STUDY OF RIVER CHANNEL MODIFICATIONS USING NUMERICAL TECHNIQUES

A DISSERTATION

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

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in

WATER RESOURCES DEVELOPMENT

By

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December, 2002

CANDIDATE'S DECLARATION

I here by certify that this work which is being presented in the dissertation entitled **"STUDYOF RIVER CHANNEL MODIFICATIONS USING NUMERICAL TECHNIQUES"**in partial fulfillment of the requirement for the award of the degree of **Master in Technology in Water Resources Development** submitted to the faculty of the Water Resource Development Training Center , IIT Roorkee , is an authentic record of my own work carried out during the period from July 2002 to December 2002 under the supervision of Prof. Dr. Nayan Sharma , WRDTC, and Associate Prof Dr. P.K.Garg, Civil Engineering Department , Indian Institute Of Technology , Roorkee.

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

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SYNOPSIS

The river stage is a function of channel geometry and fluvial properties of a stream or a channel. On some occasion, it becomes necessary to study and manage the water level variation in a channel through suitable channel modification techniques such as straightening, deepening, widening or a combination of both. In this dissertation the study has been carried out for knowing the consequences of various channel modification alternatives on stage level of a channel by using numerical model i.e. HEC RAS. The HEC River Analysis System model, better known as HEC RAS, is a product of U.S. Army Corps of Engineer's Hydrologic Engineering Center, which considers both steady and unsteady flow conditions.

For the above purposes the geometric data of the River Kakkad, which is situated in the state of Kerala have been used. The manifestation of the stage level of the river with the discharges and the cross sections were investigated with the help of numerical model (HEC RAS) for various channel modification alternatives. The out come of the study is synthesized for recommending the economical way of river channel modification technique.

Study has shown that various river channel modification alternatives can be conveniently examined with the help of computer based numerical simulation models.

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LIST OF SYMBOLS

h = water depth

u = velocity of water

A = cross sectional area

b = water surface width

x = distance along channel

t = time

g = acceleration due to gravity

S_f = friction slope

S_b = Bed slope

Q = Discharge

β = Bousinesq velocity distribution

K = Conveyance of a section

C = Chezy 's coefficient

R = Hydraulic Radius

N = Manning 's roughness coefficient

θ = time weighing factor

Δx = space interval

Δt = time interval

q = lateral inflow

INTRODUCTION

1.1 CHANNEL MODIFICATION:

As water is conveyed in a channel, energy is converted from one form to another or lost .As this loss of energy result in increased stage, stage may be reduced by minimizing energy loss. This may be accomplished by smoothening the channel boundary, straightening the channel, or minimizing the impact of obstruction to the channel. Although, water surface profiles are mostly influenced by friction forces, changes in energy grade line and the corresponding water surface elevation can also result from significant changes in stream velocity between the cross-section. These velocity changes may be due to the result of natural or artificial expansion or contraction in channel width or depth . In U.S.A., every year 400 million cubic yard of earthwork is being dredged from various rivers through channel modification technique for maintaining their stage level. These type of works involve million of dollars and result in many unintended environmental consequences. Hence, there is an utmost necessity for prior study of variation in reduction in stage for a given discharge due to impact of channel modification by using the numerical techniques.

1.2 NUMERICAL TECHNIQUES:

Numerical simulation is by and large the technique of replacing the governing transport equation with algebraic equations and obtaining a final numerical description of the phenomena in space and / or time domain. Numerical simulation involves the manipulation and solution of numbers, leaving behind an end product, which is also a collection of numbers. The final objective of most engineering investigation, whether they are analytical, experimental or numerical, is to obtain a quantitative description of problem in term of numbers. In hydraulics, the numbers usually used are measurement of depth, surface elevation, flow velocity, discharge, bed, surface and perhaps internal stress, mass exchanges between various components with in the model and between the model and surrounding salinities and temperature. .In this regard , numerical simulation technique provides readily acceptable and often most descriptive form of solution to a variety of transport problem.Numerical simulation of practical problem generally involves the repetitive manipulation of thousands or even millions of numbers a task that are feasible only with the aid of computer.

1.3 NUMERICAL MODEL (HEC-RAS):

Recent development in steady and fully dynamic unsteady flow models by computer have provided engineers with accurate hydraulic modeling methods that result in either tabular or graphical two or 3-D visualization for the purposes of various analysis. The U.S. Army Corps of Engineers, Hydraulic Engineering Center (U.S.A.) is one of the leading software developers of the world for incorporating water resources related time – series data into modelling. Earlier HEC-RAS model was developed to calculate the water surface profile for steady gradually varied flow, but it can now handle a full network of channel and streams, a dendritic system or a single river reach. All the flow regime subcritical, supercritical or mixed flow regime can be simulated through this model. For the steady state model, the basic computational procedure is based on the solution of one-dimensional energy equation.

1.3.1 HEC RAS Unsteady Flow Model:

The U.S. Army Corps of Engineers' H.E.C. has recently revamped their steady state HEC-RAS modelling software .The HEC-RAS version (3.0) can also carry out 1-D unsteady flow simulation. Numerical models for the simulation of unsteady free surface flows are based on the laws of conservation of mass and momentum. The unsteady flow is processed in HEC-RAS using the UNET algorithm developed by Dr. Robert L.Barkau (UNET1997). Like DHI's MIKE 11 model, UNET is a 1-D unsteady flow model that can simulate flow in a complex network of open channel. HEC-RAS now has the ability to import stream system schematic information and cross section data from GIS and CAD program. Mainly the software imports the x, y, z co-ordinates of all the data. This allows the HEC-RAS river system to be plotted, which are geospatially correct. The user can read flow and stage data from HEC-DSS and use for computing the water surface profile.

1.3.2 Channel Modification Using HEC-RAS :

In order to perform a channel modification analysis, firstly the hydraulic model of the existing river reach is developed .The channel modification option in HEC-RAS allows to perform a series of trapezoidal cuts in to the existing geometry. The modified geometry (after cut) is saved as a separate geometric data file for viewing the alteration of stage after the flow analysis. The program also computes and displays area and volume information for the cut and fill quantities for a cost effective decision making process.

1.4 PROBLEM DEFINITION:

In a hydroelectric project, the hydel power may fail to generate the desired out put due to the reduction of net head. This reduction of head may be due to excessive head losses in the water conductor system or it may be due to rise of tail water level during floods. Another significant reason for reduction in net head can be ascribed to higher tail water level in the river than the one considered in the design stage.

1.4.1. Objective of the Study:

The primary objective of the study was to carry out computer -based flow simulation to analyze the manifestation of high tail water level with river discharges using river channel cross- section data with the help of numerical model, and to evaluate river channel modifications to mitigate the high tail water level problem.To attain the above objective, the following steps have been envisaged.

(i) Creation of a model network and geometric data file in the numerical model HEC-RAS format, after collecting all the necessary chainage and cross section data from the map of river Kakkad which has been adopted as a case study.

(ii) Development of HEC-RAS model using boundary condition

(iii) Calibration of the model.

(iv) Verification of the model.

(v) Development and evaluation of few concepts of river channel modifications including estimation of quantities.

1.4.2 The Study Area:

The location of the problem site of the study area is in Kerala. A barrage is constructed across the Kakkad River for power generation. The design discharge passing through the turbine is $100m^3$/sec but during floods when the barrage gates are opened the tail water level goes up and consequently the net available head is significantly reduced resulting in less power generation. The water level near the confluence Point of Kakkad River with TRC is 23.0m when $100m^3$/sec of discharge is released from barrage in addition to $100m^3$/sec through the powerhouse. In normal days, the river water level for a release of $100m^3$/sec discharge from power house has been found to be 19.30m. The minimum water level at the tailrace should be 18.85m.The present study is aimed to find out the most cost- effective alternative to lower down the T.W.L. from 23.0m, so as to create more net head and generate more power through river channel modification technique.

1.5. ORGANIZATION OF THE DISSERTATION:

This dissertation documents the data development and implementation of HEC RAS steady flow model and unsteady flow model .It is divided into 6 chapters. Chapter 2 includes the review of literature related to this topic. Chapter 3 discusses the data used and the technical capabilities of the computer programme used for the model development. Chapter 4 describes the study area development in the model and about the different channel modification alternatives. Chapter 5 presents the findings of the HEC RAS model application. The discussion of the result and conclusion of the project are given in chapter 6.

Informations supplemental to the results of this dissertation are included as Appendices. Appendix A includes the formatting of field data i.e. reach length, sub station chainage length, R.L. Manning's n value in the HEC RAS format. Appendix B describes the geometric data and the bed resistance of the channel modification with different alternatives. (1-6). Appendix C is contains data dictionary describing the data used in this project.

LITERATURE REVIEW

2.1 SURFACE FLOW MODELING BY HEC-RAS MODEL:

The U.S. Army Corps of Engineers' s River Analysis System (HEC -RAS) is a soft ware that allows to perform one dimensional steady and unsteady flow river hydraulic calculations. The HEC- RAS soft ware supersedes the HEC-2 river hydraulic package, which was an one dimensional steady flow water surface profile program. The HEC-RAS software is a significant advancement over HEC-2 in terms of both hydraulic engineering and computer science. The software is a product of the Corps' civil works Hydraulic Engineering Research and development program.

The first version of HEC-RAS (version 1.0) was released in July 1995, and this version i.e. (3.0) in January 2001. This new version (3.0) includes unsteady flow routing as well as split flow optimization for steady flow modeling. HEC-2, one dimensional modeling method used steady state hydraulic modeling to determine stream water surface elevation and flows. The steady state models do not take into account of the hydrodynamic effect that more accurately depict the actual flow events. The development of 2-D and 3-D animation from steady state models also requires numerous runs at different flow inputs.

Incorporating hydraulic model results into a GIS environment has improved flood analysis in recent years. Numerous modeling techniques have been interconnected in an attempt to find an optimum combination of various methods. In an attempt to connect hydraulic results to a spatial interface, Djokic (1994) developed an interface between the Hydrologic Engineer Center ' s HEC- 2 1-D , steady – state hydraulic model and the Arc/ info spatial GIS . The interface, known as ARC/ HEC 2, exports the terrain data from Arc/ info into HEC-2. The ARC/ HEC 2 interface converts HEC 2 water surface elevations into GIS coverages in ARC/INFO .

Evans (1998) developed a data exchange format to transfer physical element description between hydrologic and hydraulic software packages and GIS software. The package studied was HEC- RAS, with the ability to import cross-section locations as XYZ coordinates from terrain models to develop channel and reach geometry. Upon completion of the hydraulic calculations, HEC -RAS exports the data back to a GIS for comparison with the terrain model. In 1998, ESRI translated and improved this method and added some utilities to facilitate its use. The result was an ARC-VIEW GIS extension called AV-RAS.

Tate (1999) further investigated how to improve upon the HEC -RAS model's accuracy by incorporating field surveyed, stream geometry and control structures into a GIS – based terrain model. His research led to the development of Avenue scripts for Arcview GIS that integrate such data. The terrain model used for the study was based on very accurate digital orthography. Andrysiak (2000) applied this approach to a larger study area using a digital elevation model (DEM) with 30-meter accuracy as the terrain model. When studying both cases, one can deduce that terrain model refinement is limited to the accuracy of the data. In addition, accuracy of the geo – referencing of the surveyed cross sections and control structures is imperative in the development of an optimum terrain model.

Azagra- Camino (1999) focussed on a smaller study area using more precise terrain data from the development of a Triangulated Irregular Network (TIN) in ARCVIEW GIS . The TIN was created from aerial photography, which resulted in a highly accurate terrain depiction of the study area. Using the AV-RAS extension, topographic information was extracted from the TIN, and imported as channel and stream geometry for use in the HEC RAS model. The flood visualization results provided highly accurate 2-D and 3 – D flood maps. This method method was limited to one output in time for each run from steady state HEC RAS model, making the process of developing flood animations tedious. The animations created required multiple runs of the HEC RAS model and importing the data into the TIN. Additionally, the cross section data were extracted directly from the terrain model. Since the terrain model was based on aerial photography, the cross section data did not account for existing water surfaces in the stream channel when the photographs were taken. Thus, results from his HEC RAS model may not have been accurate.

2.2 UNSTEADY FLOW MODELS:

More complex method of 1-D hydraulic modelling have become more accepted during recent years, as window based computer technology has emerged as the optimum graphical analysis tool. Dynamic wave routing was first used in the early 1950s but has not been widely accepted as flood analysis method. Computer limitations and the complexity of solving method initially made dynamic wave routing unpopular for practical applications.

In 1871, Saint- Venant derived the continuity and momentum equations for 1-D unsteady flow in an open channel, known as the St. Venant shallow wave equations. These equations assumed uniform cross section and bed slope for a segment of open channel. The assumption made in the analysis of 1-D unsteady flow in open channels were as follows:

(1) The wavelength of the disturbance of the flow is very long relative to the depth of flow. This "shallow water wave assumption " implies that the flow is principally 1-D & basically parallel to the walls and bottom forming the channel. Thus, streamline curvature is small, lateral and vertical accelerations are negligible to the longitudinal acceleration and there fore, the pressure distribution is hydrostatic.

(2) The channel geometry is fixed so that the effect of deposition or scour of sediment is small.

(3) The bed of the channel has a shallow slope so that (a) the tangent and sine of the angle that the bottom makes with the horizontal have nearly the same value as the angle and (b) the cosine of the angle is approximately 1.

(4) The effect of boundary friction forces can be estimated with a relation derived from steady uniform flow. Non-uniform and unsteadiness are assumed to have only a small effect on the frictional forces.

(5) The channel alignment with respect to the effect of directional changes on the conservation of momentum principle may be treated as if it were rectilinear even though the channel is curvilinear. Thus, the water surface in any cross section of the stream is assumed to be horizontal. Super

elevation effects on the water surface in channel bends are not considered in the analysis and are assumed to have a small effect on the result.

(6) The fluxes of momentum and energy along the cross section resulting from non-uniform velocity distribution may be estimated by means of average velocities and flux correction coefficient that are function of location along the stream and water surface elevation, and

(7) The flowing fluid is homogeneous or of constant density.

Fread (1976) further investigated the Saint Venant equations and developed an implicit method of solving the dynamic wave for the modeling of meandering streams. He distinguished left and right flood plains from the flow channel in a stream's cross-section. The method was used to solve for the unknown ***h*** (water surface elevation) and ***Q*** (stream flow) for specified points along the stream over a series of time steps. He made some additional assumptions to simplify the 1-D flow problem.

(1) The momentum exchange between the stream channel and flood plain is negligible, and

(2) The flow is distributed to the stream channel and flood plain according to the conveyance.

All the above assumptions led to an implicit method to solve the St. Venant equation which are simultaneous quasilinear first order partial differential equation of hyperbolic type using a finite difference solution.

For one dimensional unsteady free surface flow, the continuity and momentum equations can be written as follows: -

$$\frac{\partial h}{\partial t} + \frac{1}{b} \cdot \frac{\partial (u \cdot A)}{\partial x} = 0 \tag{1}$$

$$\frac{1}{g} \cdot \frac{\partial u}{\partial t} + \frac{u}{g} \cdot \frac{\partial u}{\partial x} + \frac{1}{g} \cdot \frac{\partial h}{\partial x} + (Sf - Sb) = 0 \tag{2}$$

The first term $\frac{1}{g} \cdot \frac{\partial u}{\partial t}$ represent the slope of the energy grade line due to variation of velocity in time (acceleration).

The second term is the slope which correspond to the variation of velocity head U^2/2.g (in steady flow) in space; for continuous function $\frac{u}{g}.\frac{\partial u}{\partial x}=\frac{\partial}{\partial x}(.\frac{u^2}{2g})$

The third term $\frac{1}{g}.\frac{\partial h}{\partial x}$ is the slope of the water surface itself.

The fourth (Sf-Sb)represent the slope due to the hydraulic resistance for bottom and side friction , surface wind and internal pressure gradient.

For computational point of view, it is generally preferable to use Q instead of u as the dependant variable. Hence the equation can be written as follows:-

$$\frac{\partial h}{\partial t}+\frac{1}{b}.\frac{\partial Q}{\partial x}=0 \qquad (3)$$

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}(\beta.\frac{Q^2}{A})+g.A.\frac{\partial h}{\partial x}+g.A.\frac{Q|Q|}{K^2}=0 \qquad (4)$$

Where β = Bousinesq velocity distribution coefficient for total cross sectional area its value is $\frac{A}{Q^2}\int_A u^2.\partial A$ .Its value range from 1.01 for straight prismatic channel to 1.33 for river valley with flood plains (Chow, 1988).

The conservation of volume (mass) principle relates to flows and changes in the quantity of water stored in the channel. No forces of any kind are considered in the conservation of mass. Forces, momentum fluxes and the momentum of water in storage are related in the conservation of momentum principle.

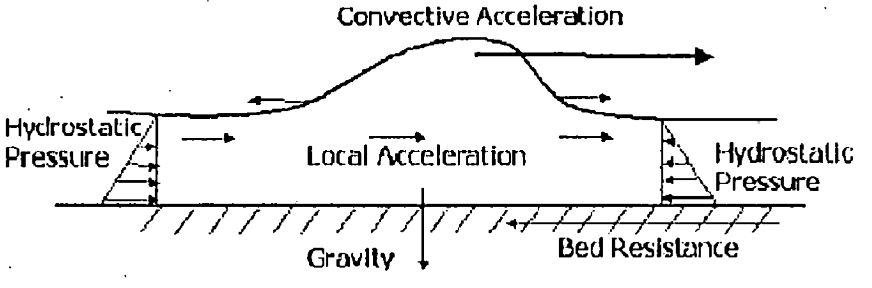


Fig.2.1. Finite element of a stream channel with force terms

To better understand unsteady flow model terms a finite segment of the stream as shown in above figure is considered. There are five acceleration and pressure terms that act on the control volume; convective acceleration, local acceleration, hydrostatic pressure, bed resistance & gravity. The convective acceleration, local acceleration and hydrostatic pressure terms are important to dynamic wave motion because they account for pressure & inertial forces which characterize the movement of a large flood wave in the stream (Chow, 1988).

Using earlier mentioned Saint Venant equation and Fread 's methodology led to the development of the U. S. National Weather Service's (NWS) DWOPER (Dynamic Wave Operational Model)and DAMBRK models . The DWOPER and DAMBRK models use the implicit method for solving the St. Venant equations for unsteady flow. The DAMBRK model was used by the NWS to analyze floods resulting from dam breaks. The NWS models eventually led to the development of the FLDWAV model by Fread (1985). FLDWAV is a dynamic wave model for 1-D unsteady flows for a single stream or a stream network. Like the DWOPER and DAMBRK models, it is based on an implicit finite – difference solutions of the St. Venant equations.

Expanding on Fread 's work , Barkau (1982) defined a new set of equations that were more convenient to solve the computation methods . He combined the convective terms for both the flood plain and channel using a velocity distribution factor. Barkau also replaced the friction slope (bed resistance term) with equivalent force terms. His work is the basis of the Hydrologic Engineering Center's 1 – D, unsteady flow model called UNET. HEC recently improved upon HEC -RAS by including un steady flow using the UNET program as an extension to the software. The unsteady flow options currently exist for HEC RAS version 3.0. Time series water surface elevations results developed from a HEC RAS model can now be imported into Arc view GIS using the HEC GeoRAS extension, a new version of AV RAS.

2.3 THEORETICAL CONCEPTS OF NUMERICAL MODEL:

2.3.1 Numerical Model:

A computer is simply a tool for carrying out arithmetical operation at high speed, so in order to produce a computer model of a flow the problem has to be formulated in suitable form . The starting point is a mathematical model – a set of equation that mathematically describe the behavior of fluid (the equation are normally partial differential or integral equation) .The numerical method substitute an algebraic form of the original equation that can be solved using simple arithmetical procedure to yield the numbers that represent the flow (such as discharge , mean velocity,or the depth of flow) at discrete points and space .

2.3.1.1 Setting up of a Numerical Model:

The basic step involve in setting up of a numerical model of a system are follows:

(i) Defining the nature of the problem

(ii) Reducing the problem to some suitable mathematical form

(iii) Making all possible simplification consistent with adequate modeling. (This implies some clear thinking about the relative importance of each aspect of the problem)

(iv) Replacing the simplified governing equation with finite difference equation).

(v) Defining the boundaries of the domain and condition (of flow) at those boundaries.

2.3.1.2 Governing Equation (or mathematical formulation of physical process):

The Saint –Venant equation equations of conservation of mass and momentum consist of the following: -

$$\frac{\partial h}{\partial t} + \frac{1}{b}\frac{\partial Q}{\partial x} = 0 \qquad (5)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}(\beta\ Q^{\wedge}2/A) + A\frac{\partial h}{\partial x} + g.AQ|Q| \ / K^{\wedge}2 = 0 \qquad (6)$$

2.3.1.3 **Essence of finite difference Scheme:**

(1) The continuum or the continuous solution is discretized and replaced by a grid of points called finite difference mesh .

(2) The continuous derivative of the differential equation are replaced by finite difference on the grid point . Thus a set of algebraic differential equation is obtained in which the variables and coefficients are specified on the grid points called nodes.

(3) The set of algebraic equation is solved using the value of the dependant variable given by the initial and boundary conditions. The solution at any time step provides value of the dependent variables for the next time step of the computational procedure. Rectangular grids are preferred for simplicity.

2.3.1.4 Preissmann ' s four point implicit scheme :

The numerical generally for the simulation of unsteady free surface flow is based on mainly Preissmann four point implicit scheme.

The computational grid is a finite set of points sharing the same domain in the (x, t) plane as the continuous argument function. This set is the domain of definition of the discrete- argument function, which is called grid function.

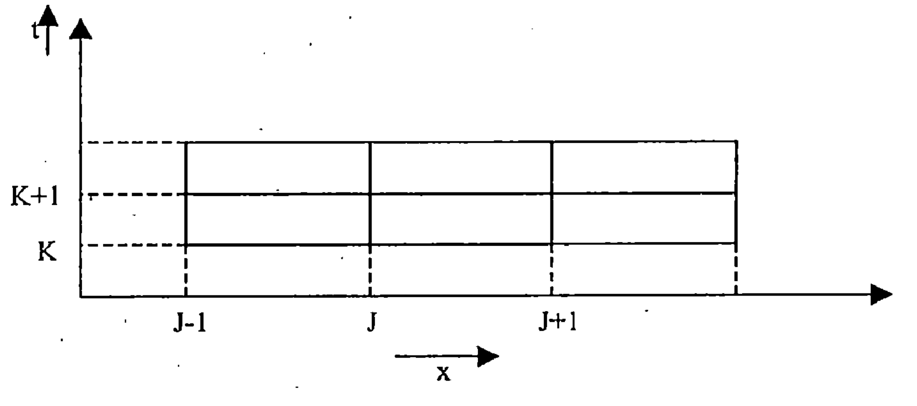


Fig.2.2 Preissman four point implicit scheme

The following relation describes the discretization of the dependant variables.

$$f(x,t) \approx \frac{\theta}{2}(f_{J+1}{}^{K+1}+f_J{}^{K+1})+\frac{1-\theta}{2}(f_{J+1}{}^{K}+f_J{}^{K})$$

$$\frac{\partial f}{\partial x}=\frac{\theta}{\Delta x}(f_{J+1}{}^{K+1}-f_J{}^{K+1})+\frac{1-\theta}{\Delta x}(f_{J+1}{}^{K}-f_J{}^{K})$$

$$\frac{\partial f}{\partial t}=\frac{1}{2.\Delta t}(f_{J+1}{}^{K+1}-f_{J+1}{}^{K}+f_J{}^{K+1}-f_J{}^{K})$$

Where f is the dependant variable, θ is a time weighing factor such that $0<\theta\leq 1$ and k, j represent time and grid number in a river reach respectively.

