CS(AR)-16/96-97

APPLICATION OF HEC-2 PROGRAMME FOR WATER SURFACE PROFILE DETERMINATION OF RIVER DIGARU AT SONAPUR, ASSAM

NATIONAL INSTITUTE OF HYDROLOGY
JAL VIGYAN BHAWAN
ROORKEE - 247 667

CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

Flow regime of any river for different discharge condition is essential to the water resources engineers, planners and managers. To know the depth of flow at different downstream reaches water surface profiles are computed using the information on the channel geometry and roughness coefficient for the river reach. Water surface profiles can be calculated by different methods. The most common and simple method is to calculate the water surface profile by standard step method in which profile is calculated reach by reach. The programme HEC-2 calculates water surface profiles for gradually varied flow by standard step method. It computes the water surface elevations and related hydraulic parameters at all locations of interests for given flow values. The data needed to perform these computations include flow regime, starting elevations, discharge, loss coefficients, cross section geometry, reach lengths etc. The effect of obstruction such as bridges, weirs and other structures in flood plain can be studied. In a single run, upto 14 profiles using the same cross section data can be computed. The programme can take into account the reduction of the flow carrying capacity due to encroachment of flood plain. The computation of water surface profiles in the tributary system, if necessary can, be made after the computation of the main stream. The Manning' "n" values for flood plain and channel can be varied.

In this report the programme HEC-2 has been implemented on river Digaru to compute water surface profile for several discharge conditions. In this reach, two bridges have been considered. The computed water surface elevations, critical water surface elevations, energy grade elevations and slope, velocity in the channel, flow area, top width and Froude Number in the reach are hereby reported in this report.

1.0 INTRODUCTION

There are several reasons for the floods to occur like heavy rain in the upstream catchment, landslide, dam break, cloud break etc. Whatever may be the cause of flood, its movement generally follows a similar pattern as it moves down. At a given site, the water level rises to reach a peak and then slowly decreases. As the water level increases, discharge of water also increases, together with an increase in the storage of water in the channel and adjoining plains. Hence a flow entering a reach may pass through the reach or may go into temporary storage. From this storage, it will be released slowly. It is necessary that the continuity equation used in flood routing accounts such balance between inflow and out flow. In this process of flow, the peak discharge entering a reach gets lowered before leaving the reach. This acts a natural regulator of flood, moderating it. Such decrease in flow is known as attenuation. This occurs in all channels, although at varying extent. These process have been explained from the figure 1a. to 1.d

The figure I.a depicts the observed hydrograph at both the ends of a reach. The difference between inflow and outflow goes into storage until the time of maximum storage T as marked S. After this time T the outflow exceeds the inflow until both are equal at H for a single flood peak. During this period flood water is released from the temporary storage. In general S should be equal to R but it may be noted that depending on the topographic nature, some part of S may escape the river system. This kind of escape is not treated in this report. The attenuation in the term of inflow can be expressed as

$$Q_{Att} = 100\ (P_I - P_O)/P_I$$

The figure I.b explains how flood water enters into storage in the longitudinal point of view. The figure I.d explains the plan view of flood movement.

Expect for prismatic channel, the superimposed cross sections at various sites of a reach would look very similar to figure I.c. This figure is only a typical situation. Number of tributaries may join the river and the discharge increase4s as one moves downstream unless large withdrawals are made in between.

Any obstruction in the waterway causes backwater effect which may even extend up to 60 km upstream and water spread area may get increased. Floodplains provide ample space for storage of flood water.

The velocity of flow in the floodplains are much different from that of channel. This is due to the fact that roughness present in floodplain is much larger and the depth of flow in the plains is shallower than that of channel. Though these characteristics complicate the flood routing, HEC-2 program handle these cases very efficiently.

