 11 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 12 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 13 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 14 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 15 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 16 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 17 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 18 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 19 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 20 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 21 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 22 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 23 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 24 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 25 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 26 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 27 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 28 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 29 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 30 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 31 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 32 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 33 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 34 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 35 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 36 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 37 ...	.033	.033	.033	.033	.033	.033	.033	.033
REACH 38 ...	.033	.033	.033	.033	.033	.033	.033	.033

1

```
        PARAMETER                                                   UNITS   VARIABLE
VALUE
***************************************    *******  ******  *********

MAX DISCHARGE AT DOWNSTREAM EXTREMITY        CFS        QMAXD        .0
MAX LATERAL OUTFLOW PRODUCING LOSSES         CFS /FEET QLL         .000
INITIAL SIZE OF TIME STEP                    HOUR       DTHM        .0000
DOWNSTREAM BOUNDARY PARAMETER                FEET       YDN       .250000
SLOPE OF CHANNEL DOWNSTREAM OF DAM           FPM        SOM         16.92
THETA WEIGHTING FACTOR                                  THETA         .00
CONVERGENCE CRITERION FOR STAGE              FEET       EPSY
.000000
TIME AT WHICH DAM STARTS TO FAIL             HOUR       TFI           .00
AT REACH=   15  DXM SHOULD BE CHANGED TO  .534  DUE TO EXP/CONTRACT
CRITERIA
AT REACH=   24  DXM SHOULD BE CHANGED TO  .164  DUE TO EXP/CONTRACT
CRITERIA

           COMPUTATIONS WILL USE THE FOLLOWING DXM VALUES

.121   .607  1.500  1.500  1.500  1.500   .785   .371  1.500
1.500  1.500  1.500  1.500  1.500   .602
1.500  1.500  1.500  1.500  1.500  1.500  1.500   .213   .166
1.500  1.500  1.500  1.500   .553  1.500
.616  1.500  1.500  1.500  1.500  1.500  1.500  1.500

        LATERAL INFLOW REACH NUMBER

                    LQX(I)

13   15
 (QL(L, 1),L=1,ITEH)
3073.    8795.   26568.   48777.   88201.  120823.  156803.
192374.
231286.  245940.  244760.  229629.  204528.  180475.  156556.
130500.
105300.   84118.   65935.   51726.   39509.   30188.   21379.
15389.
11610.    8971.    7000.    5453.    4532.    4001.    3663.
3437.
3302.    3221.    3165.    3126.    3101.

 (QL(L, 2),L=1,ITEH)
3073.    8795.   26568.   48777.   88201.  120823.  156803.
192374.
231286.  245940.  244760.  229629.  204528.  180475.  156556.
130500.
105300.   84118.   65935.   51726.   39509.   30188.   21379.
15389.
11610.    8971.    7000.    5453.    4532.    4001.    3663.
3437.
3302.    3221.    3165.    3126.    3101.
```