Accordingly, $\frac{\partial h}{\partial t}=\frac{1}{2\Delta t}(h_{J+1}{}^{K+1}-h_{J+1}{}^{K}+h_J{}^{K+1}-h_J{}^{K})$ (7)

$$\frac{\partial Q}{\partial t}=\frac{1}{2\Delta t}(Q_{J+1}{}^{K+1}-Q_{J+1}{}^{K}+Q_J{}^{K+1}-Q_J{}^{K}) \quad (8)$$

$$\frac{\partial Q}{\partial x}=\frac{\theta}{\Delta .x}(Q_{J+1}{}^{K+1}-Q_J{}^{K+1})+\frac{1-\theta}{\Delta x}(Q_{J+1}{}^{K}-Q_J{}^{K}) \quad (9)$$

$$\frac{\partial}{\partial x}\left(\frac{Q\hat{}2}{A}\right)\approx\frac{\theta}{\Delta .x}[(Q_{J+1}{}^{K+1})\hat{}2\ /A_{J+1}{}^{K+1}-(Q_J{}^{K+1})\hat{}2\ /A_J{}^{K+1}]$$

$$+\frac{1-\theta}{\Delta x}[(Q_{J+1}{}^{K})\hat{}2\ /A_{J+1}{}^{K}-(Q_J{}^{K})\hat{}2\ /A_J{}^{K}] \quad (10)$$

$$\frac{\partial h}{\partial x}\approx\frac{\theta}{\Delta .x}(h_{J+1}{}^{K+1}-h_J{}^{K+1})+\frac{1-\theta}{\Delta x}(h_{J+1}{}^{K}-h_J{}^{K}) \quad (11)$$

The space interval in the above equation is $\Delta x= x_{j+1}-x_j$. The coefficient of above equation (5) and (6), when represented according to the Preissmann formulation $f(x,t)\approx\frac{\theta}{2}(f_{J+1}{}^{K+1}+f_J{}^{K+1})+\frac{1-\theta}{2}(f_{J+1}{}^{K}+f_J{}^{K})$ would yield, together with the derivatives expressed by the above algebraic equation (7) to (11) as follows :-

$$\frac{1}{2\Delta t}(h_{J+1}{}^{K+1}-h_{J+1}{}^{K}+h_J{}^{K+1}-h_J{}^{K})+\frac{2}{\Delta .x}[\theta(Q_{J+1}{}^{K+1}-Q_J{}^{K+1})+(1-\theta)$$
$$(Q_{J+1}{}^{K}-Q_J{}^{K})]/[\theta.(b_j{}^{K+1}+b_{j+1}{}^{K+1})+(1-\theta)(b_j{}^{K}+b_{j+1}{}^{K})]=0$$

$$\frac{1}{2\Delta t}(Q_{J+1}^{K+1} - Q_{J+1}^{K} + Q_J^{K+1} - Q_J^{K}) + \frac{\theta}{\Delta.x}[(\frac{Q^2}{A})_{J+1}^{K+1} - (\frac{Q^2}{A})_J^{K+1}] + \frac{1-\theta}{\Delta.x}[(\frac{Q^2}{A})_{J+1}^{K} - (\frac{Q^2}{A})_J^{K}] + g[\frac{\theta}{2}(A_{J+1}^{K+1} + A_J^{K+1}) + \frac{1-\theta}{2}(A_{J+1}^{K} + A_J^{K})]\{[\frac{\theta}{\Delta.x}(h_{J+1}^{K+1} - h_J^{K+1}) + \frac{1-\theta}{\Delta x}(h_{J+1}^{K} - h_J^{K})] + [\frac{\theta}{2}\{(Q)_{J+1}^{K+1}./(Q)_{J+1}^{K+1}/ + (Q)_J^{K+1}./(Q)_J^{K+1}/\} + \frac{1-\theta}{2}\{(Q)_{J+1}^{K}./(Q)_{J+1}^{K}/ + (Q)_J^{K}./(Q)_J^{K}/\}][\frac{\theta}{2}\{(K_{J+1}^{K+1})^2 + (K_J^{K+1})^2\} + \frac{1-\theta}{2}\{(K_{J+1}^{K})^2 + (K_J^{K})^2\}]^{(-1)}\} = 0 ; 0.5 \le \theta \le 1.0$$

Using the above scheme for a pair of adjacent points (j , j+1) the liberalized system may be written as follows .

$$a_1.\Delta h_{j+1} + b_1.\Delta Q_{j+1} + c_1.\Delta h_j + d_1.\Delta Q_J + g_1 = 0 \quad (12)$$

$$a_2.\Delta h_{j+1} + b_2.\Delta Q_{j+1} + c_2.\Delta h_j + d_2.\Delta Q_J + g_2 = 0 \quad (13)$$

The coefficient of equations 3 and 4 are function of known parameter and can be directly evaluated knowing h_j^k, Q_j^k.

For a given river system equations (12) and (13) can be expressed as follows .

$$[C].[Z] = [D]$$

Where $[C]$ is the coefficient matrix, $[Z]$ represent either water level or discharge $[D]$ is a column matrix.

This system of equations are generally solved by double sweep algorithm for single reach river, dendritic & loop –type river net work, depending on the type of river system. Basing on the above theory, U.S. National Weather Service (NWS) developed the FLDWAV dynamic wave model for analyzing the flood. Later in 1982 Robert Barkau (1982) developed UNET algorithm for the solution of above equation with some modification.

Fig. 2.3: STEPS IN DEVELOPING THE PHYSICALLY BASED MODELS

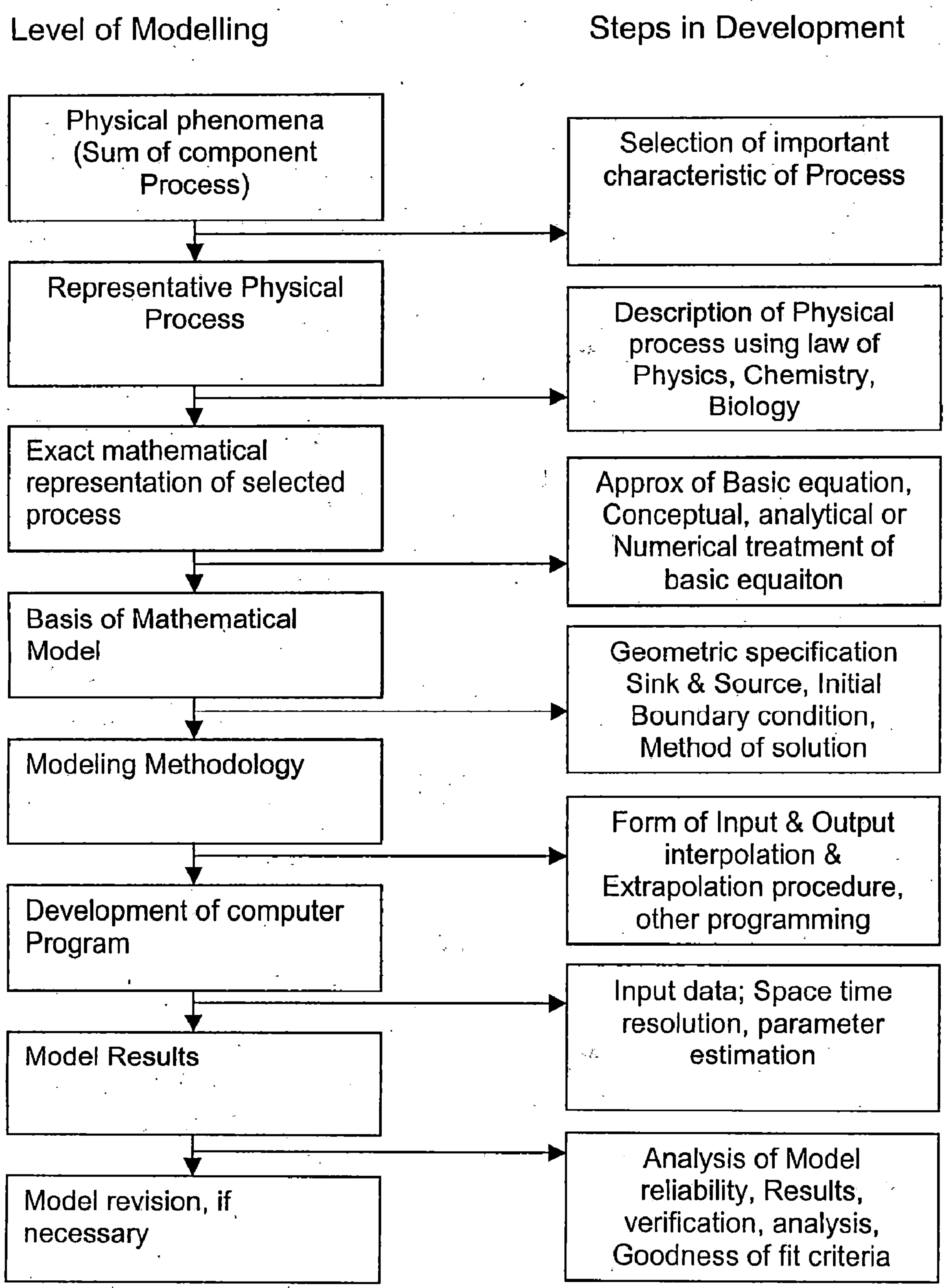


FIG. 2.4: CLASSIFICATION OF NUMERICAL METHODS FOR SOLVING St.VENANT'S EQUATIONS

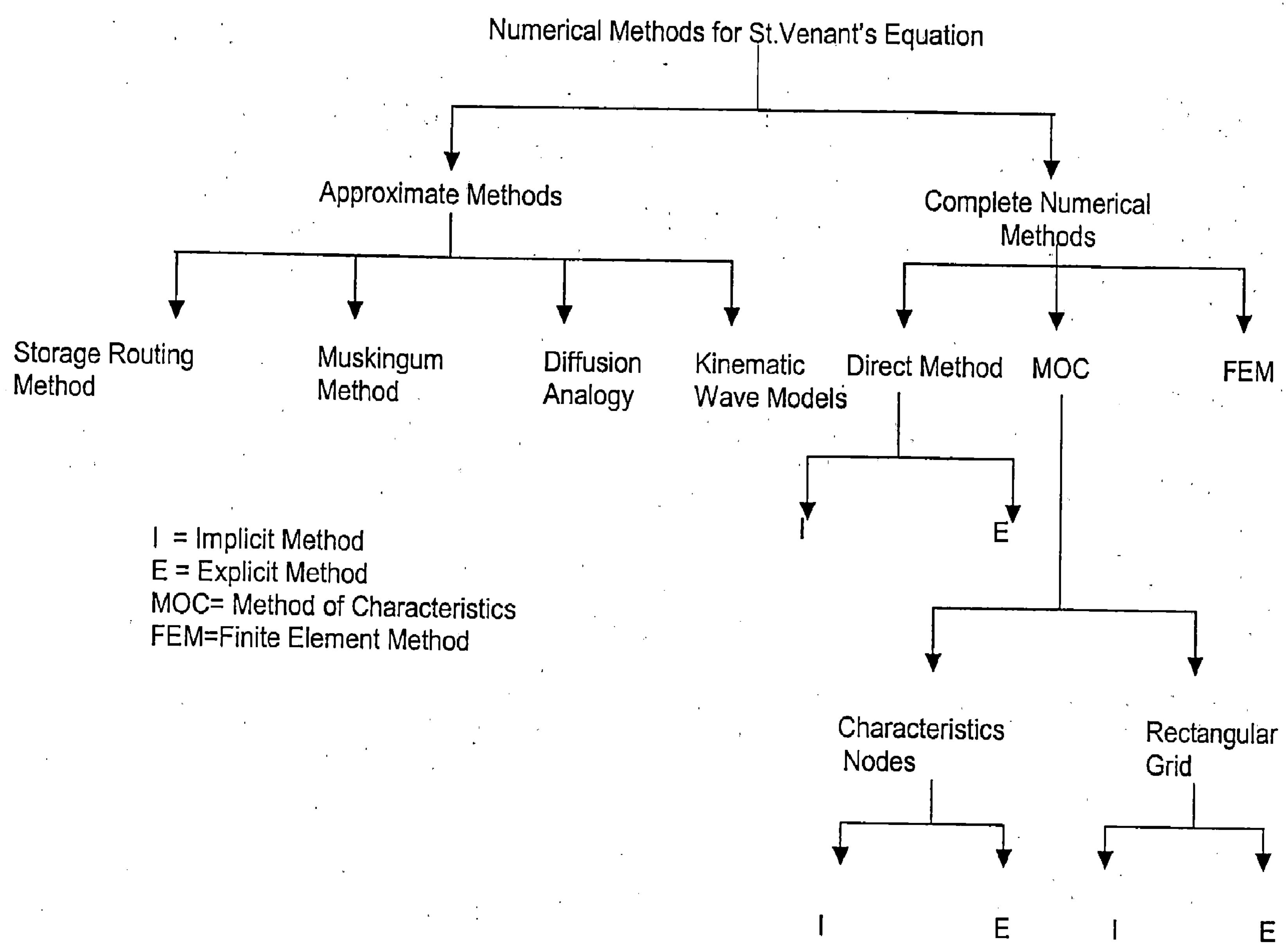


FIG. 2.5: HEC-RAS ANALYSIS SYSTEM

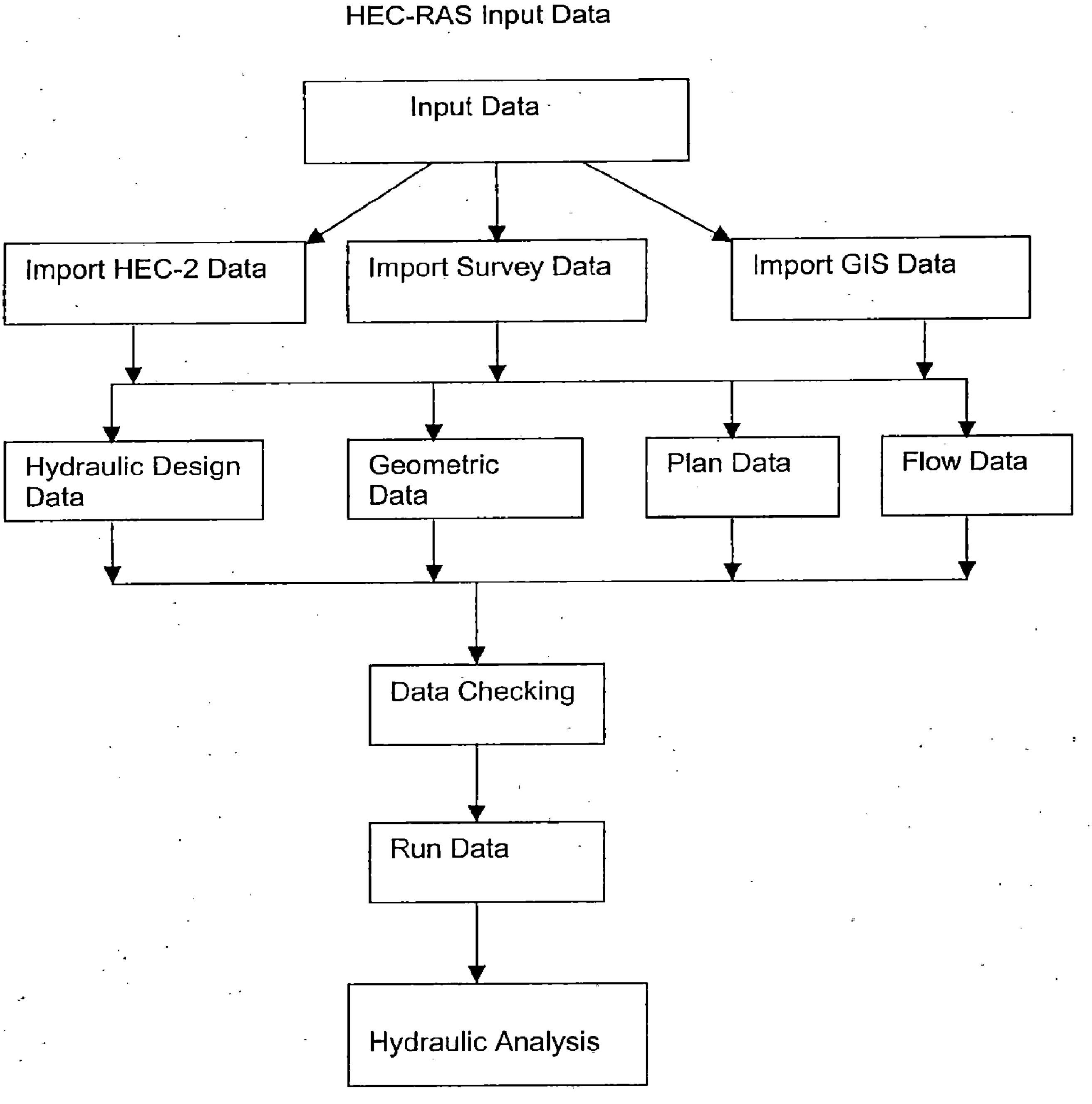
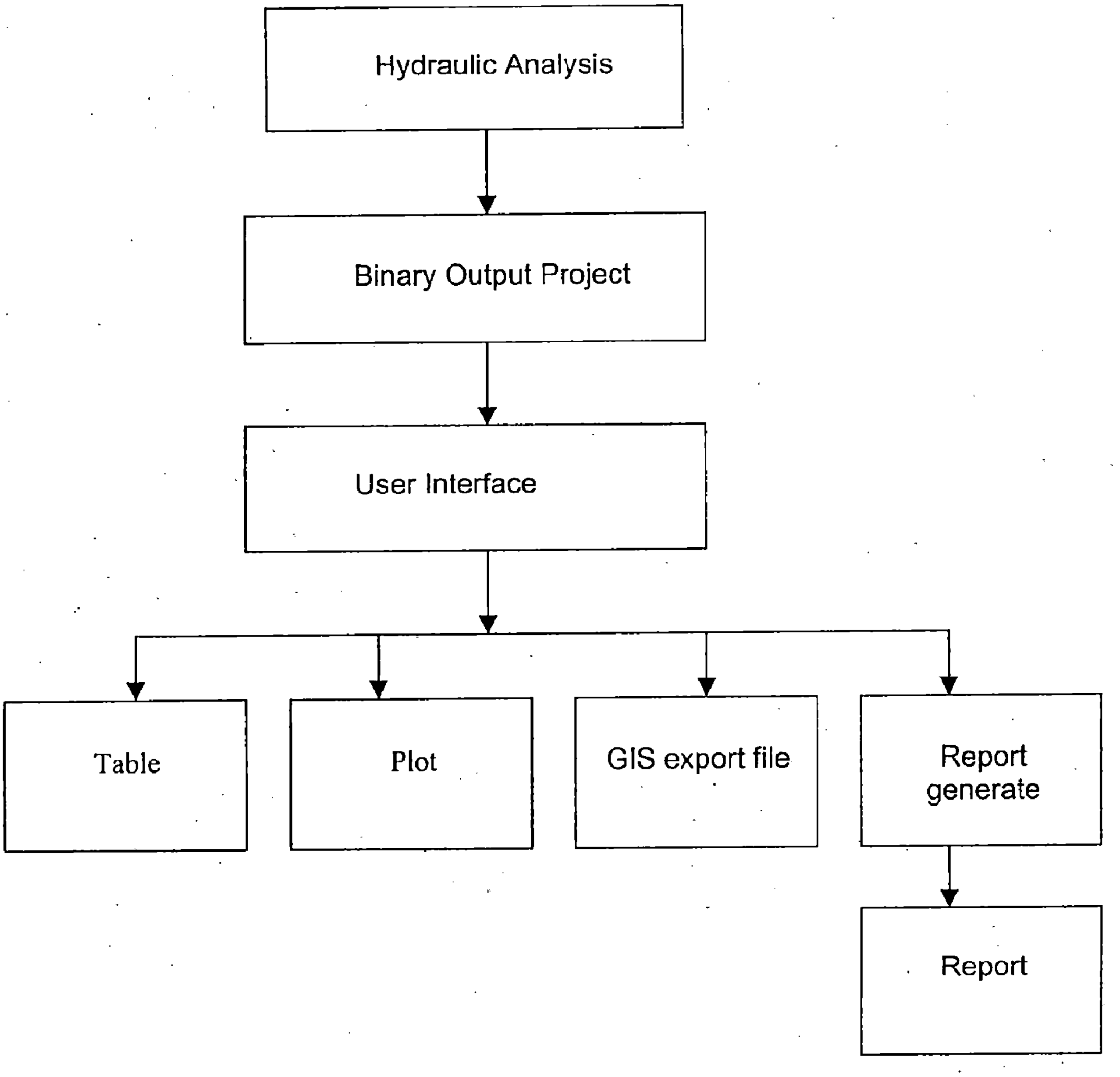


FIG. 2.6: HEC-RAS OUTPUT



2.4 CHANNEL MODIFICATION:

One form of hydro modification is channel modification. These terms (used interchangeably) describe river and stream channel engineering works undertaken for the purpose of-

(i) flood control
(ii) navigation
(iii) drainage improvement
(iv) reduction of channel migration potential (Brookes, 1990).

Activities such as straightening, widening, deepening or relocating existing stream channel and clearing or snagging operation falls in to this category. These form of hydro modification typically result in the following changes -

(i) more uniform channel cross section
(ii) steeper stream gradient
(iii) reduced average pool depth.

Earliest study of channel modification for lowering down the water level of a river was done in the year 1598. During that period Rome suffered a great flood caused by the river Tiber. Since such flood had been happening somewhat frequently, it was considered advisable to increase the capacity of the riverbed. Architect Giovanni Fontana in charge of investigation measured the area of the wetted section of the river (after the flood), which came out to be 500 canes. The amount of flow was equal to 3 times the effective section of the bed. . The unit of flow described at that time was cane, equal to 2m. In this case the term referred evidently to square cane. Fontana inferred that in order to protect Rome from damage it was necessary to open two new channel of size equal to existing one. The idea was not acceptable to Castelli on the ground that the same amount of water had been passed at a bridge (Quattrocapi) side.

Later in the year 1625 after experiment and confirmation with Galileo, Don Benedetto Castelli, who is considered as the father of hydraulics in Italy derived the formula Q = A.V. known as Castelli's law. The formula clearly express that discharge passing through a cross section depends upon the flow velocity.

Later in the year 1769, a French Engineer, Antonie Chezy postulated that the resisting force per unit area of the channel bed is proportional to square of mean velocity. Mathematically he expressed as V = C (R.S_e)^0.5 .

In the year (1891 -1895), Robert Manning found that the value of C is equal to (1/n)R_h^1/6 where n is manning' s roughness factor. Basically the Manning's roughness factor is the summation of -

(i) grain roughness

(ii) form roughness

(iii) vegetation roughness.

Grain roughness is due to the sediment particles; form roughness is due to sinuosity, bed form, vertical variability, expansion and contraction obstruction. Vegetation roughness is due to vegetation in bank, flood plain, and bed.

Roughness alteration affects the depth, temperature and sediment transport in a river. It has been seen that if the value of Manning's n is lowered from 0.035 to 0.026 through hydro modification the flow velocity is increased by 25.7 %. Hence changing or say managing the roughness parameter through channel modification water level of a stream can be altered.

2.4.1 Effect of Deepening:

(i) Up- gradient dredging create temporarily steeper gradient with attendant higher velocities. The resulting higher velocities increases the scouring forces of the stream which will tend to transport the bed material d/s in to the same equilibrium slope that persisted before the dredging began. The readjustment of the bed to equilibrium condition of channel slope considerably gentler than the residual slope resulting from dredging operation. (Allegheny river U.S.).

(ii) If the area is nearer to the sea then there is possibility of seawater mixing with the fresh water for a longer reach.

2.4.2 Effect of widening:

(i) The retain of sediment with in the widening causes temporary d/s erosion.

(ii) Braiding with in the wide river reach increases the variability of flow parameters. (flow depth, flow velocity) and instream habitat condition. On the other hand, the hydraulic load on the bank increases due to cross current.

(iii) (Flow concentration causes intense scouring at the constriction site.

(iv) The slope in the wider reach is steeper than the slope in the narrower one. The bed levels in the u/s reach increase.

2.4.3 Environmental consequence of hydro modification:

Rivers are like conveyor belts and their functions are to transport water & sediment. A river system function as a unified whole; any change in one part of the system affects the other part. It has been seen that the channel modification activities-

(i) have deprived wet lands and estuarine shore lines of enriching sediment.(Hynson et al, 1985, SherWood ,1990)

(ii) have changed the ability of natural system to both absorb hydraulic energy and filter pollutants from surface water (Anderson 1992).

(iii) have caused interruption in the different life stage of aquatic organism (Sher Wood et al ,1990)

(iv) have lowered down the ground water level (Schoof ,1980).

(v) have altered the in stream water temperature and sediment characteristics as well as the rate & path of sediment erosion , transportation and deposition .

(vi) have lowered down the dissolved oxygen level.

(vii) diversion of fresh water by flood and hurricane protection levees has reduced fresh water inputs to adjacent marshes. This has resulted in increased marshes salinity and degradation of marsh ecosystem.

2.4.4. Points to be considered while evaluating Channelization project:

Normally in an economy we depend on the market system to determine how much of a "good," such as channel modification would be produced. How ever, in cases where the production of goods and services produce externalities, i.e. unintended or 'external,' effects such as pollution, then proper evaluation is necessary to bring out the socially optimum amount of production. The following three points may be considered while evaluating the channel modification project-

(i) existing condition

(ii) potential condition, and

(iii) watershed management

(i) Existing condition:

New and existing channel modification projects should be evaluated for potential effect (both problematic & beneficial) based on existing stream and water shed condition. Site specific stream condition, such as flow rate, channel dimension, typical surface water quality, or slope, should be evaluated in conjunction with stream side condition, such as soil & vegetation type, slope or land use. Characteristics of the watershed also need to be evaluated. This phase of evaluation would identify base line condition for potential project and can be compared to historical condition or project already in place.