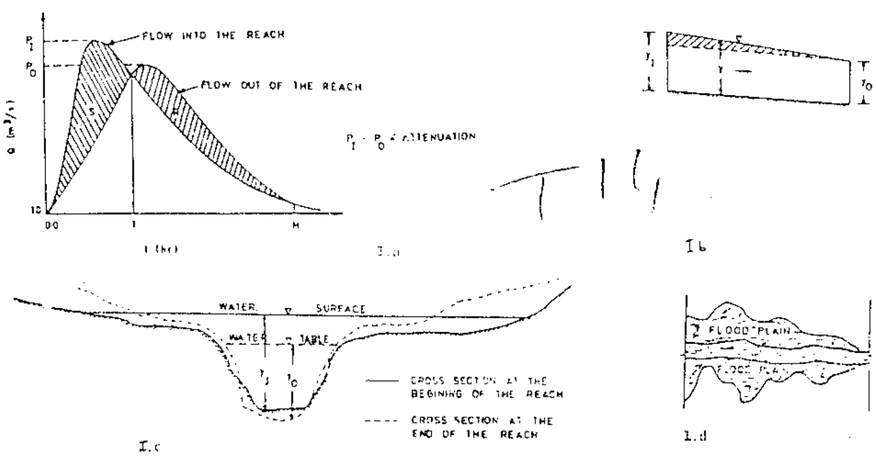


Figure I.a - Flood hydrographs at both the ends of a river reach
I.b - Idealised water surface profile explaining the storage effect
I.c - Water level marked on a typical superimposed cross-section.
___ Cross-section at the beginning of the reach ; ---- cross-section at the end of the reach
I.d - Plan view of flood plain inundation

Figure I : Schematic View of flood hydrograph

1.1 IMPORTANCE OF THE STUDY

The flood plain is the area adjoining the main river channel which gets inundated when the water level increases beyond the bank-full storage. The movement of flood along the flood plains is quiet complex. The flood plain acts as conveying as well as storage medium for passage of floods. The valley storage naturally regulates the flood flow as it moves towards the downstream end of the reach. The flow over the flood plains is relatively slower than the main channel. This is mainly due to relatively low depth of flow and also due to the increased frictional resistance.

During flood season, when the stream is in spate (sudden rush of water), the water level increases. the flow overtops the river channel banks and moves into the flood plains. In case of wide floodplains, the flood water gets stored and cause large attenuation. Over the flood-plain the flood moves in relatively shallow depths. The vegetation and bed roughness of the plains retard the flow and as such energy loss is different in the flood plains than that in the channel.

Flow regime of any river for different discharge condition is essential to the water resources engineers, planners and managers. The river training works modify the existing flow situation. For example, construction of levees for the protection of a village or town increases the depth of flood flow. The human activities and encroachments in the flood plain affects the river environment and flood flow. Embankments are constructed in order to avoid disastrous inundations. But such activities drastically change the river regime, which has to attain new equilibrium under changed conditions. Construction of any obstruction across a river like dams, weirs, barrages bridges etc. change the flow regime. For releasing water from any reservoir, knowledge of depth of flow at different downstream reach is essential for avoiding damage of lives and properties due to inundation. To know the depth of flow at different downstream reaches water surface profiles are computed using the information on the channel geometry and roughness coefficient for the river reach of the region.

1.2 LITERATURE REVIEW

The method of determining the flood hydrograph at a section, using the data on the same at an upstream site is known as flood routing. These methods are needed for many hydrologic applications like flood forecasting, flood protection, reservoir design, spillway design etc.

There have been a variety of flood routing procedure which have been developed involving various assumptions about the flow and channel characteristics depending upon available data.

Theoretical foundation for flood routing was laid by Saint-Venant in 1871, with the development of one dimensional equation of unsteady flow. The St. Venant equation express the conservation of mass and momentum as follows :

Continuity :

$$\partial Q/\partial x + \partial A/\partial t = 0 \qquad \text{...... (I.1)}$$

Momentum

$$\frac{\partial A}{\partial t} + \left(\frac{Q}{A}\right)\frac{\partial Q}{\partial x} + Ag\left(\frac{\partial Y}{\partial x} - S_o + S_f\right) = 0 \qquad \text{...... (I.2)}$$

where,

Q is the discharge (m^3/s),

A is the area of cross section of flow (m^2),

Y is the depth of flow (m),

g is the acceleration due to gravity (m/s^2)

S_o is the bed slope,

S_f is the energy slope,

x,t are space and time coordinates.

These equations are too complex to solve and hence various simplified forms are derived to solve flood propagation. A bibliography of the works carried out on flood routing is available in Miller, et al (1975). The solution procedures for the flood propagation can be classified under two categories viz (i) hydrologic flood routing, (ii) hydraulic flood routing.

Hydraulic routing considers the flow to be gradually varied unsteady and solves the partial differential equations. Hydrologic routing aims at computationally simplified approach by using routing parameters. These parameters are chosen so that they are able to reproduce the outflow hydrograph. Two distinct effects, on the hydrograph of these parameters are (i) the translation of flood wave without change of form and (ii) the attenuation, which is a reduction in flood peak.

1.3 AVAILABLE METHODOLOGY AND ADVANTAGE OF HEC-2 OVER THEM

Hydrologic routing methods use simplified forms of equations 1 and 2. The continuity equation takes the form :

$$I - O = ds/dt \qquad \text{...... (I.3)}$$

Instead of the usual momentum equation given earlier, hydrologic flood routing methods adopt storage equation as follows :

$$S = k(x\ I - (1-x)O) \qquad \text{...... (I.4)}$$

where

I is an average inflow over time dt

O is an average outflow over time dt

S is storage

k, x are routing parameters determined from flood records.

The Muskingam method uses a trial and error method to arrive at the two parameters. There are other methods evolved to determine physically meaningful parameters.

The Muskingam Cunge Procedure is one of such procedure which considers the attenuation effect of the flow during its passage through a river reach due to irregularities in the cross section and storage effect. The Muskingam procedure also considers the effect of channel geometry. Because of the use of information on the river geometry this method provides a scope of extending its application to flood larger than earlier observations.

In case of hydraulic flood routing the St. Venant equations are solved with appropriate boundary conditions by any of the following methods :

1. Finite Difference Method
2. Method of characteristics
3. Finite element methods.

A review of hydraulic routing method can be seen in Hydraulic Routing Techniques, NIH RN-21 (1985-86).

1.3.1 Flood Routing Models

Routing of floods can be achieved through black box models or through mathematical models. The black box models require data of a number of past high floods involving movement in flood plains. More the availability of data of such floods, better is the establishment of the routing parameters, assuming that channel processes have not significantly altered the river regime. The mathematical models require data on cross section of the river at close intervals and estimate of roughness coefficient. Their requirement on data of past flood is minimal. Based on data availability any one of the method can be chosen.

Black Box Models

This method makes use of the past recorded floods and finds the set of routine parameters that would best produce the out flow given the inflow. These parameters takes care of the flood and the geometry of the river. In case the floods, used in computing the parameters are large such that movement in the flood plains occurred, the parameters could conveniently be used to route the flood taking flood plains into account. Otherwise the routing parameters can not be used and Muskingam Method is of little use.

In such case, a variance of this method, the Muskingam Cunge Method takes the geometrical features of flood plain and channel for establishing the flood routing parameters, and is therefore, preferred over Muskingam method.

Mathematical Models

There are three kinds of models available in the literature to treat flood plains. They are described briefly below :

1. off-channel storage

This model assume that floodplains function only as a storage. The velocity of flow in the floodplain is assumed to be negligible and momentum effects are not considered.

2. Composite channel

This treats the channel and flood plains as one unit. A single Manning's "n" value is assumed. An excellent text on this Manning's "n" is given in Chow (1959). This model takes the carrying capacity of the plains also in lumped form.