(ii) Potential condition :

Anticipated changes to the base (or existing) condition in a stream along the stream bank & with in the watershed should be evaluated. By examining potential change caused by new condition, long-term impact can be factored in to the design or management of a channel modification project. Studies like that of Sandheinrich & Achison (1986) clearly show that short-term benefit from hydro modification activities can change to long-term problems.

(iii) Watershed management:

Evaluation of change in watershed condition is paramount in the proper design of a channelization or channel modification project. Since the design of these projects is based on hydrology, changes in watershed hydrology would certainly give impact on the proper functioning of a channelization or channel modification structure. Additionally many surface water quality changes associated with a channel modification project can be attributed to watershed change, such as different land use, agricultural practice or forestry practice.

2.4.5. Channel modification option offered by HEC- RAS:

Once a model of the existing river system is completed, the user can use the channel modification option to perform trapezoidal cuts and fills in to the existing geometry. Channel modification option in HEC RAS allows the user for

(a) multiple trapezoidal cuts (up to three)

(b) independent specification for left and right trapezoidal side slope.

(c) ability to change the Manning 's *n* value for the trapezoidal cuts.

(d) separate bottom width for each trapezoidal cuts.

(e) ability to set new channel reach length.

(f) multiple way of locating the main channel center line.

(g) user can also explicitly define the elevation of the new channel invert , or it can be based on the original channel invert or it can be based on projecting a slope from d/s cross section or u/s cross section.

Once the user has performed all of desired channel modification, then the modified geometry data are saved in to a new geometry file. The user can then create a new plan, which contains the modified geometry & the original flow data that was used under the existing condition. Computation can then be performed for the modified condition and the user can compare the water surface profile for both existing & modified conditions.

MODELLING METHOD

3.1. DATA DESCRIPTION:

This chapter analyzes the data used in this work. The first section discusses the data requirement for the steady flow and unsteady flow models in this work and the last section discusses the sources of data used along with the processing of data.

3.1.1. Type of Data: -

Both steady and unsteady 1-D flow models basically require two types of data.

(1) Qualitative data and

(2). Quantitative data

Quantitative data can be further subdivided into two types:

(i) Topographical or stream geometry data and

(ii) Hydraulic data .

There are two forms of hydraulic data

(a) Stream bed resistance factors

(b) Time series flow and / or stage height boundary condition.

3.1.1.2 Qualitative data:

Qualitative data are nothing but the physical condition that determines flow development pattern. The type of description of river, its tributaries, existence of berm in the flood plain, dykes, breach formation, elevation of roads, localized obstacles within inundated zones, preferential flow axis which are obtained by field investigation belong to this category.

3.1.1.3. Quantitative data:

(i) Topographical or Stream geometry data:

For HEC RAS model, stream bed cross sections, at locations along the network make up a significant portion of the overall geometry data. Cross sections act as upstream

& downstream boundaries for each finite element in the model. Cross sections provide the cross sectional area data required for unsteady flow computation.

The network, or stream centerline of a 1-D flow model can theoretically be modeled as a straight line since natural dispersive effects of flow are not considered. When using the HEC RAS model, the network referencing of the cross section is known as river stations, and runs opposite of chainages (start at a stream's confluence & goes upstream) . Unlike chainage, river stations can use any sequential method to identify cross sections, as long as down stream reach length from each cross section to the next are known. .

The cross sectional shape of a channel governs the pattern of velocity distribution, boundary shear stress, secondary circulation in the channel & in turn its discharge capacity. The geometric parameters are also a function of stage, which varies with discharge.

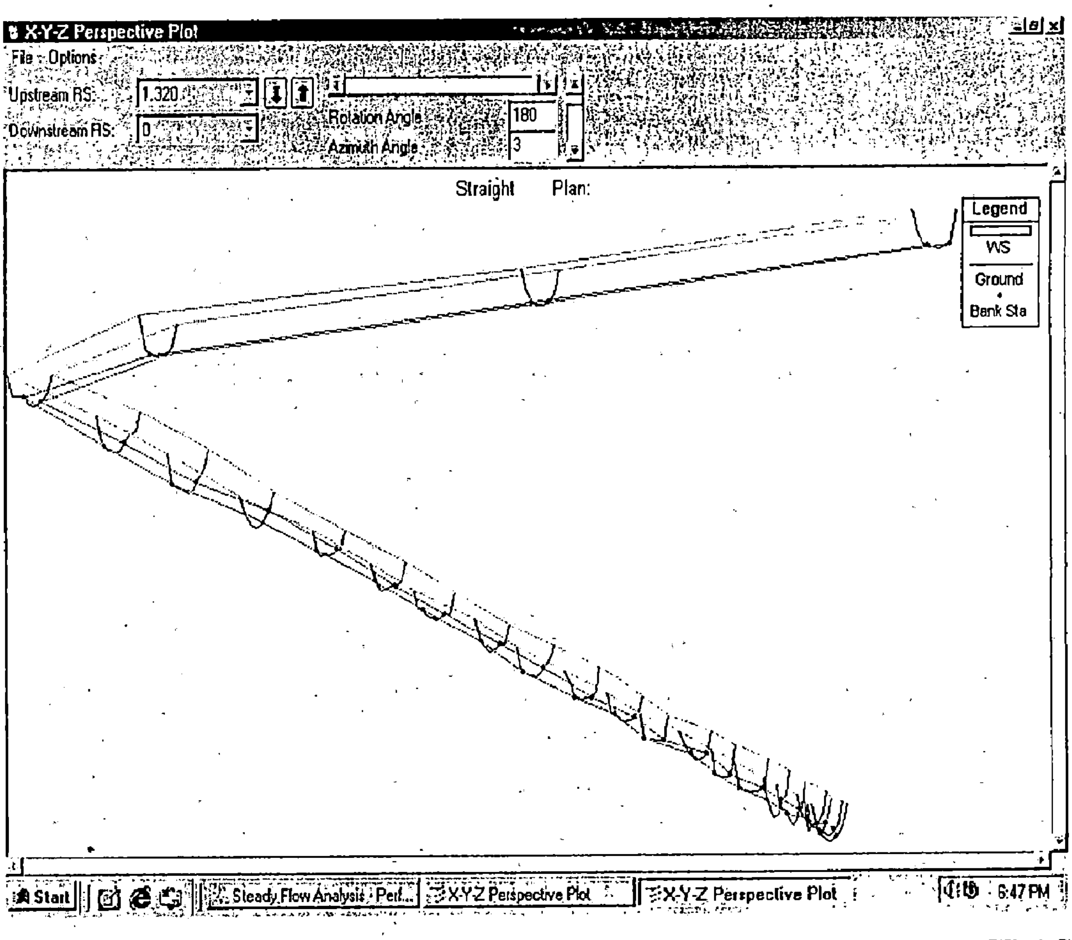


Fig-3.1 : 3-D Depiction of Stream with the Cross – Section in HECRAS

(ii) Hydraulic data:

(a) Bed resistance factors:

Bed resistance factors are also necessary when defining hydrodynamic parameters for unsteady flow models. The resistance factors are used in the bed resistance term in the Saint Venant momentum equation. Bed resistance is an essential factor of steady or unsteady flow calibration. An increase in roughness tends to increase the unsteadiness of flow. In the HEC RAS model, bed resistance is defined at each cross sections as Manning's n values.

(b) Boundary Condition:

The final hydraulic data requirement for unsteady flow model is the boundary condition. Unlike steady state condition, a boundary condition for unsteady flow may be in time series format defined by a user specified time range and time step. Time- series boundary types are discharge (Q) and stage height (h). Another commonly used boundary type not in time series format is a stage – discharge relationship known as a rating curve. In HEC- RAS, upstream boundaries are defined as stage, flow, or stage and flow hydrograph; downstream boundaries are defined as stage , flow, stage and flow , rating curve or normal depth .

Boundary conditions usually depict a flow event for a specified design flow & are obtained from upstream & / or downstream gauge stations data. Initial conditions depict base flow conditions prior to a flow event and are used in HEC RAS model to optimize the model's performances.

3.2. MODELLING METHOD:

3.2.1 General:

The first version of HEC RAS was developed in 1990 and evolved from a steady flow model called HEC-2, which was first developed in 1966. As computer capabilities improved, the HEC-2 was converted to Window – based HEC RAS software to better assist hydraulic modeling with a graphical user interface. In April 2000 , the U.S. Hydrologic Engineering Center also developed the HEC Geo RAS extension of Arc view GIS , a pre and post processing tool for the HEC RAS model . HEC Geo RAS is an upgrade to the previously used AV RAS extension.

3.2.2 The HEC RAS unsteady flow model:

The HEC RAS model (HEC- 2) was initially used for calculating water surface profiles for 1-D steady state flow . The results from the model have been applied to flood management and flood insurance studies through out the United States. The recent HEC RAS 3.0 version provides the modeler with an option to use either the steady flow or unsteady flow option. The unsteady flow option runs the UNET algorithm from the software. Results from UNET algorithm is then imported back into HEC RAS for viewing of simulations.

Along with the unsteady and steady flow options, the HEC RAS model also provides the following capabilities.

* Modeling of open channel networks and single river (for both unsteady and steady flow options).

- Analysis of bridges, weirs, and culverts (for both unsteady and steady flow options) .
- Modeling storage areas, navigation dams, tunnels, levee failure (unsteady flow options only) .
- Handling of sub critical, supercritical, and mixed- flow regimes (steady flow options only) through HEC RAS – 1998 version .

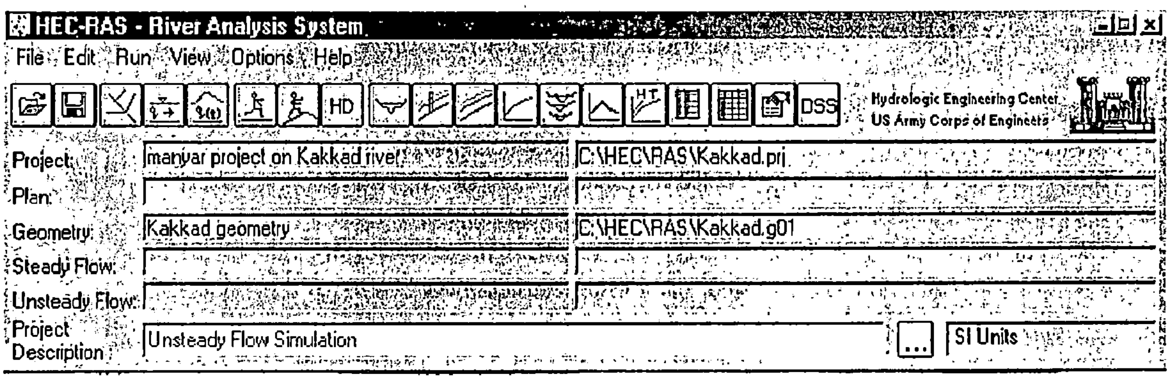


Fig-3.2: Main Menu of HECRAS with the unsteady flow option

The unsteady flow option was used in this work. To develop an unsteady flow model, three files are required. i.e.-

(i) the geometric data file,

(ii) the unsteady flow data file and

(iii) the unsteady flow analysis file .

3.2.2.1. The geometric data file:

The geometric data file contains all the pertinent geometry necessary for hydraulic modeling. It establishes the connectivity of the river network (using river stations for network referencing), cross section data (to include Manning 's n reference factor) , stream junction and hydraulic structure. The file editor allows the importing of geometric data from previous HEC RAS versions, UNET models & from arc view GIs. Editing any of the geometric features can also be accomplished from this file.

HEC RAS version 3.0 also includes a storage area editor, a hydraulic connectivity editor and HTAB parameter editor. The storage area editor provides the modeler the capacity to add and edit storage areas with in the river network system. The hydraulic connectivity editor connects the river network and cross sections with existing hydraulic structures and storage areas. The HTAB parameter editor establishes the initial condition for the unsteady flow options (initial water surface elevation at each cross section in the network) and the incremental unit values (for the change in water surface elevation) used in the UNET algorithm. The incremental unit value is the incremental change of water surface elevations used by the UNET algorithm and is set to a default of 0.1m.

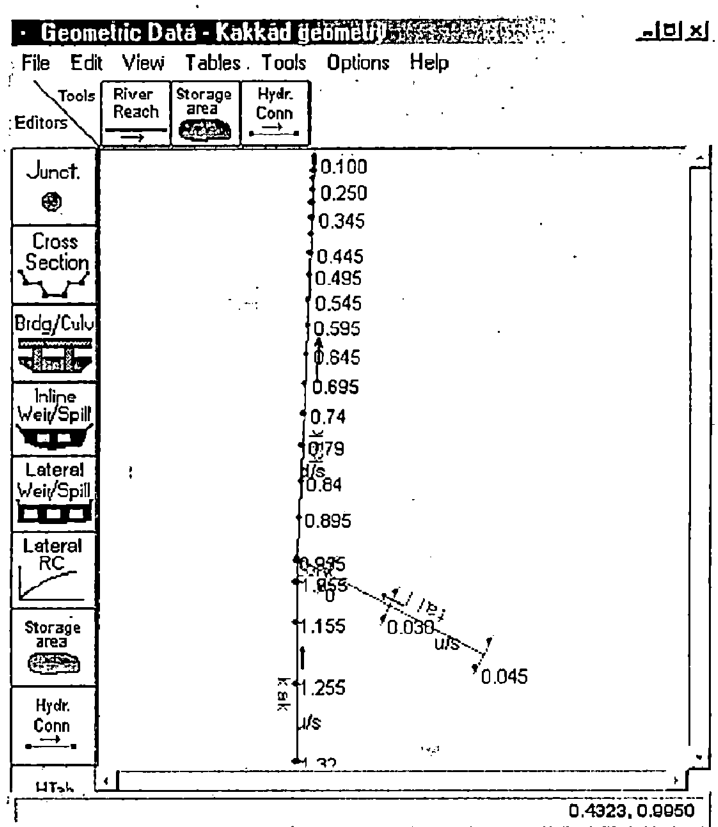


Fig-3.3 Geometric data file editor

3.2.2.2. The unsteady flow data file:

The unsteady flow data file consists of the boundary condition and initial conditions for the model. The initial condition contains the initial flow distribution for each reach with in the river network. The time series boundary conditions contain the upstream and downstream boundary condition (at a minimum) defined as a stage, flow, or stage and flow hydrograph. Internal river network boundary condition options include lateral inflows; uniform lateral inflow and ground water interflow hydrograph .The lateral inflow hydrograph option depicts tributary, point source or watershed outlet (runoff) inflows. The uniform lateral inflows depict overland inflows evenly distributed from one river station location to another along the river network.

Once the boundary and initial conditions are defined in the unsteady flow data file editor, each boundary condition is linked to a user input time series data editor. The time series data can be linked to HEC RAS model result using the DSS (data storage system) interchange or input manually in to the time series data chart . The modeler also defines the time series data 's time interval and reference starting time from the editor.

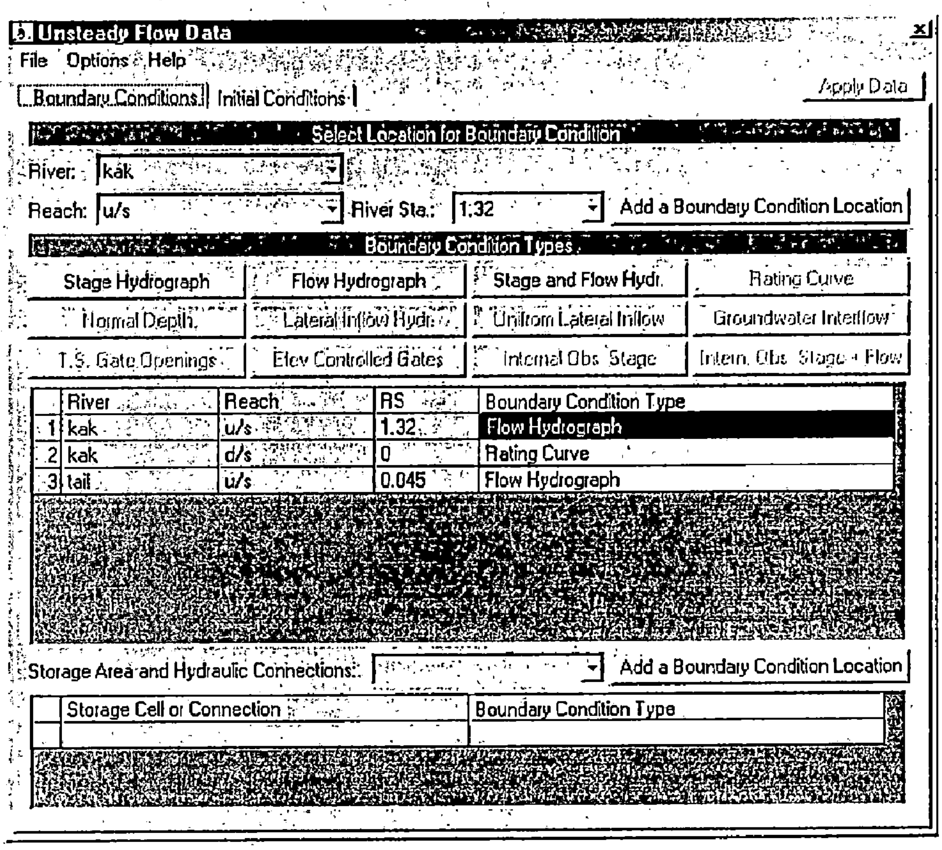


Fig-3.4 HEC RAS unsteady flow data file editor

3.2.2.3 The unsteady flow analysis file:

The unsteady flow analysis file establishes the user-specified condition for the unsteady flow simulation. In unsteady flow analysis, a starting time for the computation when all the flow values are known at the computational nodes must be established. Flow is assumed to be steady everywhere in the system at the starting time. This is the first major difference between steady flow and unsteady flow model analysis. A steady flow analysis must be completed to establish the initial condition for the unsteady flow analysis. The modeler sets the starting and ending time for the simulation and establishes the computational setting for running the UNET algorithm. The computational settings include the computational interval, hydrograph output interval. The instantantaneous profile interval must be equal to or greater than (≥) the computational interval to run the simulation.

Once the unsteady flow model is simulated, the software identifies errors in logic for the geometric, HTAB and unsteady flow data. Once all errors have been resolved, the HEC RAS software runs an HTAB algorithm to establish the initial condition for the entire river network, in preparation for running the UNET algorithm. Once that is accomplished the computer executes the UNET algorithm for the simulations. Results include the water surface profile for each cross section at each time step within the starting and ending time ranges for the entire river network.

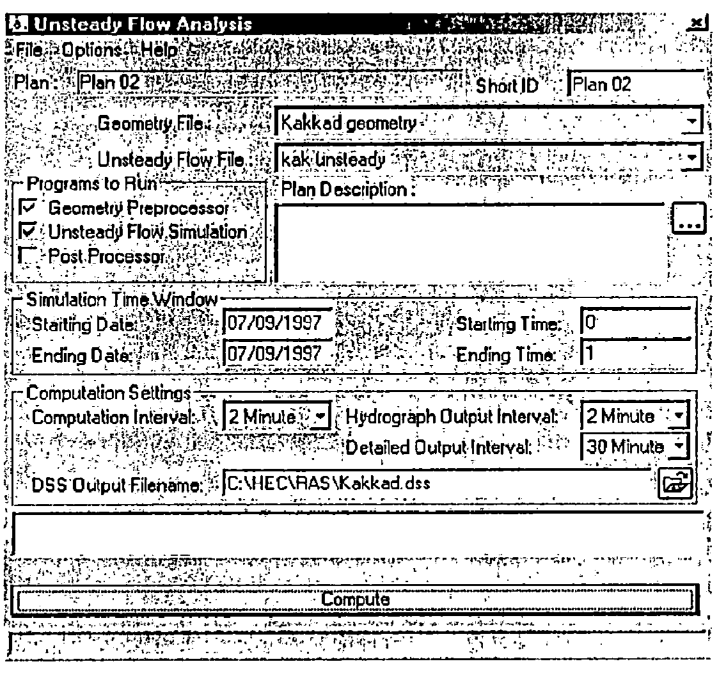


Fig-3.5 HEC RAS unsteady flow analysis file editor

Table 3.1

Similarities & differences between Steady and Unsteady flow analysis

Item	Steady State	Unsteady state
Motion equation	$g.A.\frac{\partial h}{\partial x}+\frac{\partial Q^2/A}{\partial x}=g.A(S_b-S_f)$	$\frac{\partial Q}{\partial t}+gA\frac{\partial h}{\partial x}+\frac{\partial Q^2/A}{\partial x}=g.A(S_b-S_f)$
Mass equation	$\frac{\partial Q}{\partial x}=q$	$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=q$
Exact solution	Not possible	Not possible
Approximate solution	At discrete points	At discrete points
Algebric solution	Between nodes	Between nodes
Channel description	Cross section at nodes	Cross section at nodes
Unknown	Water surface at nodes	Water surface elevation &flow at nodes
Control points	Used to start solution	Isolated in advance
Initial condition	At control points	At all nodes
Boundary condition	None	Required
Special feature	Must be isolated	Must be isolated
Computational Problem	Computational element long	computational element too long
Cross sectional element	Computed as needed	Placed in look up table

CASE STUDY

4.1 THE STUDY AREA:

The location of the study area lies at 9^0 20'N latitude and 76 ^{0} 53' E longitude in the state of Kerala . The project is known as Maniyar small hydro electric project over river Kakkad . The average annual rainfall is 3542 mm, and the catchment of the river is 280 sq. km. There is a barrage over this river; the upstream of which an intake and free flow tunnel has taken off carrying water to turbine through a fore bay. The water is released back to the Kakkad river after passing through draft tube and tail race channel.

4.1.1 Description of the Problem:

The Maniyar small hydroelectric project receives water for power generation as the contribution of the tailrace discharge from Sabarigiri Project which flows in to Kakkad river and the rainfall in the intermediate catchment up to Maniyar barrage. The installed capacity is 12MW and three Kaplan turbine of 4MW, each capacity are installed. The design discharge passing through the turbine is 100 cumec for which the water level at just below the confluence of TRC with Kakkad River is 19.30m. During floods in monsoon, when the barrage gates are raised the tail water level goes up and consequently the net available head is reduced resulting in less power generation. The water level at below the confluence point is 23.0 m when 100 cumec of discharge is released from barrage in addition to 100 cumec of discharge release through the power house . The minimum water level at tailrace channel is 18.85 m. The objective of this study is to find out the most cost effective alternative of river channel modification to lower down the T.W. L. from 23.0m so as to create more net head for more power generation using numerical modeling technique with both steady and unsteady flow approaches.

4.2 MODEL DEVELOPMENT:

Model development involve the following steps:

(i) creation of model network

(ii) preparation of the cross section data as per HEC RAS format

(iii) compilation of flow and water level data for the boundary condition

(iv) estimation of hydrodynamic model parameter

4.2.1. Creation of model network:

For the creation of a dendritic type topographical model network, the topographic data are used from the survey map of the river Kakkad . Topographic data includes the stream's centerline, stream banks, stream cross-sections, and stream channel flow path.