3. Channel-plain model

This model distinguishes the two functions of the flood plains and accounts both of them. These functions are storage and conveyance. As such the wetted area A is the sum of area of flow in the channel, area of flow in the plains and the area of dead storage.

In order to illustrate its use, application of HEC-2 to route some flood experienced in river Digaru has been made. The information on river geometry has been collected form the field survey covering the river reach upstream and downstream of the two bridges considered in the study.

1.4 OBJECTIVE

The stream-flow measurement with floats in the case of large floods which inundate flood plain involves lot of error in the records. While using such records, care has to be taken to assess and possibly to remove such errors from the data used for the analysis. A careful analysis of stage-discharge relations would help to this end. The aim should be to develop rating curve at different reaches of the river so that hydrograph can be generated at any cross section of the river of the interest.

The application of any flood routing method has to properly account for these factors, when dealing with high floods. The method used to route a flood can be broadly classified into two categories, viz (i) Black Box Models, (ii) Mathematical Models. Flood Routing by HEC-2 method belongs to the later category and is now being used in many countries. This method uses the data of river cross sections to define river channel geometry, roughness coefficient for the main channel as well as flood plain to define flow characteristics, initial boundary conditions to arrive at the necessary flood hydrograph parameters at different reaches. Even if the observed floods are not large enough so as to include measurement in flood plains also, the program HEC-2 works fine and gives correct result to apply at another section. Different approaches used in HEC-2 have also been explained in subsequent sections.

Water surface profiles can be calculated by different methods. The most common and simple method is to calculate the water surface profile by standard step method in which profile is calculated reach by reach. The HEC-2 calculates water surface profiles for gradually varied flow by standard step method. The program's objective is quite simple - compute the water surface elevation at all locations of interests for given flow values.

2.0 HEC-2 COMPUTER PROGRAMME

2.1 DATA REQUIREMENT

The data needed to perform these computations include : flow regime, starting elevations, discharge, loss coefficients, cross section geometry and reach lengths.

2.1.1 Flow Regime

Subcritical profile computed by the program are constrained to the critical depth or above and super-critical profile are constrained to critical depth or below. The program does not allow profile computations to cross critical depth except for certain bridge analysis problem. In case where flow passes from one flow regime to another as shown in figure 1, it is necessary

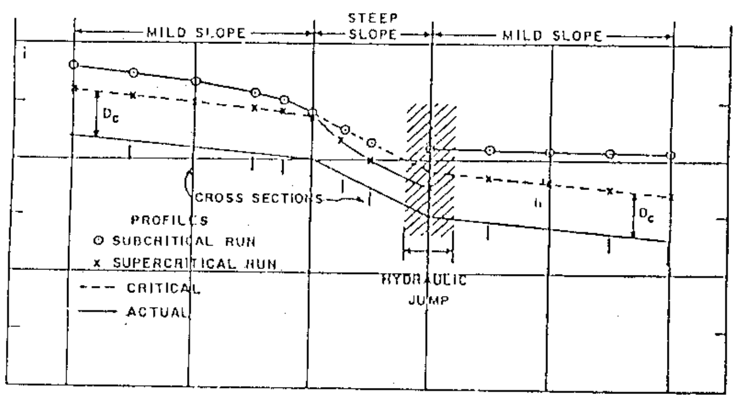


Figure 1 : Calculated Profiles for Subcritical and Supercritical Flows

to compute the profile twice, alternatively assuming subcritical and supercritical flow. HEC-2 does not contain the capability to determine the position of the Hydraulic Jump or energy losses associated with the jump.

2.1.2 Starting Elevation

The water surface elevation for the beginning cross section may be specified in one of the four way :

1) As critical depth.
2) As a known water surface elevation.
3) By the slope area method and
4) By a rating curve.

For beginning by the slope area method, estimated slope of the energy grade line is set equal to the slope of the ground and initial estimate of the water surface elevation is used. The flows computed for the fixed slope is adjusted until the computed flow is within one percent of the starting flow. The water surface elevation thus determined may be used as the starting water surface elevation for subsequent water surface profile computation.

2.1.3 Discharge

Discharge may be specified and altered in several way.

2.1.4 Energy Loss Coefficients

Several types of loss Coefficient are utilized by the program to evaluate head losses.

1) Manning's 'n' or equivalent roughens heights 'k' values for friction loss.
2) Contraction and expansion coefficient to evaluate transition (shock) losses.
3) Bridge and culvert loss coefficient to evaluate loses related to wear shape, pier configuration and pressure flow, and entrance and exit conditions.

2.1.4.1 **Manning's 'n'**

Since Manning's coefficient of roughness depends on such factors as type and amount of vegetation, channel configuration and stage, several options are available to vary 'n'. Channel and over bank roughness may be sufficient to describe the 'n' value, otherwise more 'n' values may be adequately described to define the lateral roughness variation in the cross section. 'n' may be given for individual cross section or for the reach as a whole. Data indicating variation of 'n' with river stage may also be used in the program. The program has capability to modify it by any desirable factor.

2.1.4.2 **Equivalent Roughness 'k'**

An equivalent roughness parameter 'k' commonly used in the hydraulic design of channels is provided for describing roughness in HEC-2.

The advantage of using 'k' instead of 'n' is that 'k' rejects changes in the friction factor due to stage where as Manning's 'n' alone does not. This influence can be seen in the definition of Chewy 'c' (English unit) for a rough channel

$$C = 32.6 \log_{10} \left[\frac{12.2\ R}{k}\right] \qquad \text{...... (1)}$$

where, C = Chewy roughness Co-efficient

R = Hydraulic radius (feet)

k = Equivalent roughness (feet)

From this equation it is evident that as R increase the friction factor 'C' increases. In HEC-2, 'k' is converted to a Manning's 'n' by the following equation:

$$n = \frac{R^{1/6}}{18.0 \log_{10} \left[\frac{12.2\ R}{k}\right]} \qquad \text{...... (2)}$$

Where 'n' = Manning's roughness Coefficient. Values of 'k' can be described in the program in the same way as for Manning's 'n'.

2.1.4.3 Contraction And Expansion Coefficient

Losses due to contraction and expansion of flow between cross-section are determined by Standard Step Profile calculations. Manning's equation is used to calculate friction losses and all other losses are described in terms of a coefficient times the absolute value of the change in velocity head between adjacent cross sections. When the velocity head increases in the downstream direction a contraction co-efficient is used and when the velocity head decreases, an expansion co-efficient is used.

2.1.4.4 Bridge Losses

HEC-2 computes energy losses caused by structures such as bridges and culverts in two parts. One part consists of the losses that occur in reaches immediately upstream and downstream from the structures where the contraction and expansion of the flow is taking place. The second part consists of losses at the structure itself and is calculated with either the normal bridge method or the special bridge method. As an alternative to having the losses being calculated by the program it is also possible to input the losses (or water surface elevations) determined externally from the program.

2.1.5 Cross Section Geometry

Boundary geometry for the analysis of flow in a natural streams is specified in terms of ground surface profile (cross-sections) and the measured distance between them (reach lengths). Cross-sections are located at intervals along a stream to characterize the flow carrying capability of the stream and its adjacent flood plains. They should extend across the entire flood plain and should be perpendicular to the anticipated flow line. Every effort should be made to obtain cross sections that accurately represent the stream and flood plain geometry. However, ineffective flow areas as the flood plain such as stream inlets, small ponds or indents in the valley floor should generally not be included in the cross sections geometry.

Cross-sections are required at representative locations throughout a stream reach and at a location where change occur in discharge, slope or end and at bridges or control structures such as weir. Where abrupt changes occur, several cross-sections should be used to describe the change regardless of the distance. Cross-section spacing is also a function of stream size, slope, and the uniformity of the cross section shape. In general, large uniform rivers of flat slope normally require the fewest member of cross sections per mile. The purpose of the study also affect spacing of the cross sections.