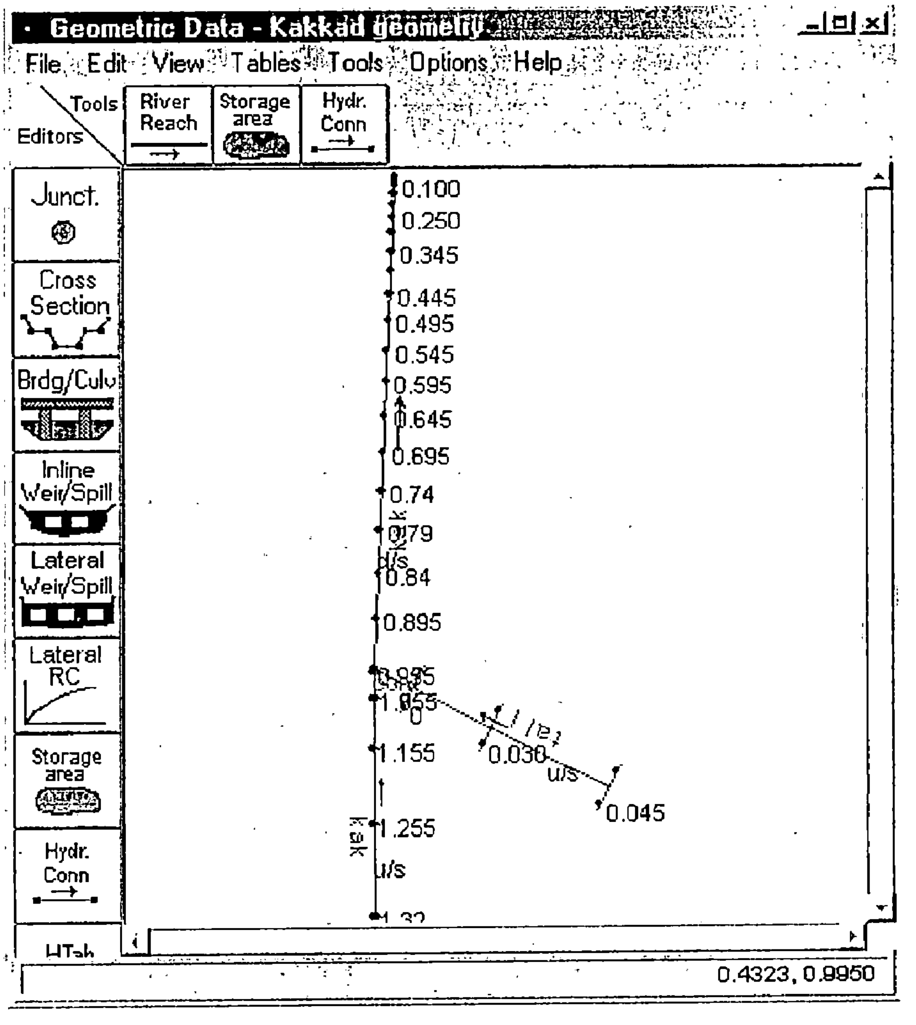


Fig. No.4.1(Discretization of study reach for model operation)

4.2.2. Preparation of cross section data file as per HEC RAS format:

After studying the cross sectional data at all the stations, the cross section data at 25 stations along the main river were selected and formatted for input into the HEC RAS model. Out of these, 5 cross section data represent the station from barrage point to confluence point and the rest 20 cross section data represent the station from confluence point to the last point.. The cross section file required manual editing of the chainage values for each station, bed resistance factor, streambed and stream bank location for each cross section. On some occasion the bank station did not get placed properly, hence those stations required some adjustment to best define the stream channel

4.2.3. Compilation of flow and water level data for the boundary condition:

There are two state of flow in a stream -

i) Steady flow, and

ii) Unsteady flow

(i) Steady flow data:

For steady flow analysis the number of profile opted is 1. That means for a flow 100 cumec from barrage i.e. (stn.no. 1.32) and another flow of 100 cumec from power house i.e. (stn. no. 0.45) making 200 cumec of flow at flow change location i.e. (stn. no. 0.955). As the sub critical flow analysis is going to be performed, normal depth is taken as the d/s boundary condition. The down stream boundary condition is taken in this case as normal condition, which means a bed slope of 0.003.

Steady flow simulation:

Steady flow calculation are required as part of an overall unsteady flow simulation study. This is required to generate water surface profile for rough data checks during calibration stage and to provide a reasonable initial condition from which an unsteady simulation can be started up. Hence steady flow simulation was run with the flow data mentioned earlier.

(ii) Unsteady flow data:

For unsteady flow analysis it is required to enter boundary condition at all of the external boundaries of the system, as well as at internal locations after setting the initial flow at the beginning of the simulation period.

(a) Upstream boundary condition:

A flow hydrograph of user defined time series data at barrage site (at station no. 1.32) is used as the upstream boundary condition. Due to the non-availability of flow hydrograph from field data, the same value of 100-cumec discharge is entered in time series format. Using a fixed start time option the flow values (of 100 cumec) are entered assuming a specified time interval of 2 minutes and a date say 07/09/1997.

(b) Internal boundary condition:

The lateral hydrograph of user defined time series data at power house site (at station 0.045) is used as an internal boundary condition . The lateral hydrograph can be entered same as the hydrograph for the u/s boundary condition assuming a specified time interval of 2 minutes and a date say 07/09/1997.

(c) Downstream boundary condition:

The rating curve option can be used as the downstream boundary condition at the last point i.e. station no. 0. The rating curve is generated from the steady state analysis. The values are mentioned below.

Stage (in m.)	Discharge (in cumec)
15.49	40
15.64	50
16.55	100
17.46	150
18.42	200

The above rating curve is used for the study and it is assumed that the rating curve is sufficient distance downstream of the study area such that any error introduced by the rating curve do not affect the study reach

(d) Establishing initial condition as base flow:

The initial flow condition were input into the unsteady flow data editor using the base flow condition. For continuity purposes, the initial flow for the downstream of the confluence point is 200 cumecs i.e. the sum of two base flows.

(e) Unsteady flow simulation:

In HEC RAS using a 2 minute time step, the range of simulation ran from 0 hour on 07/09/1997 to 1 A M. on 07/09/1997 or after 1 hour time duration. The hydrograph output interval was set to 30 minutes.

Display of result:

The flow analysis results are graphically displayed through number of ways in HEC RAS user interface.

4.3 MODEL CALIBRATION:

4.3.1 General:

Model calibration is the process of adjusting the dimensions of simplified geometric elements and the values of empirical hydraulic coefficient so that flow events simulated on the model will reproduce as faithfully as possible the comparable natural events. For incorporating the recorded values, the parameter to be adjusted is the friction coefficient. In case of river model for incorporating the value of stage and discharge the principal parameter adjusted is the friction coefficient or Manning ' s n.

4.3.2 Process:

In the initial model, the Manning ' n values were selected on the basis of field property and subsequently adjusted during the calibration procedure . Care was taken to ensure that consistent values were used through out the model.

For finding out Manning ' n the following two formula were considered-

(i) Bray 's equation i.e. $n = 0.104 . s^{0.177}$ for $0.0002 < s < 0.01$

(ii) the graph presented by Jarret [$n = 0.39 (s^{0.38}) . R^{-0.16}$] (1984)

It was initially in the range of 0.024 to 0.034 for the main channel, but later it was seen that only maximum flow level (for 200 cumec flow) tally with the 23.0m but not the lower flow level or with 19.30m (for 100 cumec flow) . Hence the longitudinal profile for each segment depicting the invert levels were studied properly. It is revealed that the channel sections from confluence point to the last point gives a mixed (both concave and convex) profile, hence needed higher n values. Taking n values 0.048 for bed & 0.118 for side at suitable location it was seen that both the flows i.e. maximum and minimum tally with the value of recorded stage. Calibration was considered complete when successive changes to parameters, achieved no net improvement to the overall result; that is, when an improvement in one area was offset by an equivalent degeneration in another.

4.3.3 Model Accuracy:

The accuracy and reliability of results from a model simulation depend on the quality and the integrity of the input data used to calibrate and verify the model. The abundance of calibration data provided for this study has ensured that a reliable model is produced despite the fact that the last point (stn no. 0) is not sufficiently away from study.

4.4 MODEL VERIFICATION:

Following calibration, the model was tested to measure its performance under a different set of boundary conditions. For this verification of the model, the flow data, which is marginally higher than the initial boundary condition data, was used. The flow of 120 cumec from barrage site & another 120 cumec of discharge from power house site, both in steady state and unsteady state (incremental increase from 100 cumec) was released . The same rating curve at the last point was taken for study purpose.

For steady state condition the water level at below the confluence point came to 23.46m and under unsteady flow condition the water level came to 23.20m which clearly shows that the water stage height for the steady flow model being significantly higher than that for the unsteady flow model. It is due to the fact that (i) most steady flow models consider the peak discharge as occurring simultaneously everywhere, which may not be peak everywhere at the same time. (ii) the unsteady flow model also considers flow duration as a factor in the flow analysis .

Table 4.1 Data input for verifying the stage level of the river at barrage site:

Serial Number	Date	Simulation time in minute	Flow in cumec
1	06/Sep/97/2400	0.00	100
2	07/Sep/97/0002	0.02	100
3	07/Sep/97/0004	0.04	100
4	07/Sep/97/0006	0.06	100
5	07/Sep/97/0008	0.08	102
6	07/Sep/97/0010	0.10	102
7	07/Sep/97/0012	0.12	102
8	07/Sep/97/0014	0.14	104
9	07/Sep/97/0016	0.16	104
10	07/Sep/97/0018	0.18	104
11	07/Sep/97/0020	0.20	108
12	07/Sep/97/0022	0.22	108
13	07/Sep/97/0024	0.24	108
14	07/Sep/97/0026	0.26	108
15	07/Sep/97/0028	0.28	108
16	07/Sep/97/0030	0.30	108
17	07/Sep/97/0032	0.32	112
18	07/Sep/97/0034	0.34	112
19	07/Sep/97/0036	0.36	112
20	07/Sep/97/0038	0.38	112
21	07/Sep/97/0040	0.40	116
22	07/Sep/97/0042	0.42	116
23	07/Sep/97/0044	0.44	116
24	07/Sep/97/0046	0.46	116
25	07/Sep/97/0048	0.48	116
26	07/Sep/97/0050	0.50	120
27	07/Sep/97/0052	0.52	120
28	07/Sep/97/0054	0.54	120
29	07/Sep/97/0056	0.56	120
30	07/Sep/97/0058	0.58	120
31	07/Sep/97/01:00	01:00	120

Table 4.2 Data input for verifying the stage level of the river at Power house site

Serial Number	Date	Simulation time in minute	Flow in cumec
1	06/Sep/97/2400	0.00	100
2	07/Sep/97/0002	0.02	100
3	07/Sep/97/0004	0.04	100
4	07/Sep/97/0006	0.06	103
5	07/Sep/97/0008	0.08	103
6	07/Sep/97/0010	0.10	103
7	07/Sep/97/0012	0.12	105
8	07/Sep/97/0014	0.14	105
9	07/Sep/97/0016	0.16	105
10	07/Sep/97/0018	0.18	108
11	07/Sep/97/0020	0.20	108
12	07/Sep/97/0022	0.22	108
13	07/Sep/97/0024	0.24	108
14	07/Sep/97/0026	0.26	108
15	07/Sep/97/0028	0.28	111
16	07/Sep/97/0030	0.30	111
17	07/Sep/97/0032	0.32	111
18	07/Sep/97/0034	0.34	111
19	07/Sep/97/0036	0.36	115
20	07/Sep/97/0038	0.38	115
21	07/Sep/97/0040	0.40	115
22	07/Sep/97/0042	0.42	115
23	07/Sep/97/0044	0.44	118
24	07/Sep/97/0046	0.46	118
25	07/Sep/97/0048	0.48	118
26	07/Sep/97/0050	0.50	118
27	07/Sep/97/0052	0.52	120
28	07/Sep/97/0054	0.54	120
29	07/Sep/97/0056	0.56	120
30	07/Sep/97/0058	0.58	120
31	07/Sep/97/01:00	01:00	120

4.5 RIVER CHANNEL MODIFICATION ALTERNATIVES:

4.5.1 General:

To begin to address the problem of rising of water level during monsoon period, 6 nos. of possible channel modification alternatives were developed and simulated with the HEC RAS model in order to test their effectiveness from a technical perspective and to provide a basis for estimating the quantiy of dredging of each possibility. Environmental impacts of these modification alternatives were not considered in this part of the study. The following 6 nos. of channel modification alternatives were developed to lower down the stage level of river during monsoon periods.

4.5.1.1 Alternative 1:

(a) Fixing the bed level of station no. 0.79 at level 16.65 (lowering the bed by 0.35m) & cutting the bed at a slope 0.00025 along the length of the river up to station no. 0.695 for a width of 12m,

(b) Again fixing the bed level of station no 0.545 at level 16.30m (lowering the bed by 0.57m) & dredging the bed at a slope 0.00025 along the length of river up to station no. 0.305 for a width of 12m.

4.5.1.2 Alternative 2:

(a) Widening the channel at a width of 16m from station no. 0.79 to 0.595

(b) Again widening the channel at a width of same 16m from station no. 0.495 to 0.395.

4.5.1.3 Alternative 3 :

(a) Deepening the bed level to 16.30m from station no. 0.79 to 0.695 (average deepening = 0.50m) for a width of 20m.

(b) Deepening the bed level to 16.30m from station no. 0.545 to 0.445(average deepening = 0.60m) for a width of 20m .

(c) Deepening the bed level to 16.30m from station no. 0.395 to 0.305 (average deepening = 0.54m) for a width of 20m .

4.5.1.4 Alternative 4:

(a) Deepening the bed level to 16.40m from station no. 0.79to 0.695 (average deepening =0.42m) for a width of 14m .

(b) Deepening the bed level to 16.40m from station no. 0.545 to 0.445 (average deepening = 0.50m) for a width of 14m.

(c) Deepening the bed level to 16.40m from station no. 0.445 to 0.345 (average deepening = 0.44m) for a width of 14m .

(d) Deepening the bed level to 16.40m from station no. 0.345 to 0.305 (average deepening = 0.46m) for a width of 14m.

4.5.1.5 Alternative 5:

(a) Deepening the bed level to 16.40m from station no. 0.79 to 0.695 (average deepening = 0.42m) for a width of 8m. .

(b) Deepening the bed level to 16.40m from station 0.545 to 0.445 (average deepening = 0.50m) for a width of 8m.

(c) Deepening the bed level to 16.40m from station 0.445 to 0.345 (average deepening = 0.59m) for a width of 8m .

(d) Deepening the bed level to 16.40m from station no. 0.345 to 0.305 (average deepening = 0.46m) for a width of 8m .

4.5.1.6 Alternative 6:

(a) Deepening the bed level to 16.40m from station no. 0.79 to 0.695 (average deepening = 0.50m) for a width of 10m .

(b) Deepening the bed level to 16.40m from station no. 0.545 to 0.445 (average deepening = 0.50m) for a width of 10m .

(c) Deepening the bed level to 16.40m from station 0.445 to 0.345 (average deepening = 0.59m) for a width of 10m .

(d) Deepening the bed level to 16.40m from station 0.345 to 0.305 (average deepening = 0.46m) for a width of 10m .

PRESENTATION OF FINDINGS

5.1. FINDINGS FROM MODEL CALIBRATION:

HEC RAS was run to simulate the water surface level at below the confluence point . Simulation was primarily done for water surface level of 23.0m and for a discharge of 200 cumec. The manning 's n roughness values ranged from 0.034 to 0.048 for bed & 0.065 to 0.118 for side of embankment. This is shown in Appendix-A. The simulated water surface came out to be 22.98m as shown in figures 5.1 and 5.3.

5.2. FINDING FROM CHANNELMODIFICATION:

HEC RAS was run with 6 different channel modification alternatives along the different reach of the river. The alternative-wise findings are mentioned below:

5.2.1 Alternative no. 1 i.e. (a) fixing the bed level of station no. 0.79at16.65m (lowering by 0.35m) and dredging the bed up to station no. 0.695 at a slope 0.00025 & (b) fixing the bed level of station no. 0.0.545 at 16.30m (lowering by 0.57m) and dredging the bed up to station no. 0.305 at a slope 0.00025 width a width of 12m. The water surface comes to 22.46m. The quantity of excavation required to be 65984 m^3. This is shown in figure no 5.7 and table no. 5.1.

5.2.2. Alternative no. 2 i.e. widening the channel at a width of 16m from (a) station no. 0.79 to 0.595 (b) again from station no 0.495 to 0.395 by dredging. The water surface comes to 22.38m .The quantity of excavation required to be 57847 m^3. This is shown in figure no 5.8 and table no. 5.2.

5.2.3. Alternative no. 3 i.e. deepening the bed level to 16.30m & widening with a width of 20m. (a) From station 0.79 to 0.695 (average deepening = 0.5m) (b) fromstation 0.545 to 0.445 (average deepening = 0.6m) (c) from station 0.445 to 0.305 (average deepening = 0.54m). The water surface level comes to 22.17m for 200cumec discharges. But lowered down from 19.30m to 19.06m for100cumecdischarge. The quantity of excavation required to be 88474 m^3. This is shown in figure no.5.9and table no.5.3

5.2.4 Alternative 4 i.e. deepening the bed level to 16.40m for a width of14m (a)from station no. 0.79 to 0.695 (average deepening = 0.42m) (b) from station no. 0.545 to 0.445 (average deepening = 0.50m) (c) from station 0.445 to 0.345(average deepening = 0.59m) (d) from station no. 0.345 to 0.305 (average deepening = 0.46m) The water surface comes to 22.46m. The quantity of excavation required to be 49727 m^3. This is shown in figure no.5.10 and table no.5.4.

5.2.5 Alternative no 5 i.e. Deepening the bed level to 16.40m for a width of 8m. (a) from station no. 0.79 to 0.695 (average deepening = 0.42m) (b)from station 0.545 to 0.445 (average deepening = 0.50m) (c) from station 0.445 to 0.345 (average deepening = 0.59m). (d) from station 0.345 to 0.305 (average deepening = 0.46m) . The water surface comes to 22.63m. The quantity of excavation required to be 27328 m^3. This is shown in figure no.5.11and table no.5.5

5.2.6. Alternative no. 6 i.e. Deepening the bed level to 16.40m for a width of 10m.(a) from station no. 0.79 to 0.695 (average deepening = 0.42m) (b)from station 0.545 to 0.445 (average deepening = 0.50m) (c) from station 0.445 to 0.345 (average deepening = 0.59m). (d) from station 0.345 to 0.305 (average deepening = 0.46m) . The water surface comes to 22.59m. The quantity of excavation required to be 34310 m^3. This is shown in figure no. 5.12and table no. 5.6.

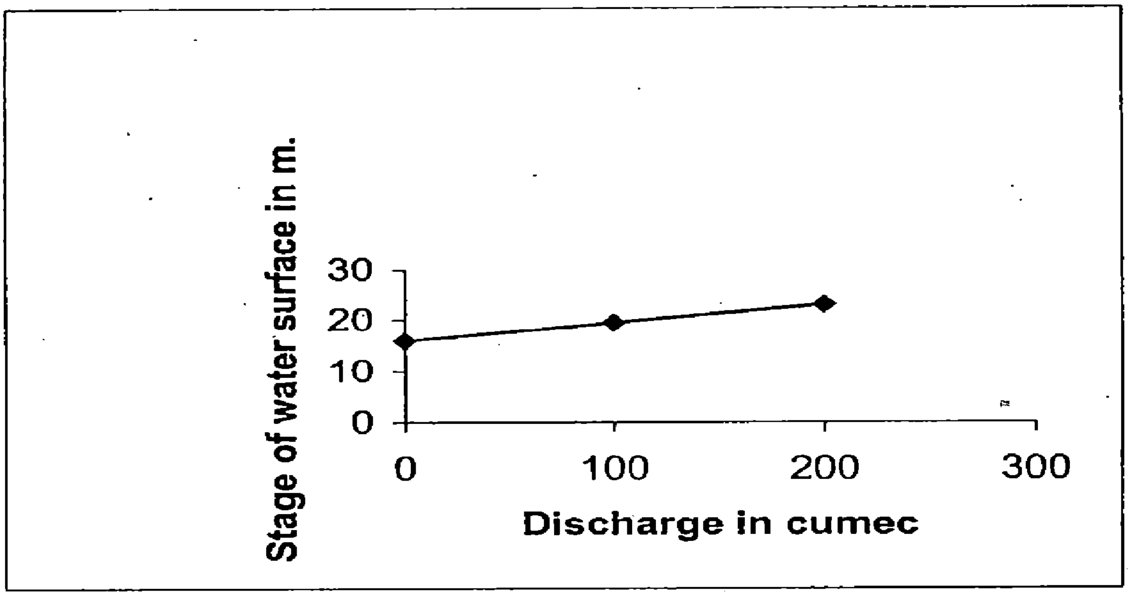


Fig. 5.1 Verification of W.L. through calibration for steady flow at stn. no. 0.895