2.1.6 Reach Lengths

The measured distance between cross-sections are referred as reach length. The reach length for the left over bank, right over bank and channel are specified separately. Channel reach length is typically measured along the thalweg. Over bank reach length should be measured along the anticipated path of the center of mass of the over bank flow. Often these three values will be equal. But in the condition such as at river bends or where the channel meanders considerably and the over banks are straight, these three values will differ. When these three lengths are different, a discharge wetted reach length is determined based on the discharge in the main channel and left and right over bank segments of the reach.

2.2 HEC-2 CAPABILITIES

Hec-2 has numerous capabilities that allow the user to determine flood plains and floodways; to evaluate energy loss at obstruction such as weirs, culverts and bridges and to analyze improvements to drainage systems. Other program option include the capabilities to select from alternative friction loss equations, calculate critical depth, solve directly for Manning's 'n', automatically insert program generated cross sections, specify ineffective flow areas, analyze tributary streams, perform multiple profile analysis in a single execution of the program, analyze flow in ice covered stream.

2.2.1. Multiple Profile Analysis

In a single run HEC-2 can compute up to 14 profiles using the same cross-sectional data. After the last profile of a multiple profile run, a summary print out is generated which provides a concise summary of results for all profiles for each cross section.

2.2.2 Critical Depth

Several options related to the computation of critical depth are available in HEC-2. Critical depth may be required for each cross section of a subcritical run. Critical depth is calculated automatically for cross section of subcritical profiles whenever the calculated velocity head exceeds a test velocity head. The tolerance normally used is 2.5 % of the depth. The user can specify an alternate tolerance to be used for the automatic calculation of critical depth by indicating a positive value for tolerance.

2.2.3 Effective Flow Option

HEC-2 has capability to restrict flow to the effective flow areas of cross section. There are option to simulate sediment deposition, confine flow to the leveed channels, to block out road fills and bridge decks and to analyze flood plain encroachments. These options are illustrated in **figure 2.** Cross section with low over bank areas or levees require special consideration in computing water surface profile because of possible overflow into areas outside the main channel. Normally, computations are based on the assumption that all area below the water surface elevation is effective in passing the discharge. However, if the water surface elevation at a particular cross-section is less

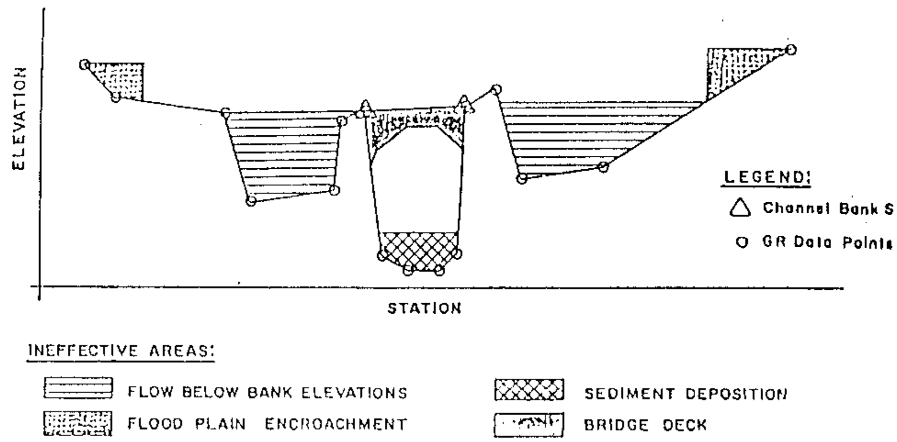


Figure 2 : Types of Effective Flow Options

than the top of levee elevation and if the water can not enter or leave the over bank upstream of cross section, the flow areas in these over bank should not be used in the computation. It is important for the user to study carefully the flow pattern of the river where levees exist.

2.2.4 Encroachment Option

Six methods of specifying encroachments for floodway studies can be used:

-- Stations and elevation of the left and/or right encroachment can be specified for individual cross section as desired (figure 3a).

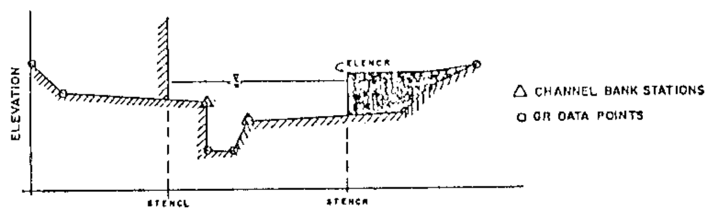


Figure 3(a) : Encroachment Method 1

A floodway with a fixed top can be specified which will be used for all cross sections until changed. The left and right encroachment station are made equidistant from the centerline of the channel which is half way between the left and right bank stations (figure 3b).

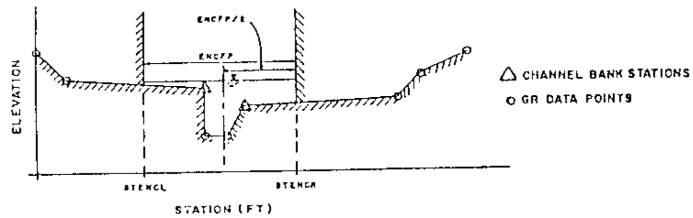


Figure 3(b) : Encroachment Method 2

Encroachment can be specified by percentage which indicate the desired proportional reduction in the natural discharge carrying capacity of each cross section (figure 3c).

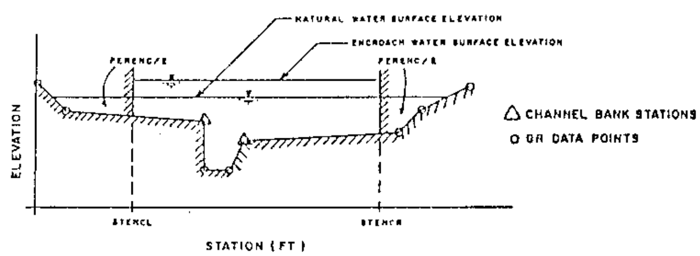


Figure 3(c) : Encroachment Method 3

Encroachment can be determined so that each modified cross-section will have the same discharge carrying capacity (at some higher elevation) as the natural cross section. This higher elevation is specified as a fixed amount above the natural profile (figure 3d). The encroachment is

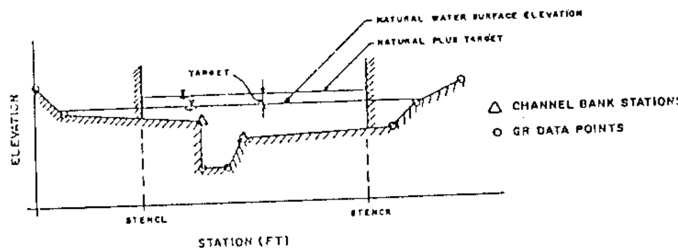


Figure 3(d) : Encroachment Method 4

determined so that at higher elevation possibly an equal loss of conveyance occurs on each side of the channel.

Determination of water surface elevation so that difference between the natural and encroached condition is such that target difference is obtained as near as possible (figure 3e).

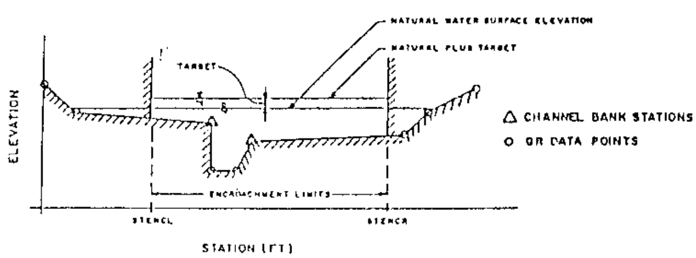


Figure 3(e) : Encroachment Method 5

-- Determination of energy grade line elevation so that difference between the natural and encroached condition is such that target difference is obtained as near as possible (figure 3f).