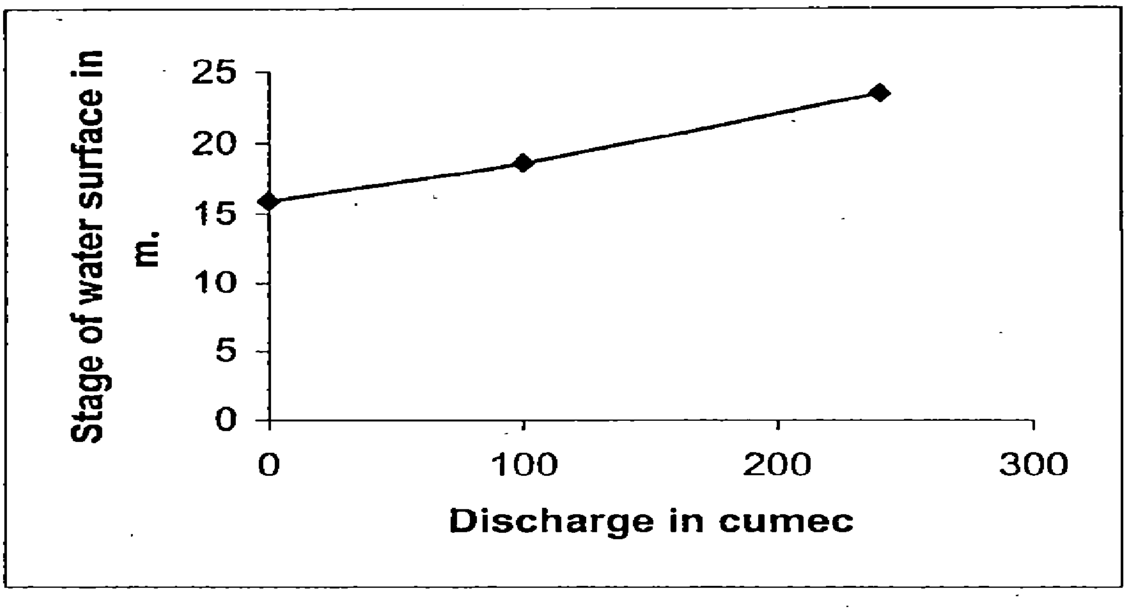


Fig- 5.2 Verification of stage for higher discharge (240 cumec) at stn no. 0.895

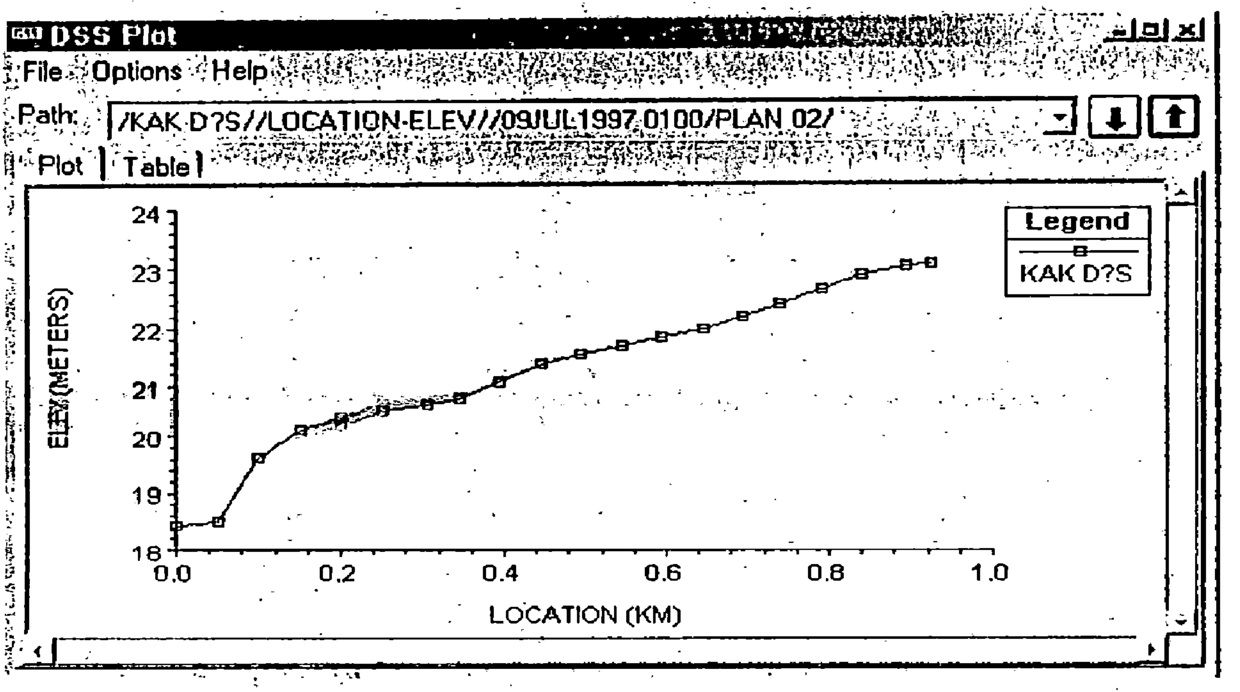


Fig 5.3 The elevation of water surface During model calibration

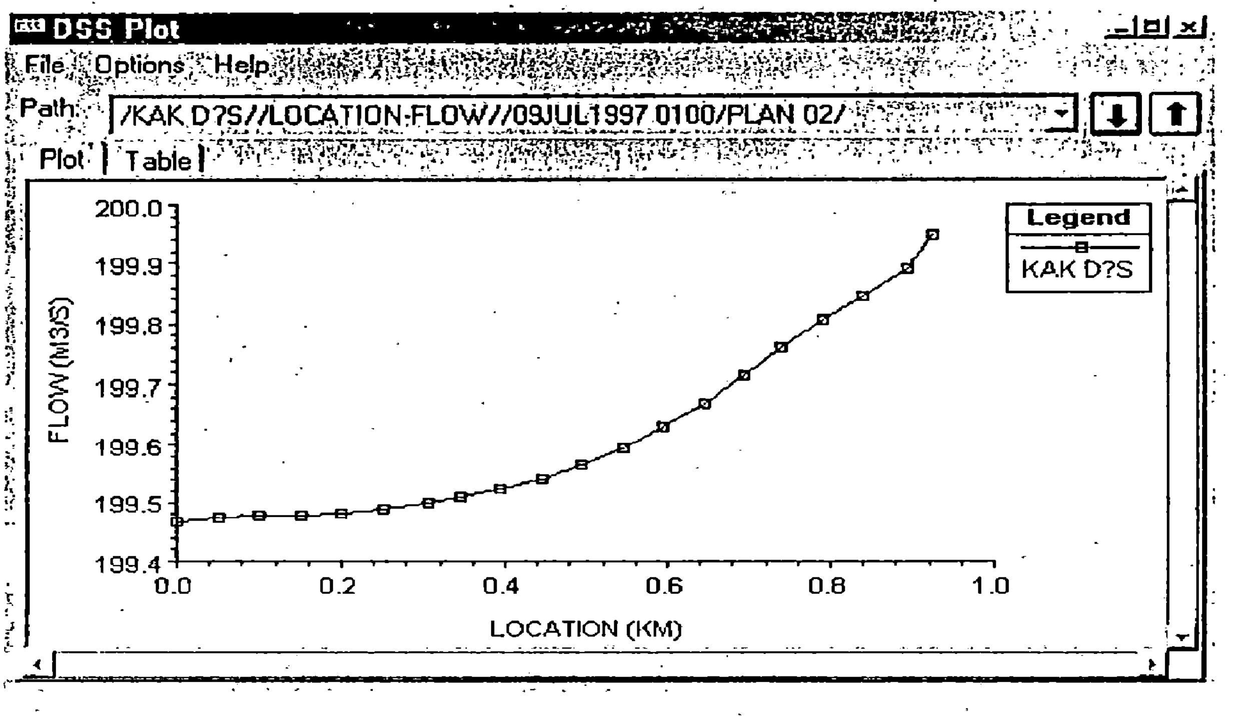


Fig 5.4 Flow **amount at different stations.**

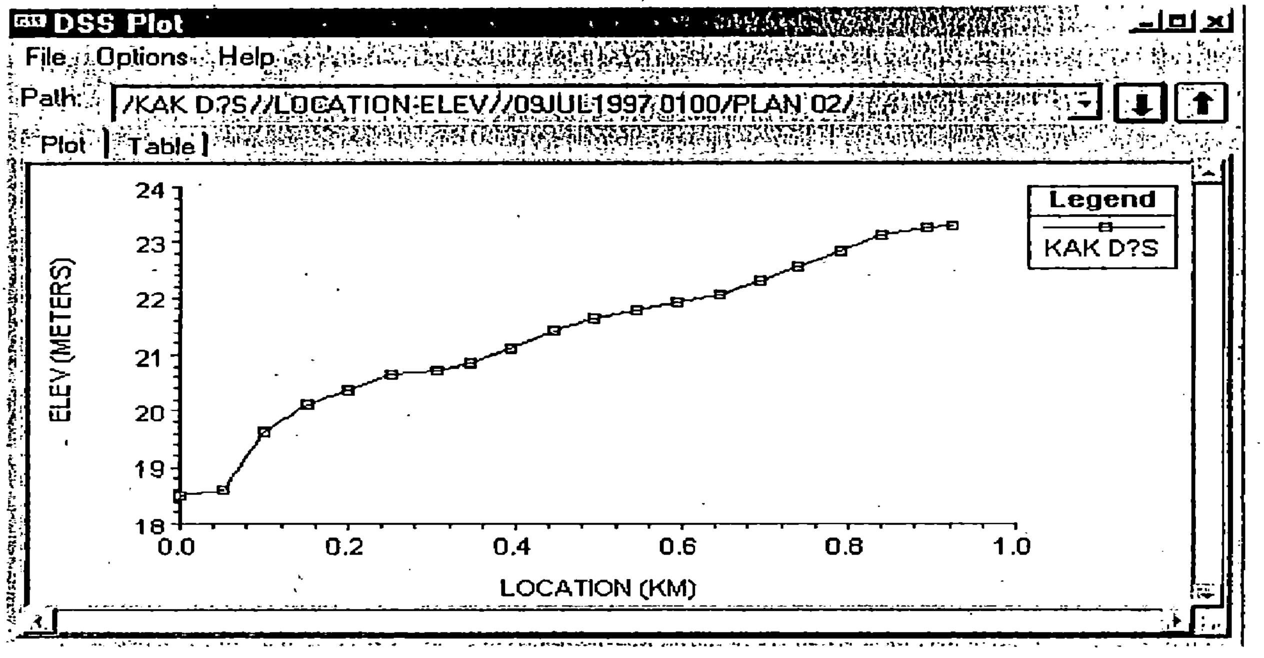


Fig 5.5 The elevation of water surface during model verification.

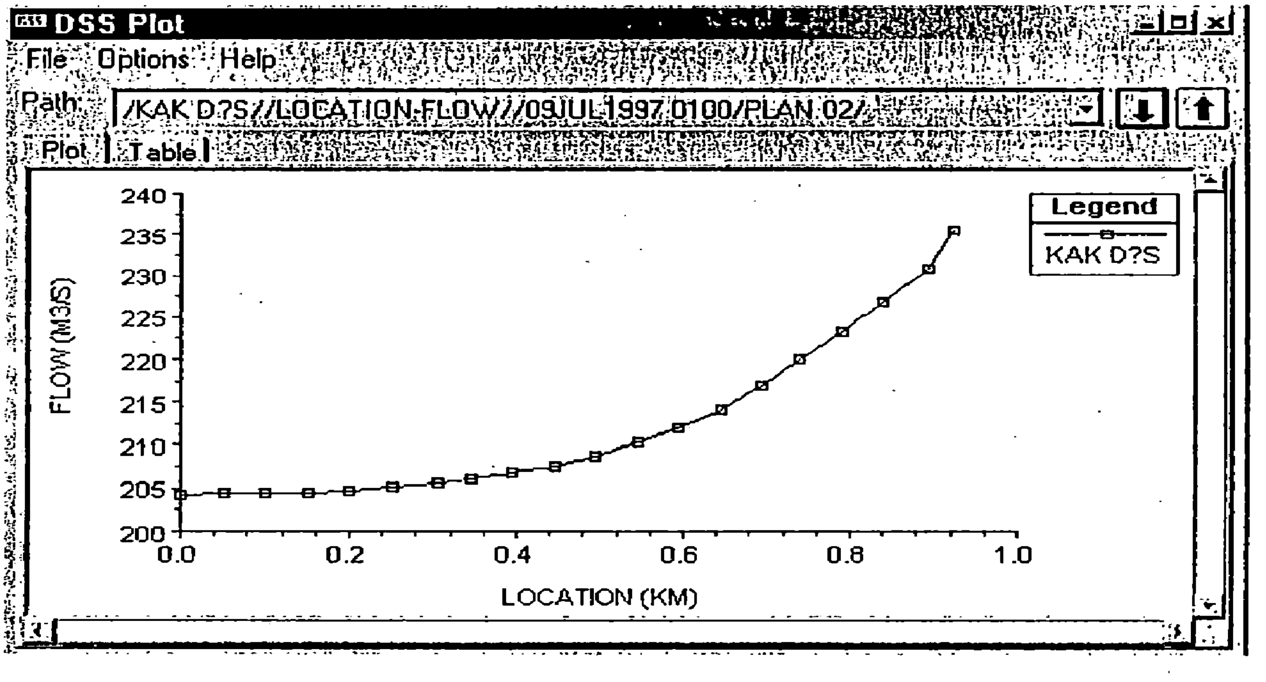


Fig 5.6 flow amounts at different river stations .

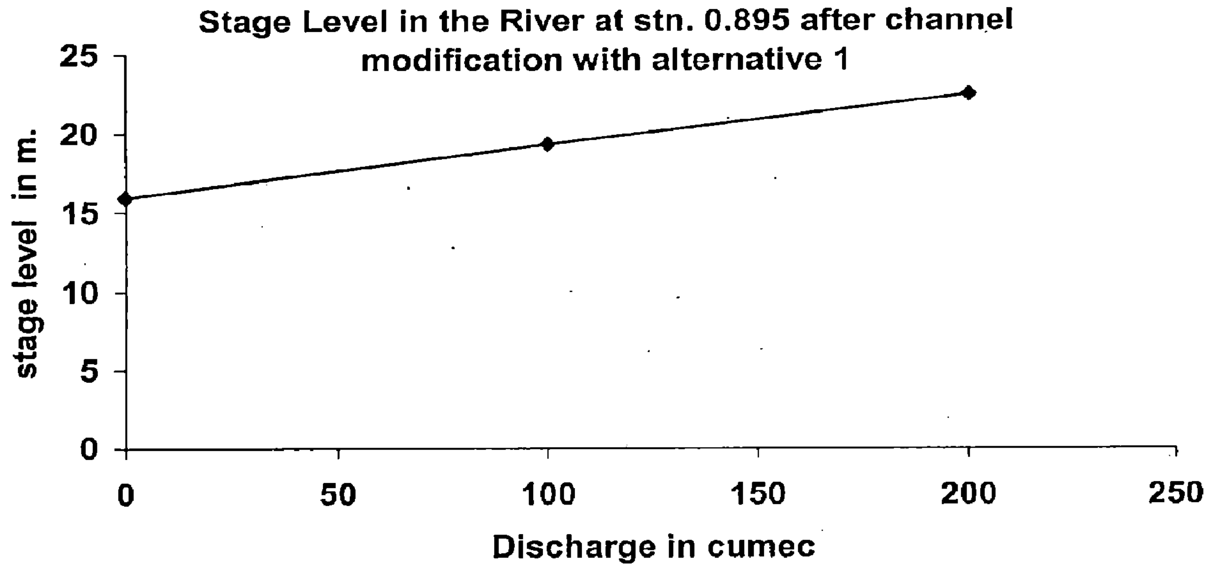

Stage Level in the River at stn. 0.895 after channel modification with alternative 1
stage level in m.
25
20
15
10
5
0
0
50
100
150
200
250
Discharge in cumec

Fig. 5.7

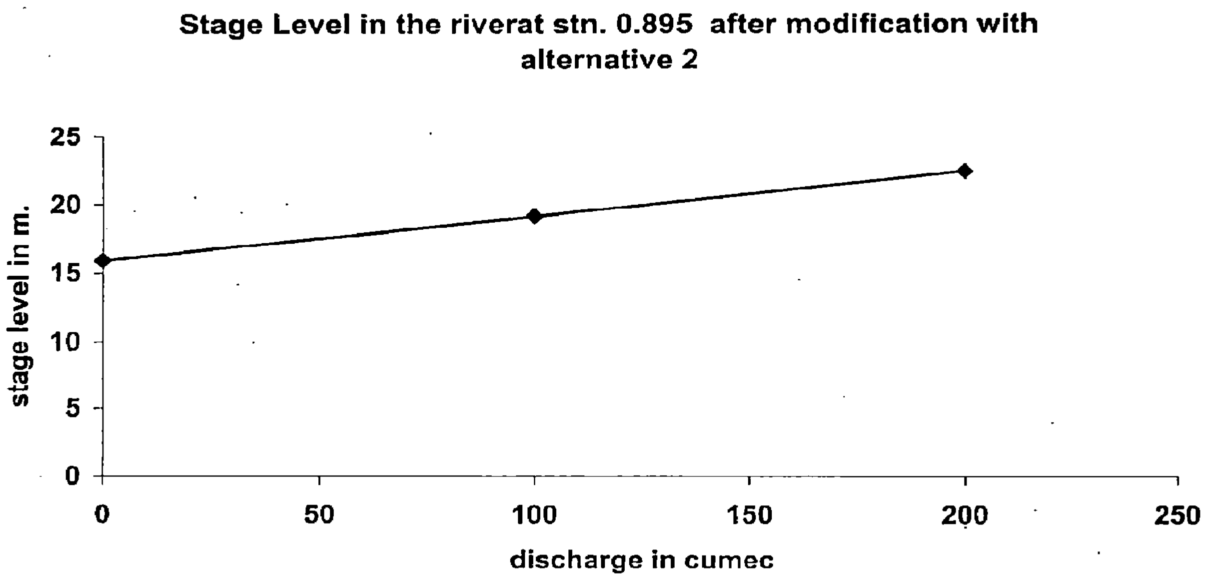

Stage Level in the riverat stn. 0.895 after modification with alternative 2
stage level in m.
25
20
15
10
5
0
0
50
100
150
200
250
discharge in cumec

Fig. 5.8

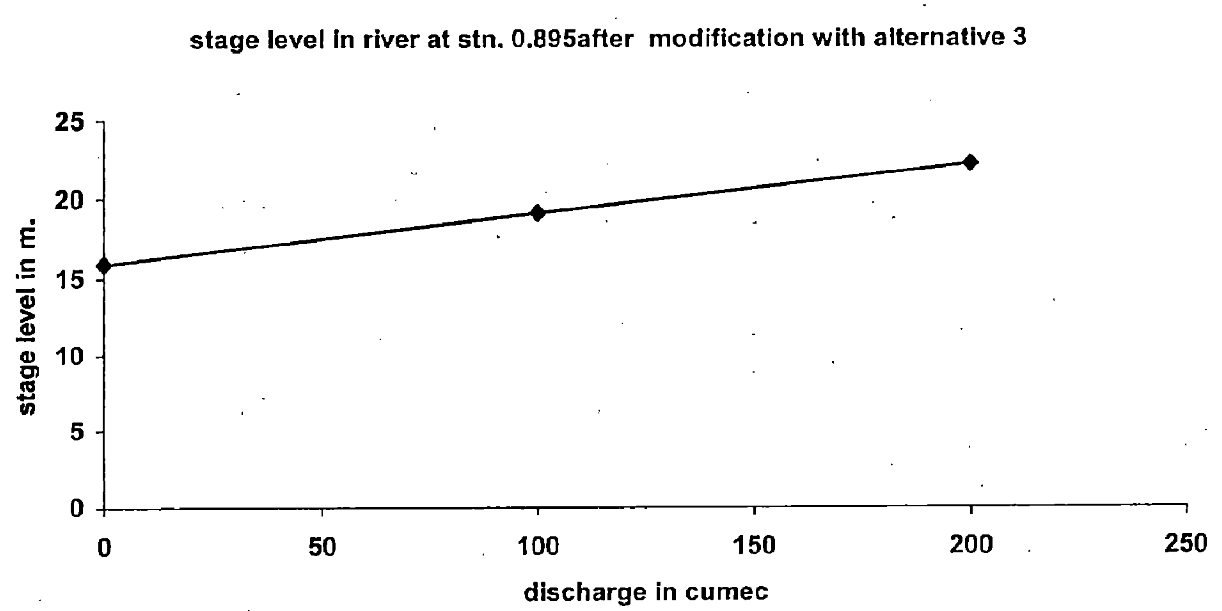


Fig. 5.9

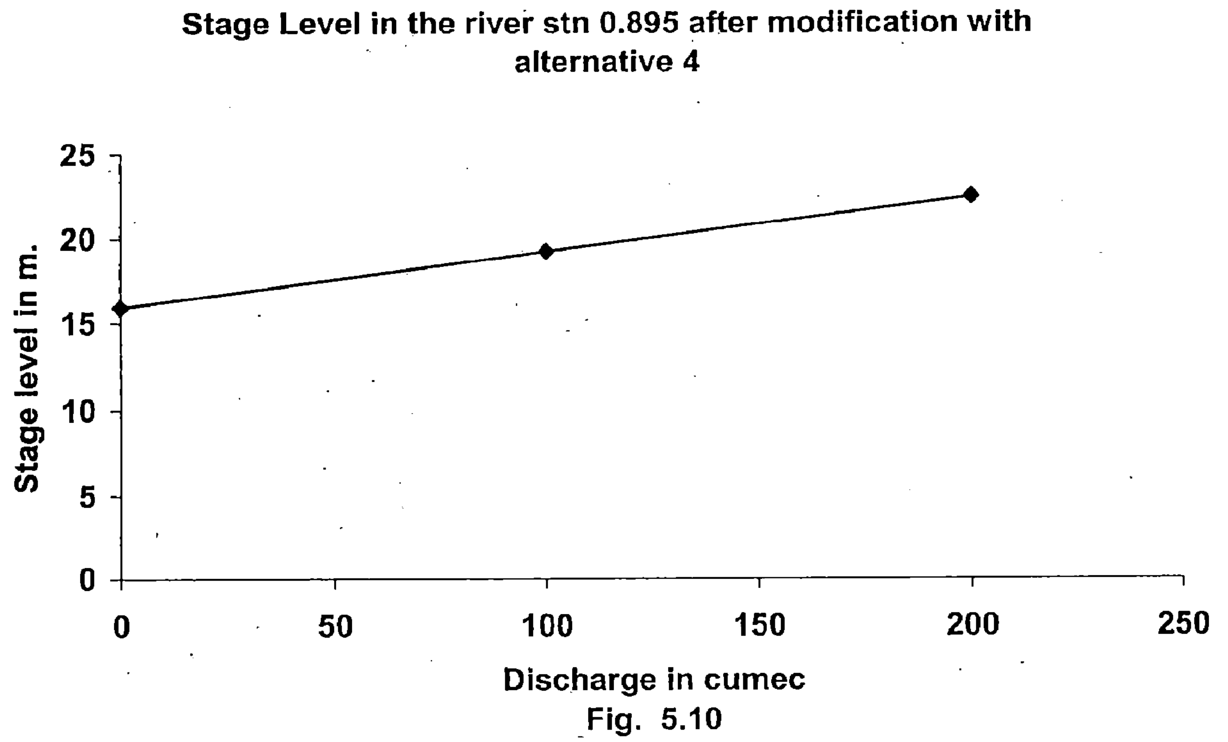


Fig. 5.10

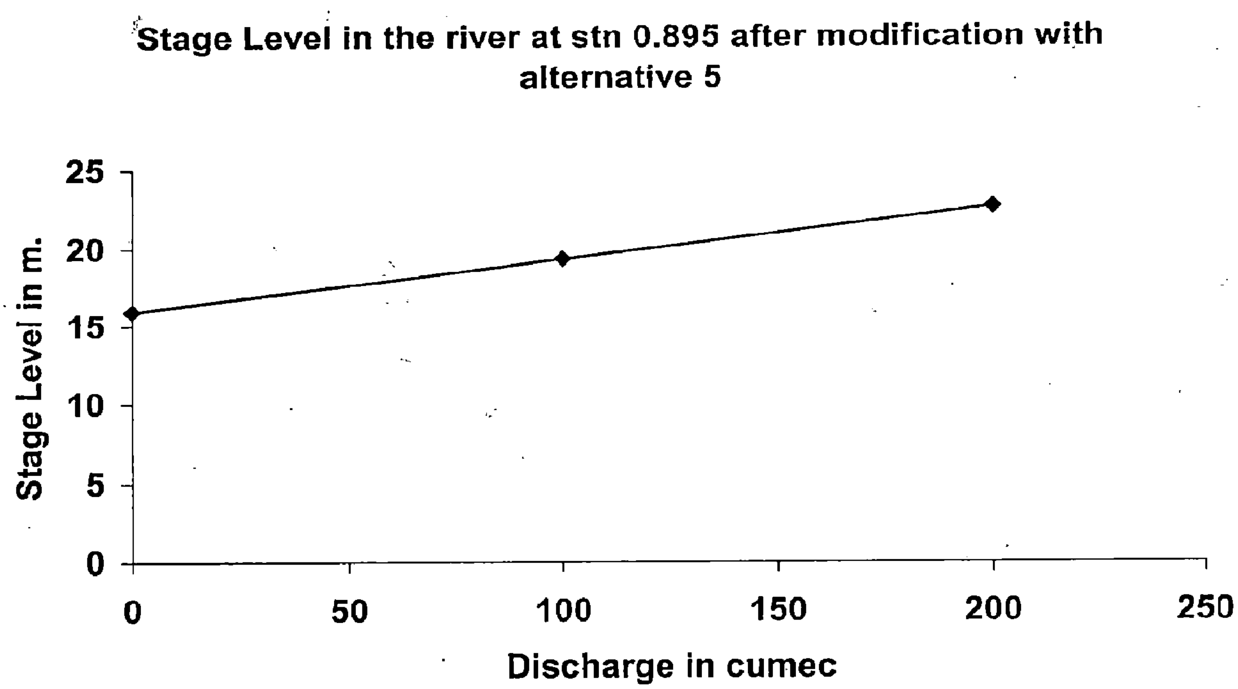


Fig. 5.11

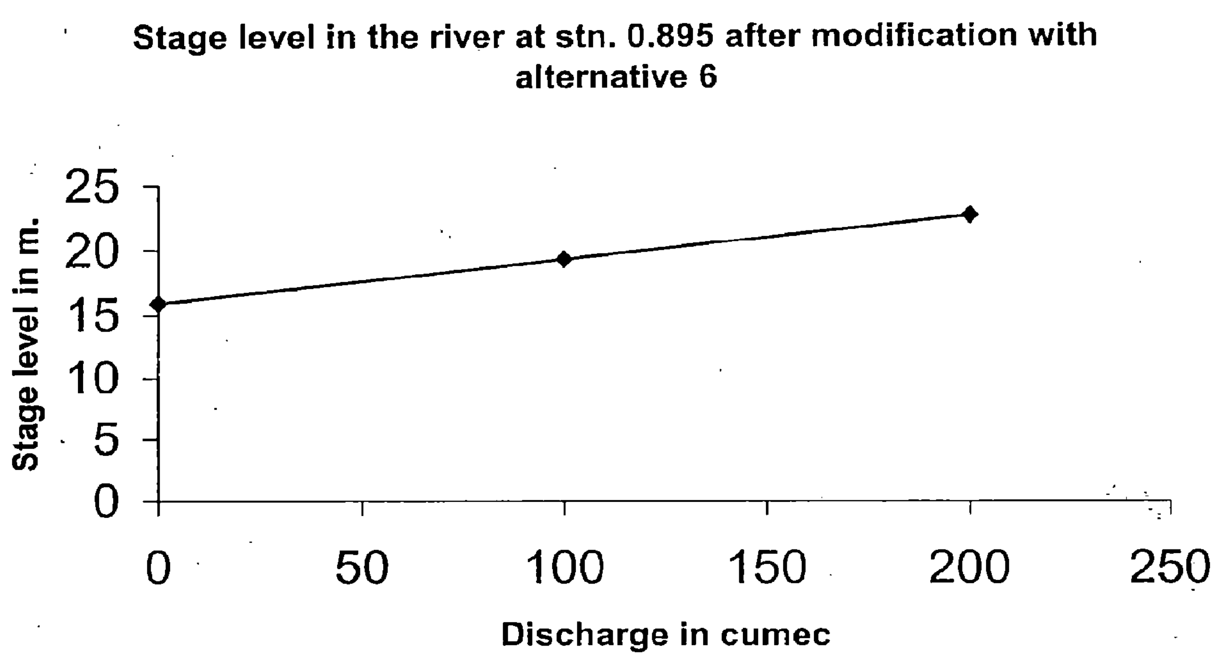


Fig. 5.12

Quantity of Excavation to be Required by channel modification with Alternative-(1)

River *Kakkad* **Reach** *d/s*

Table 5.1

river	option	Area L	Area C.H.	Area R	Area T	Volume L	Vol C.H.	vol R	Vol T
station	cut/fill	sqm	sqm	sqm	sqm	cum	cum	cum	cum
	cut	0	14	0	14	0	10244	0	10244
0.79	fill	0	0	0	0	0	0	0	0
	net	0	14	0	14	0	10244	0	10244
	cut	0	12	0	12	0	9874	0	9874
0.74	fill	0	0	0	0	0	0	0	0
	net	0	12	0	12	0	9874	0	9874
	cut	0	15	0	15	0	5191	0	5191
0.695	fill	0	0	0	0	0	0	0	0
	net	0	15	0	15	0	5191	0	5191
	cut	0	8	0	8	0	4877	0	4877
0.545	fill	0	0	0	0	0	0	0	0
	net	0	8	0	8	0	4877	0	4877
	cut	0	10	0	10	0	7230	0	7230
0.495	fill	0	0	0	0	0	0	0	0
	net	0	10	0	10	0	7230	0	7230
	cut	0	20	0	20	0	4364	0	4364
0.445	fill	0	0	0	0	0	0	0	0
	net	0	20	0	20	0	4364	0	4364
	cut	0	15	0	15	0	5010	0	5010
0.345	fill	0	0	0	0	0	0	0	0
	net	0	15	0	15	0	5010	0	5010
	cut	0	15	0	15	0	2215	0	2215
0.305	fill	0	0	0	0	0	0	0	0
	net	0	15	0	15	0	2215	0	2215
	cut	0				0	49005	0	49005
total	fill	0				0	0	0	0
	net	0				0	49005	0	49005

Quantity of Excavation to be Required by channel modification with Alternative-(2)

River *Kakkad* **Reach** *d/s*

Table 5.2

river	option	Area L	Area C.H.	Area R	Area T	Volume L	Vol C.H.	vol R	Vol T
station	cut/fill	sqm	sqm	sqm	sqm	cum	cum	cum	cum
	cut	0	13	0	13	0	10403	0	10403
0.79	fill	0	0	0	0	0	0	0	0
	net	0	13	0	13	0	10403	0	10403
	cut	0	14	0	14	0	5108	0	5108
0.74	fill	0	0	0	0	0	0	0	0
	net	0	14	0	14	0	5108	0	5108
	cut	0	14	0	14	0	7550	0	7550
0.74	fill	0	0	0	0	0	0	0	0
	net	0	14	0	14	0	7550	0	7550
	cut	0	7	0	7	0	2294	0	2294
0.695	fill	0	0	0	0	0	0	0	0
	net	0	7	0	7	0	2294	0	2294
	cut	0	7	0	7	0	4052	0	4052
0.695	fill	0	0	0	0	0	0	0	0
	cut	0	7	0	7	0	4052	0	4052
	cut	0	5	0	5	0	1632	0	1632
0.645	fill	0	0	0	0	0	0	0	0
	cut	0	5	0	5	0	1632	0	1632
	cut	0	5	0	5	0	3536	0	3536
0.645	fill	0	0	0	0	0	0	0	0
	net	0	5	0	5	0	3536	0	3536
	cut	0	6	0	6	0	1757	0	1757
0.595	fill	0	0	0	0	0	0	0	0
	net	0	6	0	6	0	1757	0	1757
	cut	0		0		0	2167	0	2167
0.495	fill	0	0	0	0	0	0	0	0
	net	0		0		0	2167	0	2167
	cut	0	9	0	9	0	2714	0	2714
0.445	fill	0	0	0	0	0	0	0	0
	net	0	9	0	9	0	2714	0	2714
	cut	0	4	0	4	0	710	0	710
0.395	fill	0	0	0	0	0	0	0	0
	net	0	4	0	4	0	710	0	710
	cut	0				0	41923	0	41923
total	fill	0				0	0	0	0
	net	0				0	41923	0	41923

Quantity of Excavation to be Required by channel modification with Alternative-(3)

River *Kakkad* **Reach** *d/s*

Table 5.3

river	option	Area L	Area C.H.	Area R	Area T	Volume L	Vol C.H.	vol R	Vol T
station	cut/fill	sqm	sqm	sqm	sqm	cum	cum	cum	cum
	cut	0	28	1	29	0	18583	272	18855
0.79	fill	0	0	0	0	0	0	0	0
	net	0	28	1	29	0	18583	272	18855
	cut	0	19	0	19	0	13997	0	13997
0.74	fill	0	0	0	0	0	0	0	0
	net	0	19	0	19	0	13997	0	13997
	cut	0	19	0	19	0	6672	0	6672
0.695	fill	0	0	0	0	0	0	0	0
	net	0	19	0	19	0	6672	0	6672
	cut	0	15	0	15	0	8074	0	8074
0.545	fill	0	0	0	0	0	0	0	0
	net	0	15	0	15	0	8074	0	8074
	cut	0	14	0	14	0	9438	0	9438
0.495	fill	0	0	0	0	0	0	0	0
	net	0	14	0	14	0	9438	0	9438
	cut	0	24	0	24	0	5272	0	5272
0.445	fill	0	0	0	0	0	0	0	0
	net	0	24	0	24	0	5272	0	5272
	cut	0	24	0	24	0	9742	471	10213
0.445	fill	0	0	0	0	0	0	0	0
	net	0	24	0	24	0	9742	471	10213
	cut	0	20	0	20	0	6826	418	7245
0.395	fill	0	0	0	0	0	0	0	0
	net	0	20	0	20	0	6826	418	7245
	cut	0	14	0	14	0	2498	0	2498
0.345	fill	0	0	0	0	0	0	0	0
	net	0	14	0	14	0	2498	0	2498
	cut	0	14	0	14	0	4468	0	5272
0.345	fill	0	0	0	0	0	0	0	0
	net	0	14	0	14	0	4468	0	5272
	cut	0	11	0	11	0	1742	0	710
0.305	fill	0	0	0	0	0	0	0	0
	net	0	11	0	11	0	1742	0	710
	cut	0				0	87085	1161	88246
total	fill	0				0	0	0	0
	net	0				0	87085	1161	88246

Quantity of Excavation to be Required by channel modification with Alternative-(4)

River *Kakkad* **Reach** *d/s*

Table 5.4

river station	option cut/fill	Area L sqm	Area C.H. sqm	Area R sqm	Area T sqm	Volume L cum	Vol C.H. cum	vol R cum	Vol T cum
0.79	cut	0	17	0	17	0	10674	0	10674
	fill	0	0	0	0	0	0	0	0
	net	0	17	0	17	0	10674	0	10674
0.74	cut	0	10	0	10	0	7541	0	7541
	fill	0	0	0	0	0	0	0	0
	net	0	10	0	10	0	7541	0	7541
0.695	cut	0	11	0	11	0	3659	0	3659
	fill	0	0	0	0	0	0	0	0
	net	0	11	0	11	0	3659	0	3659
0.545	cut	0	8	0	8	0	4392	0	4392
	fill	0	0	0	0	0	0	0	0
	net	0	8	0	8	0	4392	0	4392
0.495	cut	0	8	0	8	0	5688	0	5688
	fill	0	0	0	0	0	0	0	0
	net	0	8	0	8	0	5688	0	5688
0.445	cut	0	15	0	15	0	3390	0	3390
	fill	0	0	0	0	0	0	0	0
	net	0	15	0	15	0	3390	0	3390
0.445	cut	0	15	0	15	0	5841	0	5841
	fill	0	0	0	0	0	0	0	0
	net	0	15	0	15	0	5841	0	5841
0.395	cut	0	11	0	11	0	3787	0	3787
	fill	0	0	0	0	0	0	0	0
	net	0	11	0	11	0	3787	0	3787
0.345	cut	0	8	0	8	0	1406	0	1406
	fill	0	0	0	0	0	0	0	0
	net	0	8	0	8	0	1406	0	1406
0.345	cut	0	8	0	8	0	3377	0	3377
	fill	0	0	0	0	0	0	0	0
	net	0	8	0	8	0	3377	0	3377
0.305	cut	0	11	0	11	0	1742	0	1742
	fill	0	0	0	0	0	0	0	0
	net	0	11	0	11	0	1742	0	1742
total	cut	0				0	51497	0	51497
	fill	0				0	0	0	0
	net	0				0	51497	0	51497

Quantity of Excavation to be Required by channel modification with Alternative-(5)

River *Kakkad* **Reach** *d/s*

Table-5.5

river	option	Area L	Area C.H.	Area R	Area T	Volume L	Vol C.H.	vol R	Vol T
station	cut/fill	sqm	sqm	sqm	sqm	cum	cum	cum	cum
	cut	0	10	0	10	0	6035	0	6035
0.79	fill	0	0	0	0	0	0	0	0
	net	0	10	0	10	0	6035	0	6035
	cut	0	5	0	5	0	4005	0	4005
0.74	fill	0	0	0	0	0	0	0	0
	net	0	5	0	5	0	4005	0	4005
	cut	0	5	0	5	0	1862	0	1862
0.695	fill	0	0	0	0	0	0	0	0
	net	0	5	0	5	0	1862	0	1862
	cut	0	4	0	4	0	2403	0	2403
0.545	fill	0	0	0	0	0	0	0	0
	net	0	4	0	4	0	2403	0	2403
	cut	0	4	0	4	0	3363	0	3363
0.495	fill	0	0	0	0	0	0	0	0
	net	0	4	0	4	0	3363	0	3363
	cut	0	9	0	9	0	2058	0	2058
0.445	fill	0	0	0	0	0	0	0	0
	net	0	9	0	9	0	2058	0	2058
	cut	0	9	0	9	0	4510	0	4510
0.445	fill	0	0	0	0	0	0	0	0
	net	0	9	0	9	0	4510	0	4510
	cut	0	11	0	11	0	3787	0	3787
0.395	fill	0	0	0	0	0	0	0	0
	net	0	11	0	11	0	3787	0	3787
	cut	0	8	0	8	0	1406	0	1406
0.345	fill	0	0	0	0	0	0	0	0
	net	0	8	0	8	0	1406	0	1406
	cut	0	8	0	8	0	3377	0	3377
0.345	fill	0	0	0	0	0	0	0	0
	net	0	8	0	8	0	3377	0	3377
	cut	0	11	0	11	0	1742	0	1742
0.305	fill	0	0	0	0	0	0	0	0
	net	0	11	0	11	0	1742	0	1742
	cut	0				0	34548	0	34548
total	fill	0				0	0	0	0
	net	0				0	34548	0	34548

Quantity of Excavation to be Required by channel modification with Alternative-(6)

Table 5.6

River *Kakkad* **Reach** *d/s*

river station	option cut/fill	Area L sqm	Area C.H. sqm	Area R sqm	Area T sqm	Volume L cum	Vol C.H. cum	vol R cum	Vol T cum
0.79	cut	0	12	0	12	0	7476	0	7476
	fill	0	0	0	0	0	0	0	0
	net	0	12	0	12	0	7476	0	7476
0.74	cut	0	7	0	7	0	5110	0	5110
	fill	0	0	0	0	0	0	0	0
	net	0	7	0	7	0	5110	0	5110
0.695	cut	0	7	0	7	0	2411	0	2411
	fill	0	0	0	0	0	0	0	0
	net	0	7	0	7	0	2411	0	2411
0.545	cut	0	4	0	4	0	2403	0	2403
	fill	0	0	0	0	0	0	0	0
	net	0	4	0	4	0	2403	0	2403
0.495	cut	0	4	0	4	0	3363	0	3363
	fill	0	0	0	0	0	0	0	0
	net	0	4	0	4	0	3363	0	3363
0.445	cut	0	9	0	9	0	2058	0	2058
	fill	0	0	0	0	0	0	0	0
	net	0	9	0	9	0	2058	0	2058
0.445	cut	0	9	0	9	0	4510	0	4510
	fill	0	0	0	0	0	0	0	0
	net	0	9	0	9	0	4510	0	4510
0.395	cut	0	11	0	11	0	3787	0	3787
	fill	0	0	0	0	0	0	0	0
	net	0	11	0	11	0	3787	0	3787
0.345	cut	0	8	0	8	0	1406	0	1406
	fill	0	0	0	0	0	0	0	0
	net	0	8	0	8	0	1406	0	1406
0.345	cut	0	6	0	6	0	1679	0	1679
	fill	0	0	0	0	0	0	0	0
	net	0	6	0	6	0	1679	0	1679
0.305	cut	0	4	0	4	0	613	0	613
	fill	0	0	0	0	0	0	0	0
	net	0	4	0	4	0	613	0	613
total	cut	0				0	34816	0	34816
	fill	0				0	0	0	0
	net	0				0	34816	0	34816

Table 5.7 Additional power during flood when there is spill of 100 cumec of discharge from barrage and another 100 cumec of release from powerhouse after different channel modification alternative (1-6)

Level at fore bay (m)	TWL during flood before channel improvement (m)	Head available before channel improvement. (m)	Theoretical power produced in MW.	Level at fore bay after channel improvement, (m)	TWL after channel improvement. (m)	Head due to channel improvement. (m)	Theoretical power after channel improvement, (MW)	Extra power due to channel improvement. (MW)
32.5	23	9.5	7.448	32.5	22.46	10.04	7.871	0.423
32.5	23	9.5	7.448	32.5	22.38	10.12	7.933	0.485
32.5	23	9.5	7.448	32.5	22.17	10.33	8.099	0.6510
32.5	23	9.5	7.448	32.5	22.46	10.04	7.871	0.423
32.5	23	9.5	7.448	32.5	22.63	9.87	7.738	0.290
32.5	23	9.5	7.448	32.5	22.59	9.91	7.769	0.321

DISCUSSION OF RESULTS AND CONCLUSIONS

6.1. GENERAL:

This chapter presents the results of this work and the conclusions leading from the application of unsteady and steady numerical model for studying the modification of the river channel by deepening, widening or combination of both.

6.1. 1 Model Results:

Due to the non availability of the flow hydrograph at the barrage site and at the power house site for input into unsteady flow model, steady flow data was used in the unsteady flow analysis format in the software for calibration of the river model. Hence the accuracy of the unsteady flow model could not be tested. However the results of this study can be analyzed qualitatively to assess the efficiency of the unsteady flow modeling technologies.

6.1.2. Results of Comparison of Steady and Unsteady Flow Modelling:

An additional analysis of the steady state flow was conducted using the HEC RAS steady flow model to compare steady flow modelling to unsteady flow modelling. When using a steady flow model, most modelers consider the peak flows at the boundary condition for a specified flow event, resulting in water stage height being significantly higher than (26cm in this case) one for the unsteady flow model This occurs because the steady flow model does not account for the time differences in the flow hydrographs.

Based on this comparison, the unsteady flow model may be considered to provide significant points to merit for future design and modelling

6.2 GENERAL COMMENTS ON THE PRESENT MODELLING RESULT:

6.2.1 Model Calibration :

During calibration process apart from friction resistance form resistance plays a significant role in selecting the Manning ' s n values.The value of Manning ' s n is increased from 0.034 to 0.048 for the bed at the different reaches after studying the vertical variability of the invert levels of different cross sections from the longitudinal profile.

6.2.2. Unsteady flow model operation:

Many factors create difficulties in the modelling process for unsteady flow models that are not an issue for steady flow models. Those are on the basis of present modelling.

The unsteady flow algorithms are not as numerical stable as steady flow models. When a water stage height is calculated at a value less than the stream beds elevation the simulation process crashes. To tackle such difficulties of unsteady flow modeling for a natural stream , additional cross section may be added to the model , which decreases the incremental steps in the stage difference from node to node in the system.

6.3 COMMENTS ON RESULTS OF DIFFERENT CHANNEL MODIFICATION ALTERNATIVES

Type of Alternative.	Mitigation steps taken in different alternatives	Gain in head in m.	Gain in power in MW	Quantity of-dredging in cum
Alternative. 1	Limited deepening & 12m widening.	0.54	0.423	65984
Alternative.2	No deepening only 16m widening	0.51	0.400	57847
Alternative. 3	Deepening by 0.55m on an average & 20m widening.	0.83	.6510	88474
Alternative.4	Deepening by 0.45m on an average & 14m widening.	0.54	0.423	49727
Alternative.5	Deepening by 0.45m on an average & 8m widening.	0.37	0.290	27328
Alternative.6	Deepening by 0.45m on an average & 10m widening.	0.41	0.321	34310

Table-6.1

From the above findings it is clear that if we deepen the main channel by 0.55m, widen the right side of the main channel by 20m at selective reaches (as per alternative -3) and lower the Manning's n value from 0.034 to 0.024(by 25%) the stage level of the water surface can be lowered by 0.83m during the flood periods. This lowering down of TWL results in a gain of head about 8.8% in the monsoon period.

6.4. CONCLUSIONS:

From the present, the study following important conclusions may be drawn:-

(i) It is evident from the study that any channel modification from below the confluence point (TRC and main river) will contribute to head gain.

(ii) A channel improvement attempt in which bed has been deepened by 0.55m at selective reaches and right side of main channel is widened by 20m has been found to produce the best result. Since the rock is good, the side slope which has been taken as 0.75: 1 is with in limit. The total excavation comes to be 88474 m^3. The gain in power is 0.6510 MW.

(iii) Beyond 0.55m bed deepening there is a possibility of additional power generation during the floods, but during lean period this results in TWL reaching below the MWL (18.85m) which may result in problem of cavitation.

6.5 FUTURE SCOPE:

(i) Remote sensing images may provide physical parameters of channel network for improved modelling parameters .

(ii) The parameters may be collected through field experimentation for verification of results obtained from the modelling procedure.

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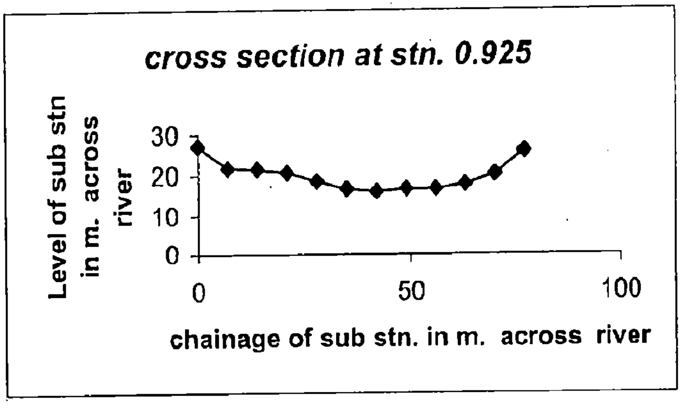
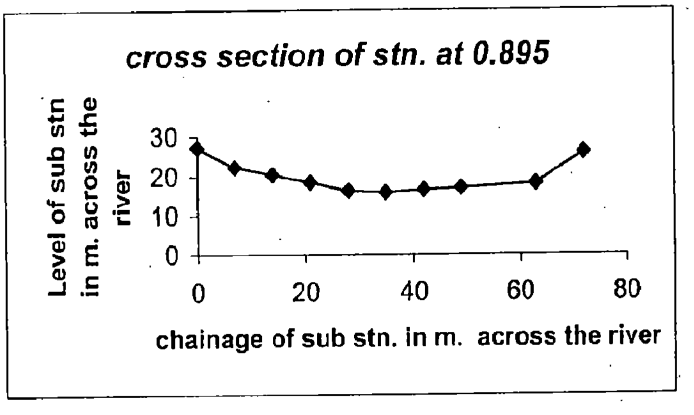
Appendix A-1

CROSS SECTION OF RIVER AT DIFFERMENT STATION WITH CALCULATED 'N' VALUE
FROM CONFLUENCE TO CONTROL POINT

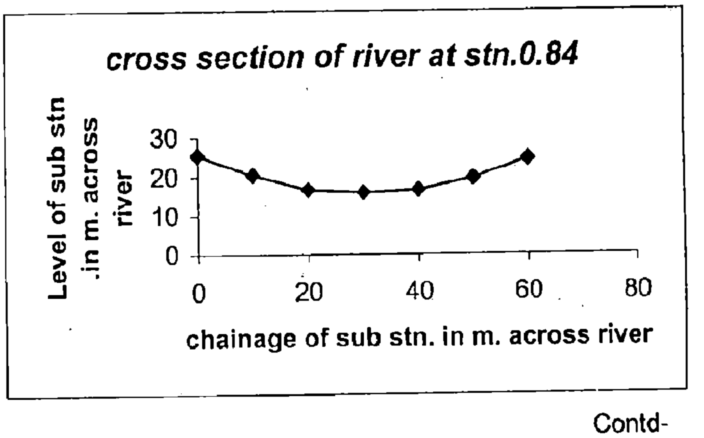
Stn. No.	Chainage	Level	Manning'n
	0	27.196	0.065
	7	21.776	0.065
	14	21.488	0.065
	21	20.65	0.065
	28	18.56	0.065
0.925	35	16.729	0.034
	42	16.129	0.034
	49	16.729	0.034
	56	16.729	0.034
	63	17.865	0.065
	70	20.26	0.05
	77	26	0.065



	0	27.2	0.065
	7	22.3	0.065
	14	20.4	0.065
	21	18.4	0.065
	28	16.31	0.034
0.895	35	15.9	0.034
	42	16.5	0.034
	49	17	0.034
	63	18.1	0.065
	72	26	0.065

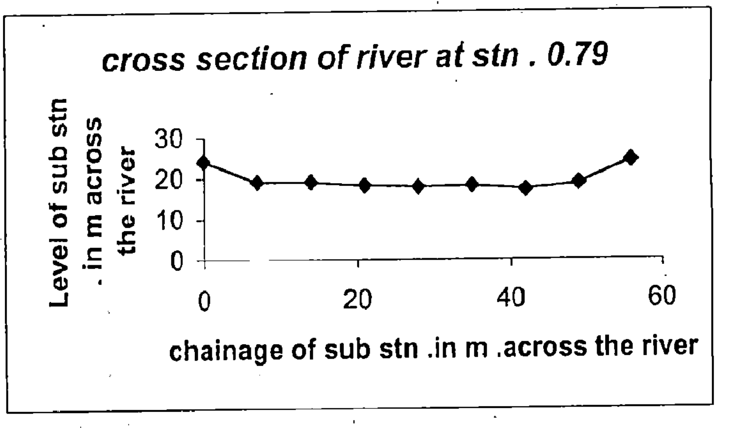


	0	25.5	0.065
	10	20.4	0.065
	20	16.7	0.065
0.84	30	15.9	0.034
	40	16.6	0.065
	50	19.6	0.065
	60	24.5	0.065

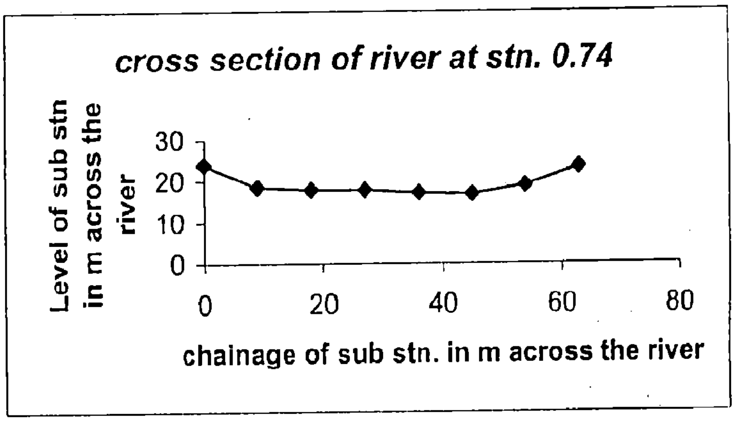


Contd-

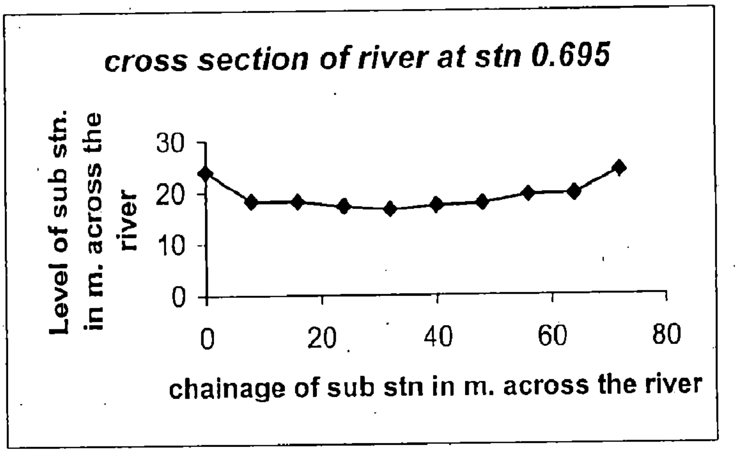
	0	24	0.118
	7	19.01	0.118
	14	18.93	0.118
	21	18.01	0.118
	28	17.6	0.048
0.79	35	17.91	0.048
	42	17	0.048
	49	18.5	0.118
	56	24	0.118



	0	23.96	0.118
	9	18.5	0.118
	18	17.8	0.048
	27	17.84	0.048
0.74	36	17.1	0.048
	45	16.76	0.048
	54	18.9	0.118
	63	23.38	0.118

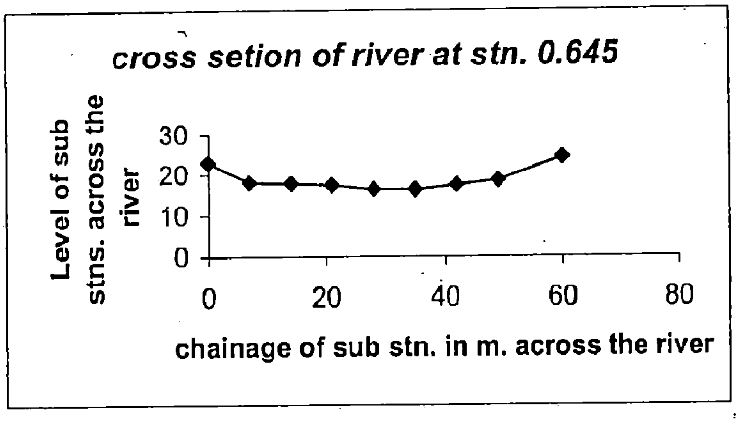


	0	23.96	0.118
	8	18.37	0.118
	16	18.28	0.118
	24	17.4	0.048
	32	16.7	0.048
	40	17.5	0.048
0.695	48	17.9	0.118
	56	19.5	0.118
	64	19.7	0.118
	72	24.2	0.118

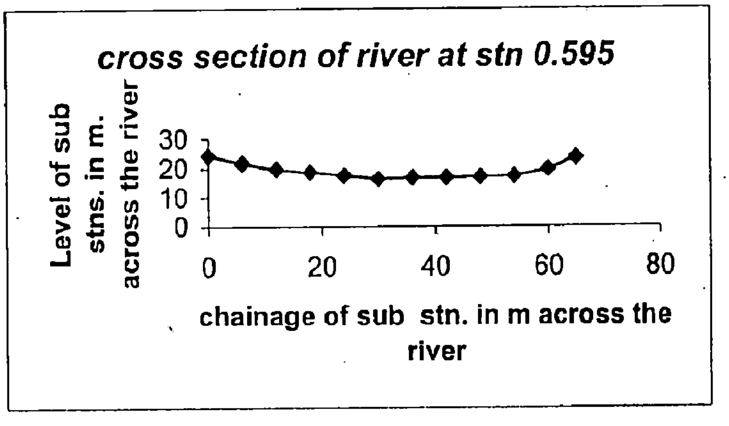


Contd-

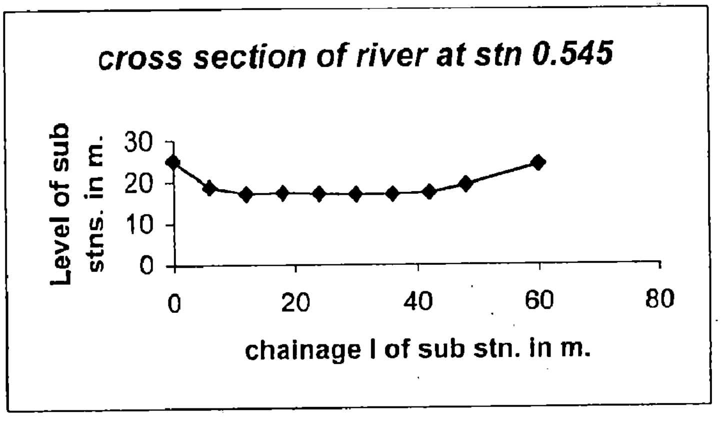
	0	23	0.118
	7	18	0.118
	14	17.7	0.048
	21	17.3	0.048
	28	16.3	0.048
0.645	35	16.15	0.048
	42	17.3	0.048
	49	18.5	0.118
	60	24.216	0.118



	0	24.2	0.065
	6	21.7	0.065
	12	19.6	0.065
	18	18.58	0.065
	24	17.4	0.034
	30	16.3	0.034
	36	16.7	0.034
0.595	42	16.7	0.034
	48	17	0.034
	54	17.29	0.065
	60	19.6	0.065
	65	23.7	0.065

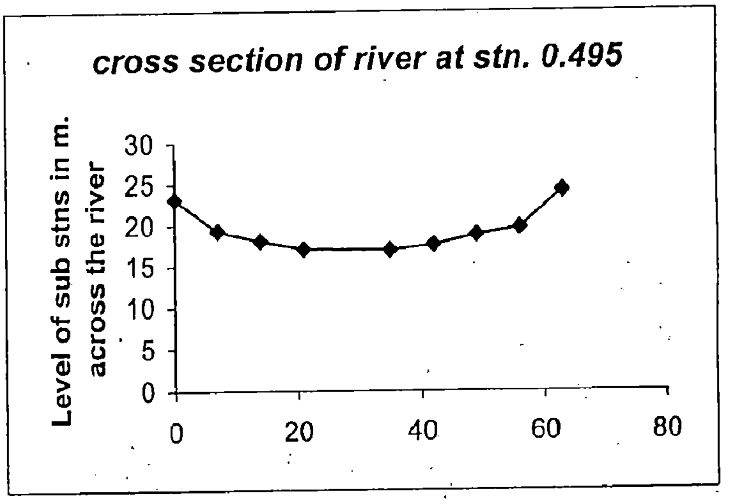


	0	24.96	0.065
	6	18.6	0.065
	12	17.059	0.034
	18	17.326	0.034
	24	17.084	0.034
	30	16.87	0.034
0.545	36	17	0.034
	42	17.3	0.065
	48	19.2	0.065
	60	24.137	0.065

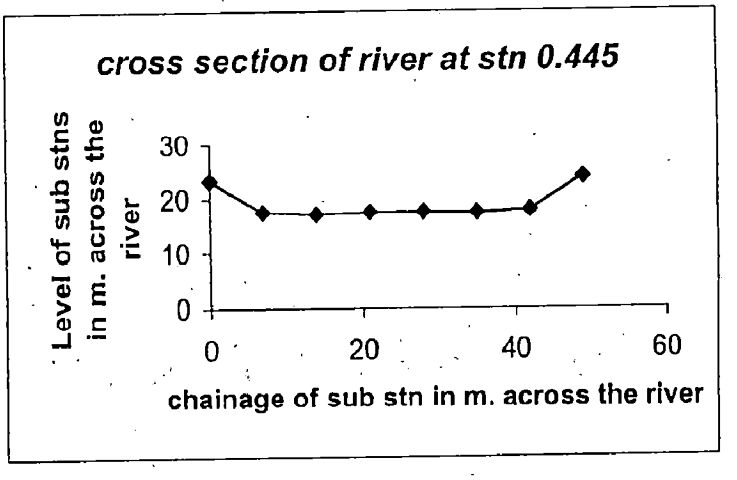


Contd-

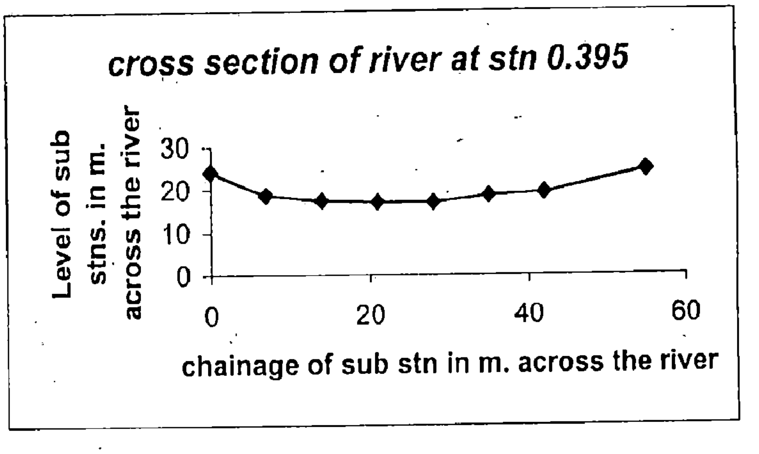
	0	23.105	0.065
	7	19.262	0.065
	14	18.017	0.065
	21	17.002	0.034
	35	16.87	0.034
0.495	42	17.526	0.065
	49	18.77	0.065
	56	19.6	0.065
	63	24.137	0.065



	0	23.283	0.065
	7	17.538	0.065
	14	17.17	0.034
	21	17.534	0.034
0.445	28	17.6	0.034
	35	17.396	0.034
	42	17.928	0.065
	49	23.94	0.065

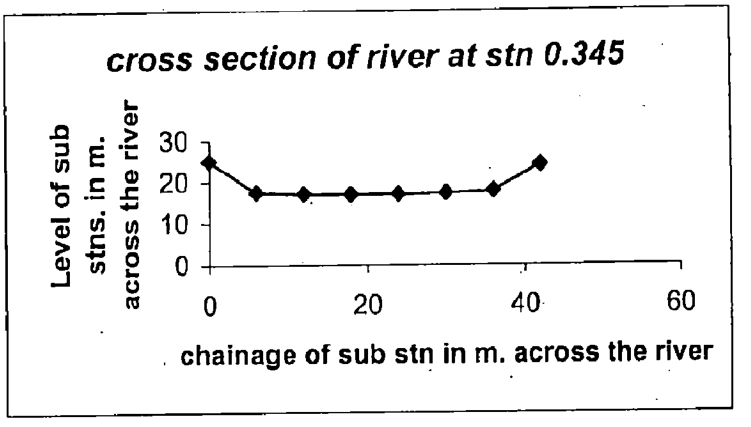


	0	24.137	0.065
	7	18.6	0.065
	14	17.316	0.034
	21	17	0.034
0.395	28	16.906	0.034
	35	18.59	0.065
	42	19.26	0.065
	55	24.416	0.065

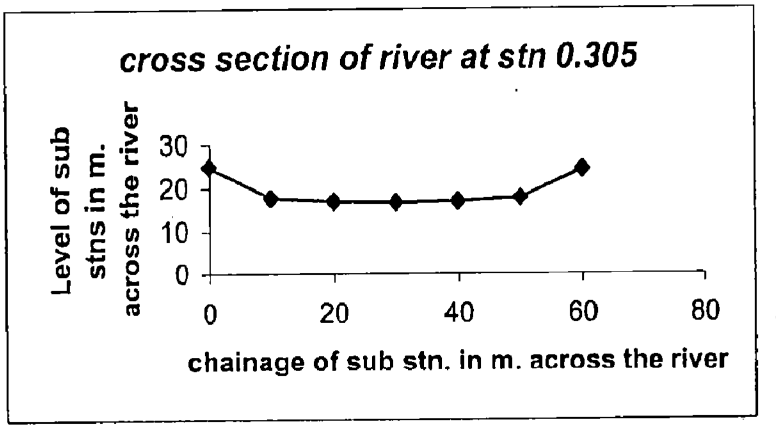


Contd-

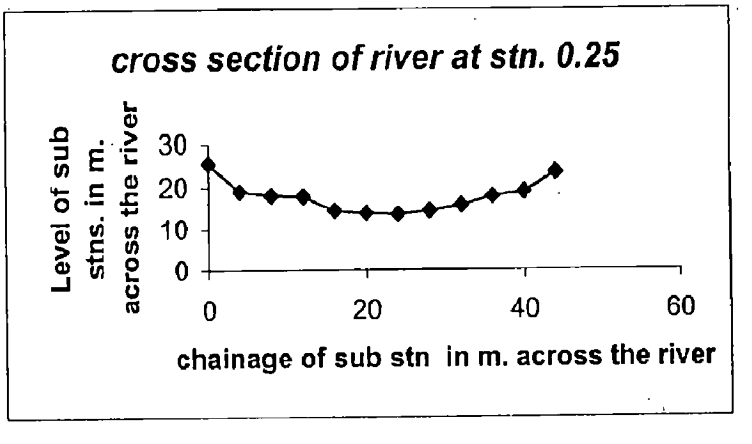
	0	24.9	0.065
	6	17.4	0.065
	12	16.95	0.034
	18	16.9	0.034
0.345	24	17	0.034
	30	17.2	0.034
	36	17.7	0.065
	42	23.9	0.065



	0	24.91	0.065
	10	17.7	0.065
	20	16.97	0.034
	30	16.72	0.034
0.305	40	17	0.065
	50	17.7	0.065
	60	24.3	0.065

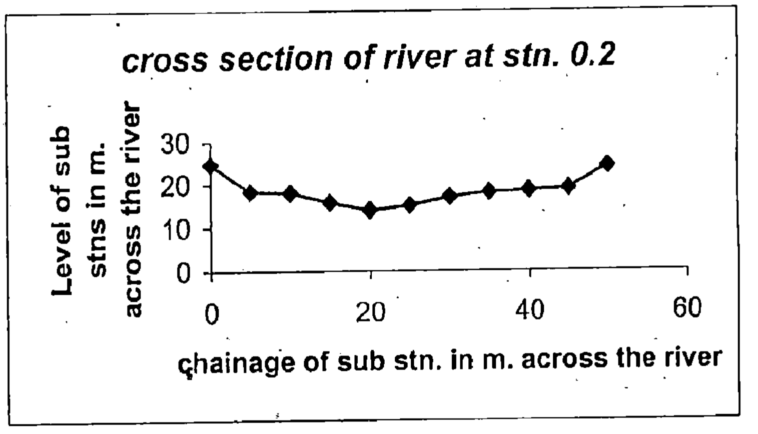


	0	25.5	0.065
	4	19	0.065
	8	18.1	0.065
	12	17.8	0.065
	16	14.39	0.034
	20	13.8	0.034
0.25	24	13.6	0.034
	28	14.39	0.034
	32	15.69	0.034
	36	17.84	0.065
	40	18.92	0.065
	44	23.35	0.065

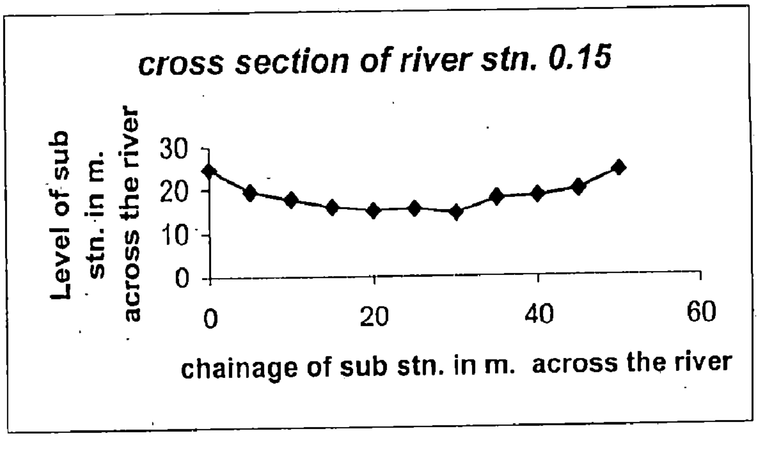


Contd-

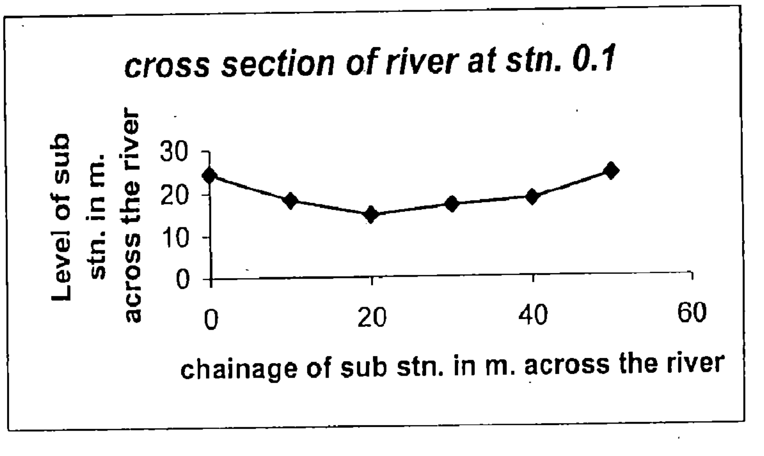
	0	24.7	0.115
	5	18.31	0.115
	10	18.01	0.115
	15	15.75	0.115
	20	13.96	0.048
0.2	25	15	0.048
	30	17	0.115
	35	18	0.115
	40	18.45	0.115
	45	18.9	0.115
	50	24.01	0.115



	0	24.639	0.115
	5	19.466	0.115
	10	17.715	0.115
	15	15.88	0.115
	20	15.081	0.048
	25	15.48	0.048
0.15	30	14.481	0.048
	35	17.88	0.115
	40	18.4	0.115
	45	19.79	0.115
	50	24.115	0.115

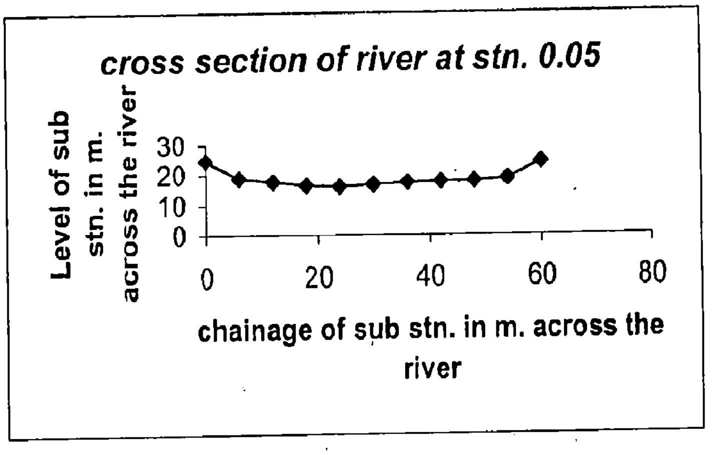


	0	24.466	0.115
	10	18.27	0.115
0.1	20	14.806	0.048
	30	17.002	0.115
	40	18.36	0.115
	50	24.02	0.115

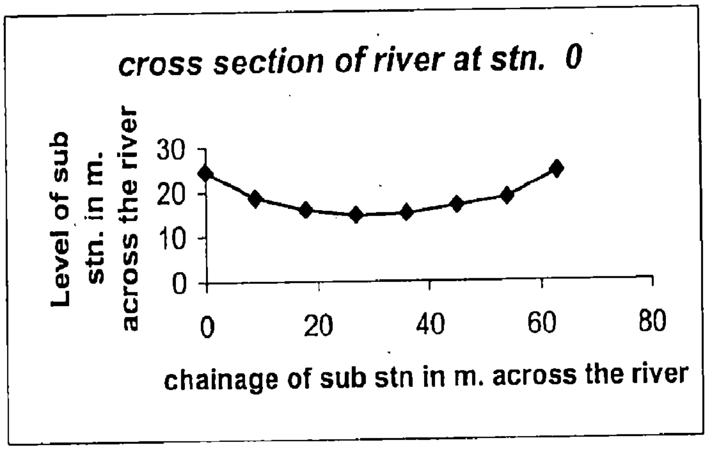


Contd-

	0	24.508	0.115
	6	18.718	0.115
	12	17.513	0.115
	18	16.308	0.048
	24	15.908	0.048
0.05	30	16.508	0.048
	36	17.108	0.115
	42	17.508	0.115
	48	17.608	0.115
	54	18.403	0.115
	60	23.89	0.115

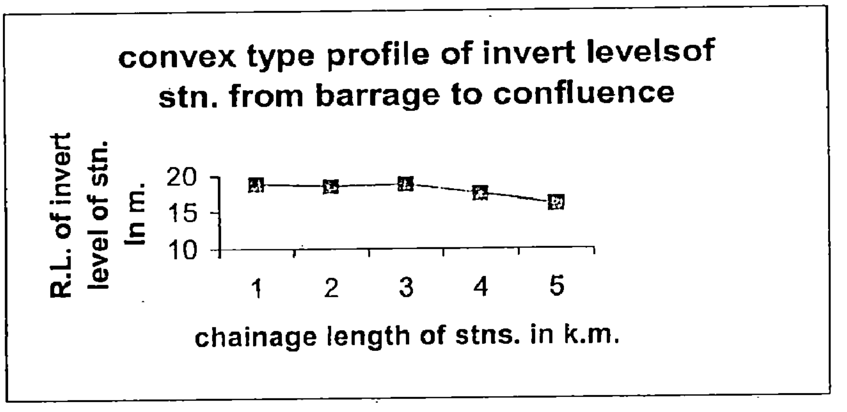


	0	24.572	0.115
	9	18.653	0.115
	18	16	0.115
0	27	14.72	0.048
	36	15.124	0.048
	45	16.926	0.115
	54	18.621	0.115
	63	24.525	0.115



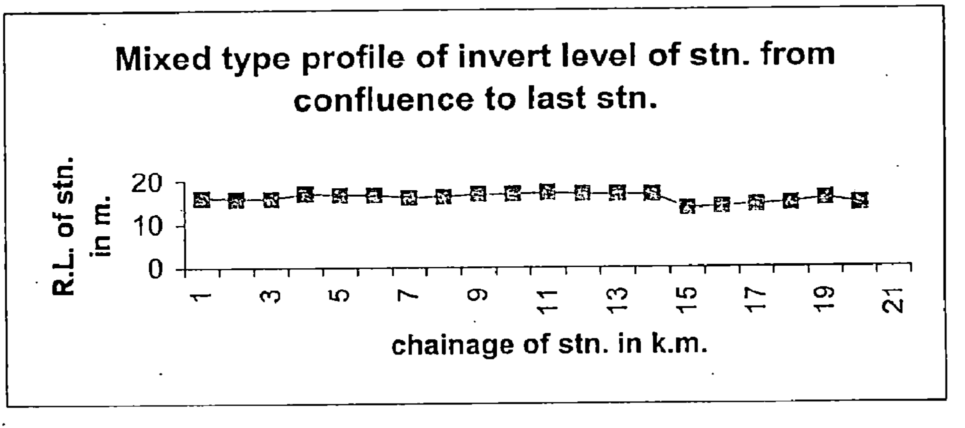
SHAPE OF LONGITUDINAL PROFILE OF RIVER FROM BARRAGE TO CONFLUENCE

1.32	18.818
1.255	18.508
1.155	18.75
1.055	17.486
0.955	16.129



SHAPE OF LONGITUDINAL PROFILE OF RIVER FROM CONFLUENCE TO LAST POINT

0.925	16.129
0.895	15.9
0.84	15.9
0.79	17
0.74	16.76
0.695	16.7
0.645	16.15
0.595	16.3
0.545	16.87
0.495	16.87
0.445	17.17
0.395	16.906
0.345	16.9
0.305	16.72
0.25	13.6
0.2	13.96
0.15	14.48
0.1	14.806
0.05	15.908
0	14.72



MODIFIED GEOMETRY

ALTERNATIVE 1: Keeping R.L. at 16.65m at station 0.79 and cutting the bed at a slope 0.00025 . Again fixing R.L at 16.30m at station 0.545 and cutting the bed width 12m at a slope 0.00025 up to 0.305

Modified cros section of river below the confluence

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.	ADOPTED MANNING 'N
0.79	0	24.000	0.118
	7	19.010	0.118
	14	18.930	0.118
	21	18.010	0.048
	23.071	17.889	0.024
	24	16.650	0.024
	36	16.650	0.024
	36.772	17.680	0.048
	42	17.000	0.048
	49	18.500	0.118
	56	24.000	0.118
0.74	0	23.960	0.118
	9	18.500	0.048
	18	17.800	0.048
	25.693	17.835	0.024
	27	16.453	0.024
	39	16.453	0.024
	39.389	16.972	0.048
	45	16.760	0.048
	54	18.900	0.118
	63	23.380	0.118
0.695	0	23.960	0.118
	8	18.370	0.118
	16	18.280	0.018
	24	17.400	0.048
	32	16.700	0.048
	33.559	16.856	0.024
	34	16.268	0.024
	46	16.268	0.024
	47.194	17.860	0.048
	48	17.900	0.118
	56	19.500	0.118
	64	19.700	0.118
	72	24.200	0.118
0.545	0	24.960	0.065
	6	18.600	0.034
	12	17.059	0.034
	18	17.326	0.034
	23.394	17.108	0.024
	24	16.300	0.024
	36	16.300	0.024
	36.545	17.027	0.034
	42	17.300	0.065
	48	19.200	0.065
	60	24.137	0.065

Contd...

0.495	0	23.050	0.065
	7	19.262	0.065
	14	18.017	0.034
	21	17.002	0.034
	23.388	16.979	0.024
	24	16.164	0.024
	36	16.164	0.024
	36.645	17.024	0.034
	42	17.526	0.065
	49	18.770	0.065
	56	19.600	0.065
	63	24.137	0.065
0.445	0	23.283	0.065
	7	17.538	0.034
	14	17.170	0.034
	17.997	17.378	0.024
	19	16.040	0.024
	31	16.040	0.024
	32.081	17.481	0.034
	35	17.396	0.034
	42	17.928	0.065
	49	23.940	0.065
0.395	0	24.137	0.065
	7	18.600	0.034
	14	17.316	0.034
	18.099	17.131	0.024
	19	15.929	0.024
	31	15.929	0.024
	32.555	18.002	0.034
	35	18.590	0.065
	42	19.260	0.065
	55	24.416	0.065
0.345	0	24.900	0.065
	6	17.400	0.034
	12	16.950	0.034
	13.167	16.940	0.024
	14	15.830	0.024
	26	15.830	0.024
	26.951	17.098	0.034
	30	17.200	0.034
	36	17.700	0.065
	42	23.900	0.065
0.305	0	24.910	0.065
	10	17.700	0.034
	20	16.970	0.034
	28.235	16.764	0.024
	29	15.744	0.024
	41	15.744	0.024
	42.05	17.143	0.034
	50	17.700	0.065
	60	24.300	0.065

RESULT : AT 200 CUMEC :R.L. 22.46

ALTERNATIVE-2
WIDENING THE CHANNEL AT A WIDTH OF 16M
FROM 0.79 TO 0.595 AND FROM 0.495 TO 0.395

MODIFIED CROSS SECTION OF RIVER BELOW THE CONFLUENCE

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.	ADOPTED MANNING'N
	0	24.000	0.118
	7	19.010	0.118
	14	18.930	0.118
	21	18.010	0.048
0.79	26.483	17.689	0.024
	27	17.000	0.024
	42	17.000	0.024
	43	17.000	0.024
	43.191	17.255	0.048
	49	18.500	0.118
	56	24.000	0.118
	0	23.960	0.118
	9	18.500	0.048
	18	17.800	0.048
	27	17.840	0.048
0.74	27.187	17.825	0.024
	28	16.740	0.024
	44	16.740	0.024
	44.042	16.796	0.048
	45	16.760	0.048
	54	18.900	0.118
	63	23.380	0.118
	0	23.960	0.118
	8	18.370	0.118
	16	18.280	0.048
	24	17.400	0.048
	32	16.700	0.048
0.695	32.4	16.740	0.024
	48	16.740	0.024
	49.024	18.105	0.118
	56	19.500	0.118
	64	19.700	0.118
	72	24.200	0.118
	0	23.000	0.118
	7	18.000	0.048
	14	17.700	0.048
	21	17.300	0.048
0.645	26.841	16.466	0.024
	27	16.254	0.024
	30.147	16.254	0.024
	35	16.150	0.024
	35.633	16.254	0.024
	43	16.254	0.024
	44.048	17.651	0.048
	49	18.500	0.118
	60	24.216	0.118

Contd...

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.	ADOPTED MANNING 'N
0.595	0	24.200	0.065
	6	21.700	0.065
	12	19.600	0.065
	18	18.580	0.034
	24	17.400	0.034
	29.96	16.307	0.024
	30	16.254	0.024
	46	16.254	0.024
	46.503	16.925	0.034
	48	17.000	0.034
	54	17.290	0.065
	60	19.600	0.065
	65	23.700	0.065
0.495	0	23.105	0.065
	7	19.262	0.065
	14	18.017	0.034
	21	17.002	0.034
	35	16.870	0.034
	36.28	16.990	0.024
	51	16.990	0.024
	52.661	19.204	0.065
	56	19.600	0.065
	63	24.137	0.065
0.445	0	23.283	0.065
	7	17.538	0.034
	14	17.170	0.034
	16.469	17.298	0.024
	16.7	16.990	0.024
	32.7	16.990	0.024
	33.047	17.453	0.034
	35	17.396	0.034
	42	17.928	0.065
	49	23.940	0.065
0.395	0	24.137	0.065
	7	18.600	0.034
	14	17.316	0.034
	21	17.000	0.034
	28	16.906	0.034
	28.349	16.990	0.034
	44	16.990	0.024
	47.27	21.350	0.065
	55	24.416	0.065

RESULT: AT 200 CUMEC 22.38

MODIFIED GEOMETRY

ALTERNATIVE: 3 :

Deepening the B.L. to 16.30 from stn. 0.79to 0.695 , from 0.545 to 0.305 with a width of 20m.

Modified cros section of river below the confluence

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.	ADOPTED MANNING"N
	0	24.000	0.118
	7	19.010	0.048
	14	18.930	0.048
	21	18.010	0.048
0.79	26.98	17.660	0.048
	28	16.300	0.024
	48	16.300	0.024
	50.583	19.743	0.118
	56	24.000	0.118
	0	23.960	0.118
	9	18.500	0.048
	18	17.800	0.048
	25.693	17.839	0.048
0.74	28	16.300	0.024
	48	16.300	0.024
	49.071	17.728	0.048
	54	18.900	0.118
	63	23.380	0.118
	0	23.960	0.118
	8	18.370	0.048
	16	18.280	0.048
	24	17.400	0.048
	24.187	17.384	0.024
0.695	25	16.300	0.024
	45	16.300	0.024
	46.13	17.806	0.048
	48	17.900	0.118
	56	19.500	0.118
	64	19.700	0.118
	72	24.200	0.118
	0	23.000	0.118
	7	18.000	0.118
	14	17.700	0.048
	21	17.300	0.048
0.645	28	16.300	0.048
	35	16.150	0.048
	42	17.300	0.048
	49	18.500	0.118
	60	24.216	0.118
	0	24.200	0.065
	6	21.700	0.065
	12	19.600	0.065
	18	18.580	0.065
	24	17.400	0.034
0.595	30	16.300	0.034
	36	16.700	0.034
	42	16.700	0.034
	48	17.000	0.034
	54	17.290	0.065
	60	19.600	0.065
	65	23.700	0.065

Contd...

0.545	0	24.960	0.065
	6	18.600	0.034
	12	17.059	0.034
	18	17.326	0.034
	19.269	17.275	0.024
	20	16.300	0.024
	40	16.300	0.024
	40.701	17.235	0.034
	42	17.300	0.065
	48	19.200	0.065
	60	24.137	0.065
0.495	0	23.105	0.065
	7	19.262	0.065
	14	18.017	0.034
	19.287	17.250	0.024
	20	16.300	0.024
	40	16.300	0.024
	40.838	17.417	0.034
	42	17.526	0.065
	49	18.770	0.065
	56	19.600	0.065
	63	24.137	0.065
0.445	0	23.283	0.065
	7	17.538	0.034
	9.157	17.425	0.024
	10	16.300	0.024
	30	16.300	0.024
	30.911	17.515	0.034
	35	17.396	0.034
	42	17.928	0.065
	49	23.940	0.065
0.395	0	24.137	0.065
	7	18.600	0.034
	14	17.316	0.034
	14.246	17.305	0.024
	15	16.300	0.024
	35	16.300	0.024
	36.85	18.767	0.065
	42	19.260	0.065
	55	24.416	0.065
0.345	0	24.900	0.065
	6	17.400	0.034
	9.364	17.148	0.024
	10	16.300	0.024
	30	16.300	0.024
	30.72	17.260	0.034
	36	17.700	0.065
	42	23.900	0.065
0.305	0	24.910	0.065
	10	17.700	0.034
	19.468	16.970	0.034
	20	16.764	0.024
	40	15.744	0.024
	40.554	15.744	0.024
	50	17.143	0.034
	60	17.700	0.065

RESULT : AT 200 CUMEC :R.L. 22.17

Appendix B-4

ALTERNATIVE: 4

Deepening the B.L. to 16.4 from stn. 0.79to 0.695 , from 0.545 to 0.445 from 0.445 to 0.345 & from 0.345to 0.305 width a width of 14m.

Modified cros section of river below the confluence

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.	ADOPTED MANNING"N
0.79	0	24.00	0.118
	7	19.01	0.048
	14	18.93	0.048
	21	18.01	0.048
	28	17.60	0.048
	30.032	17.69	0.024
	31	16.40	0.024
	45	16.40	0.024
	46.11	17.88	0.048
	49	18.50	0.118
	56	24.00	0.118
0.74	0	23.96	0.118
	9	18.50	0.048
	18	17.80	0.048
	27	17.84	0.048
	30.112	17.58	0.024
	31	16.40	0.024
	45	16.40	0.024
	45.329	16.84	0.048
	54	18.90	0.118
	63	23.38	0.118
0.695	0	23.96	0.118
	8	18.37	0.048
	16	18.28	0.048
	24	17.40	0.048
	27.478	17.10	0.024
	28	16.40	0.024
	42	16.40	0.024
	42.935	17.65	0.048
	48	17.90	0.118
	56	19.50	0.118
	64	19.70	0.118
	72	24.20	0.118
0.645	0	23.00	0.118
	7	18.00	0.118
	14	17.70	0.048
	21	17.30	0.048
	28	16.30	0.048
	35	16.15	0.048
	42	17.30	0.048
	49	18.50	0.118
	60	24.22	0.118
0.595	0	24.20	0.065
	6	21.70	0.065
	12	19.60	0.065
	18	18.58	0.065
	24	17.40	0.034
	30	16.30	0.034
	36	16.70	0.034
	42	16.70	0.034
	48	17.00	0.034
	54	17.29	0.065
	60	19.60	0.065
	65	23.70	0.065

Contd...

0.545	0	24.96	0.065
	6	18.60	0.034
	12	17.06	0.034
	18	17.33	0.034
	22.44	17.15	0.024
	23	16.40	0.024
	37	16.40	0.024
	37.506	17.08	0.034
	42	17.30	0.065
	48	19.20	0.065
	60	24.14	0.065
0.495	0	23.11	0.065
	7	19.26	0.065
	14	18.02	0.034
	21	17.00	0.034
	22.56	16.99	0.024
	23	16.40	0.024
	37	16.40	0.024
	37.53	17.11	0.034
	42	17.53	0.065
	49	18.77	0.065
	56	19.60	0.065
	63	24.14	0.065
0.445	0	23.28	0.065
	7	17.54	0.034
	12.358	17.26	0.024
	13	16.40	0.024
	27	16.40	0.024
	27.899	17.60	0.034
	28	17.60	0.034
	35	17.40	0.034
	42	17.93	0.065
	49	23.94	0.065
0.395	0	24.14	0.065
	7	18.60	0.034
	14	17.32	0.034
	17.429	17.16	0.024
	18	16.40	0.024
	32	16.40	0.024
	33.344	18.19	0.034
	35	18.59	0.065
	42	19.26	0.065
	55	24.42	0.065
0.345	0	24.90	0.065
	6	17.40	0.034
	12	16.95	0.034
	12.591	16.95	0.024
	13	16.40	0.024
	27	16.40	0.024
	27.538	17.12	0.034
	30	17.20	0.034
	36	17.70	0.065
	42	23.90	0.065
0.305	0	24.91	0.065
	10	17.70	0.034
	20	16.97	0.034
	22.622	16.90	0.024
	23	16.40	0.024
	37	16.40	0.024
	37.395	16.93	0.034
	40	17.00	0.034
	50	17.70	0.065
	60	24.00	0.065

RESULT : AT 200 CUMEC :R.L. 22.46

MODIFIED GEOMETRY

Alternative 5:

Appendix B-5

Deepening the B.L. to 16.40 from station 0.79to 0.695, from 0.545 to 0.445 , from 0.445 to 0.345 and from 0.345 to 0.305 with a width of 8m.

Modified cros section of river below the confluence

STATION NO	REACH LENGTH IN M	REDUCED LEVEL IN M	ADOPTED MANNING'N
0.79	0	24.000	0.118
	7	19.010	0.118
	14	18.930	0.118
	21	18.010	0.048
	28	17.660	0.048
	32.936	17.818	0.024
	34	16.400	0.024
	42	16.400	0.024
	42.536	17.115	0.048
	49	18.500	0.118
	56	24.000	0.118
0.74	0	23.960	0.118
	9	18.500	0.118
	18	17.800	0.048
	27	17.839	0.048
	33.309	17.321	0.024
	34	16.400	0.024
	42	16.400	0.024
	42.345	16.860	0.048
	45	16.760	0.048
	54	18.900	0.118
	63	23.380	0.118
0.695	0	23.960	0.118
	8	18.370	0.118
	16	18.280	0.018
	24	17.400	0.048
	30.689	16.815	0.024
	31	16.400	0.024
	39	16.400	0.024
	39.811	17.481	0.048
	40	17.500	0.048
	48	17.900	0.118
	56	19.500	0.118
	64	19.700	0.118
	72	24.200	0.118
0.645	0	23.000	0.118
	7	18.000	0.118
	14	17.700	0.048
	21	17.300	0.048
	28	16.300	0.048
	35	16.150	0.048
	42	17.300	0.048
	49	18.500	0.118
	60	24.216	0.118

Contd...

0.595	0	24.200	0.065
	6	21.700	0.065
	12	19.600	0.065
	18	18.580	0.065
	24	17.400	0.034
	30	16.300	0.034
	36	16.700	0.034
	42	16.700	0.034
	48	17.000	0.034
	54	17.290	0.065
	60	19.600	0.065
	65	23.700	0.065
0.545	0	24.960	0.065
	6	18.600	0.065
	12	17.059	0.034
	18	17.326	0.034
	24	17.084	0.034
	25.528	17.030	0.024
	26	16.400	0.024
	34	16.400	0.024
	34.424	16.966	0.034
	36	17.000	0.034
	42	17.300	0.065
	48	19.200	0.065
	60	24.137	0.065
0.495	0	23.105	0.065
	7	19.262	0.065
	14	18.017	0.065
	21	17.002	0.034
	25.581	16.959	0.024
	26	16.400	0.024
	34	16.400	0.024
	34.357	16.876	0.034
	35	16.870	0.034
	42	17.526	0.065
	49	18.770	0.065
	56	19.600	0.065
	63	24.137	0.065
0.445	0	23.283	0.065
	7	17.538	0.034
	14	17.170	0.034
	15.369	17.241	0.024
	16	16.400	0.024
	24	16.400	0.024
	24.878	17.571	0.034
	28	17.600	0.034
	35	17.396	0.034
	42	17.928	0.065
	49	23.940	0.065

0.395	0	24.137	0.065
	7	18.600	0.065
	14	17.316	0.034
	20.534	17.021	0.024
	21	16.400	0.024
	29	16.400	0.024
	29.683	17.311	0.034
	35	18.590	0.065
	42	19.260	0.065
	55	24.416	0.065
0.345	0	24.900	0.065
	6	17.400	0.034
	12	16.950	0.034
	15.61	16.920	0.024
	16	16.400	0.024
	24	16.400	0.024
	24.462	17.015	0.034
	30	17.200	0.034
	36	17.700	0.065
	42	23.900	0.065
0.305	0	24.910	0.065
	10	17.700	0.034
	20	16.970	0.034
	25.679	16.828	0.024
	26	16.400	0.024
	34	16.400	0.024
	34.331	16.841	0.034
	40	17.000	0.034
	50	17.700	0.065
	60	24.300	0.065

RESULT : AT 200 CUMEC :R.L. 22.63

Deepening the B.L. to 16.40 from station 0.79to 0.695, from 0.545 to 0.445 , from 0.445 to 0.345 and from 0.345 to 0.305 with a width of 10m.

Modified cross section of the river below the confluence

STATION NO	REACH LENGTH IN M	REDUCED LEVEL IN M	ADOPTED MANNING'N
0.79	0	24.000	0.118
	7	19.010	0.118
	14	18.930	0.118
	21	18.010	0.048
	28	17.660	0.048
	31.968	17.776	0.024
	33	16.400	0.024
	43	16.400	0.024
	43.728	17.370	0.048
	49	18.500	0.118
	56	24.000	0.118
0.74	0	23.960	0.118
	9	18.500	0.118
	18	17.800	0.048
	27	17.839	0.048
	32.243	17.409	0.024
	33	16.400	0.024
	43	16.400	0.024
	43.318	16.824	0.048
	45	16.760	0.048
	54	18.900	0.118
	63	23.380	0.118
0.695	0	23.960	0.118
	8	18.370	0.118
	16	18.280	0.018
	24	17.400	0.048
	29.619	16.908	0.024
	30	16.400	0.024
	40	16.400	0.024
	40.857	17.543	0.048
	48	17.900	0.048
	56	19.500	0.118
	64	19.700	0.118
	72	24.200	0.118
0.645	0	23.000	0.118
	7	18.000	0.118
	14	17.700	0.048
	21	17.300	0.048
	28	16.300	0.048
	35	16.150	0.048
	42	17.300	0.048
	49	18.500	0.118
	60	24.216	0.118

Contd....

0.595	0	24.200	0.065
	6	21.700	0.065
	12	19.600	0.065
	18	18.580	0.065
	24	17.400	0.034
	30	16.300	0.034
	36	16.700	0.034
	42	16.700	0.034
	48	17.000	0.034
	54	17.290	0.065
	60	19.600	0.065
	65	23.700	0.065
0.545	0	24.960	0.065
	6	18.600	0.065
	12	17.059	0.034
	18	17.326	0.034
	24	17.084	0.034
	24.5	17.066	0.024
	25	16.400	0.024
	35	16.400	0.024
	35.441	16.988	0.034
	36	17.000	0.034
	42	17.300	0.065
	48	19.200	0.065
	60	24.137	0.065
0.495	0	23.105	0.065
	7	19.262	0.065
	14	18.017	0.065
	21	17.002	0.034
	24.574	16.968	0.024
	25	16.400	0.024
	35	16.400	0.024
	35.379	16.906	0.034
	42	17.526	0.034
	49	18.770	0.065
	56	19.600	0.065
	63	24.137	0.065
0.445	0	23.283	0.065
	7	17.538	0.065
	14	17.170	0.034
	14.407	17.191	0.024
	15	16.400	0.024
	25	16.400	0.024
	25.885	17.571	0.034
	28	17.600	0.034
	35	17.396	0.034
	42	17.928	0.065
	49	23.940	0.065

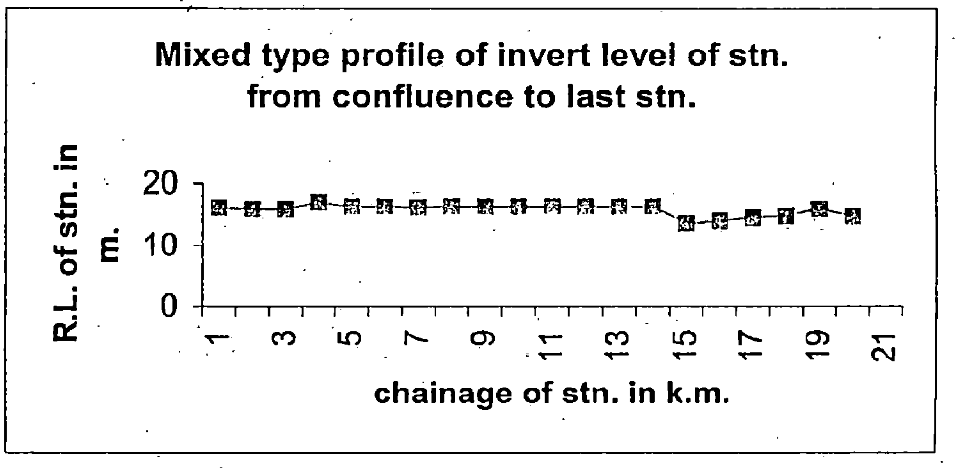
Contd...

0.395	0	24.137	0.065
	7	18.600	0.065
	14	17.316	0.034
	19.499	17.021	0.024
	20	16.400	0.024
	30	16.400	0.024
	30.903	17.604	0.034
	35	18.590	0.065
	42	19.260	0.065
	55	24.416	0.065
0.345	0	24.900	0.065
	6	17.400	0.034
	12	16.950	0.034
	14.604	16.400	0.024
	15	16.400	0.024
	25	17.050	0.034
	25.487	17.050	0.034
	30	17.200	0.034
	36	17.700	0.065
	42	23.900	0.065
0.305	0	24.910	0.065
	10	17.700	0.034
	20	16.970	0.034
	24.66	16.854	0.034
	25	16.400	0.024
	35	16.400	0.024
	35.352	16.870	0.034
	40	17.000	0.034
	50	17.700	0.065
	60	24.300	0.065

Result: AT 200 cumec ; R.L.=22.59m.

SHAPE OF LONGITUDINAL PROFILE OF RIVER FROM CONFLUENCE TO LAST POINT AFTER THE MODIFICATION WITH ALTERNATIVE :3

0.925	16.129
0.895	15.9
0.84	15.9
0.79	17
0.74	16.3
0.695	16.3
0.645	16.15
0.595	16.3
0.545	16.3
0.495	16.3
0.445	16.3
0.395	16.3
0.345	16.3
0.305	16.3
0.25	13.6
0.2	13.96
0.15	14.48
0.1	14.806
0.05	15.908
0	14.72



Appendix C

CROSS SECTION DATA COLLECTED FROM MAP FOR REACH FROM BARRAGE TO CONFLUENCE POINT

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.
1.32	0	25.090
	10	20.220
	20	19.646
	30	18.818
	40	19.410
	50	19.650
	60	20.270
	70	22.650
	80	24.216
1.255	0	26.028
	10	21.119
	20	19.434
	30	18.698
	40	18.508
	50	19.313
	60	21.746
	70	24.110
1.155	0	25.908
	10	20.900
	20	19.011
	30	19.098
	40	18.750
	50	19.026
	60	20.670
	70	29.420
1.055	0	26.037
	10	20.775
	20	18.770
	30	17.486
	40	18.016
	50	19.798
	60	20.319
	70	20.217
	80	26.037
0.955	0	27.196
	7	21.776
	14	21.488
	21	20.650
	28	18.560
	35	16.729
	42	16.129
	49	16.729
	56	16.729
	63	17.865
	70	20.260
	77	26.000

Contd...

CROS SECTION DETAIL OF REACH FROM CONFLUENCE POINT TO CONTROL POINT

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.
0.925	0	27.196
	7	21.776
	14	21.488
	21	20.650
	28	18.560
	35	16.729
	42	16.129
	49	16.729
	56	16.729
	63	17.865
	70	20.260
	77	26.000
0.895	0	27.200
	7	22.300
	14	20.400
	21	18.400
	28	16.310
	35	15.900
	42	16.500
	49	17.000
	63	18.100
	72	26.000
0.84	0	25.500
	10	20.400
	20	16.700
	30	15.900
	40	16.600
	50	19.600
	60	24.500
0.79	0	24.000
	7	19.010
	14	18.930
	21	18.010
	28	17.600
	35	17.910
	42	17.000
	49	18.500
	56	24.000
0.74	0	23.960
	9	18.500
	18	17.800
	27	17.840
	36	17.100
	45	16.760
	54	18.900
	63	23.380

Contd...

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.
0.695	0	23.960
	8	18.370
	16	18.280
	24	17.400
	32	16.700
	40	17.500
	48	17.900
	56	19.500
	64	19.700
	72	24.200
0.645	0	23.000
	7	18.000
	14	17.700
	21	17.300
	28	16.300
	35	16.150
	42	17.300
	49	18.500
	60	24.216
0.595	0	24.200
	6	21.700
	12	19.600
	18	18.580
	24	17.400
	30	16.300
	36	16.700
	42	16.700
	48	17.000
	54	17.290
	60	19.600
	65	23.700
0.545	0	24.960
	6	18.600
	12	17.059
	18	17..326
	24	17.084
	30	16.870
	36	17.000
	42	17.300
	48	19.200
	60	24.137
0.495	0	23.105
	7	19.262
	14	18.017
	21	17.002
	35	16.870
	42	17.526
	49	18.770
	56	19.600
	63	24.137

Contd...

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.
0.445	0	23.283
	7	17.538
	14	17.170
	21	17.534
	28	17.600
	35	17.396
	42	17.928
	49	23.940
0.395	0	24.137
	7	18.600
	14	17.316
	21	17.000
	28	16.906
	35	18.590
	42	19.260
	55	24.416
0.345	0	24.900
	6	17.400
	12	16.950
	18	16.900
	24	17.000
	30	17.200
	36	17.700
	42	23.900
0.305	0	24.910
	10	17.700
	20	16.970
	30	16.720
	40	17.000
	50	17.700
	60	24.300
0.25	0	25.500
	4	19.000
	8	18.100
	12	17.800
	16	14.390
	20	13.800
	24	13.600
	28	14.390
	32	15.690
	36	17.840
	40	18.920
	44	23.350
0.2	0	24.700
	5	18.310
	10	18.010
	15	15.750
	20	13.960
	25	15.000
	30	17.000
	35	18.000
	40	18.450
	45	18.900
	50	24.100

Contd...

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.
0.15	0	24.639
	5	19.466
	10	17.715
	15	15.880
	20	15.081
	25	15.480
	30	14.481
	35	17.880
	40	18.400
	45	19.790
	50	24.115
0.1	0	24.466
	10	18.270
	20	14.806
	30	17.002
	40	18.360
	50	24.020
0.05	0	24.508
	6	18.718
	12	17.513
	18	16.308
	24	15.908
	30	16.508
	36	17.108
	42	17.508
	48	17.608
	54	18.403
	60	23.890
0	0	24.572
	9	18.653
	18	16.000
	27	14.720
	36	15.124
	45	16.926
	54	18.621
	63	24.525

CROSS SECTION OF TAIL RACE CHANNEL

STATION NO	REACH LENGTH IN M.	REDUCED LEVEL IN M.
0.045	0	27.196
	5	16.429
	30	16.429
	35	27.196
0.03	0	27.196
	5	16.219
	30	16.219
	35	27.196
0	0	27.196
	5	16.129
	30	16.129
	35	27.196