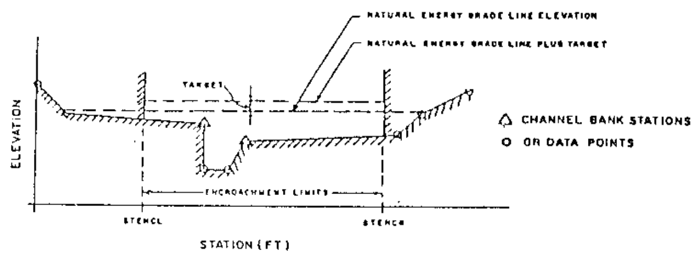


Figure 3(f) : Encroachment Method 6

2.2.5 Optional Frictional Loss Equations

The friction loss between adjacent cross section is computed as the product of the representative rate of friction loss (friction slope) and the wetted reach length. The profile allows the user to select from the following friction loss equation:

-- Average Conveyance Equation.

-- Average Friction Slope Equation.

-- Harmonic Mean Friction Slope Equation.

-- Geometric Mean Friction Slope Equation.

Any of the above equation will produce satisfactory estimate provided that reach lengths are not too long. The advantage of using alternate equation is to maximize reach length without sacrificing profile accuracy, since the program selects the most appropriate of the preceding four equations on a reach to reach basis depending upon the flow conditions within the reach.

2.2.6 Channel Improvements

Cross sectional data may be modified automatically by the program to analyze improvements made to natural stream sections. Up to five different bottom widths may be specified for the execution of a single run and improvements of three cross-sections can be utilized. **Figure 4 shows a sample** application of channel improvement option.

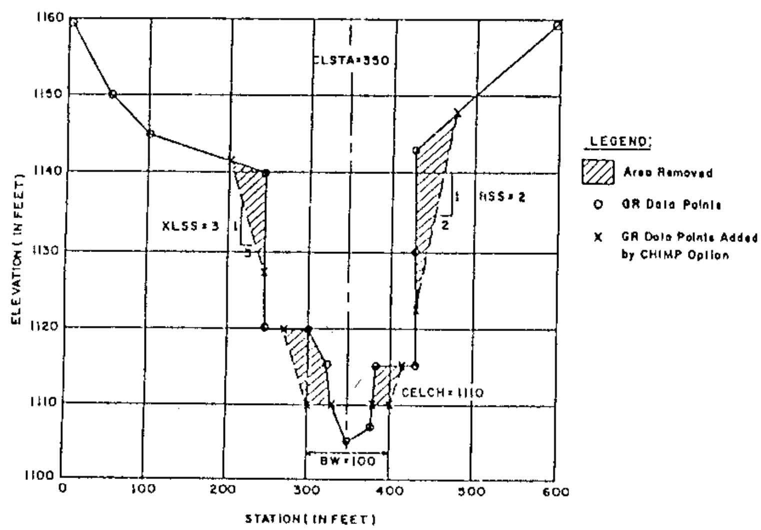


Figure 4 : A Stream Cross Section Before and After Channel Modification

2.2.7 Interpolated Cross Section

Occasionally it is necessary to insert cross-section between those specified by input, because the change in velocity head (δHV) is too great to accurately determine the energy gradient. Additional cross-sections may be coded manually or a program option may be requested to input interpolated cross-section.

Interpolated cross-sections are determined by raising or lowering and expanding or contracting the current cross-section's shape. They are inserted uniformly between the two input cross-sections. A proportion of the elevation difference determined from the minimum elevations of the two input cross-sections is added (or subtracted) to the elevation coordinates of the current cross-section. The modification of the horizontal coordinate is a function of the ratio of channel areas of the two input cross-section. The channel area (between bank stations) of the current cross-section is determined with the depth of flow from the previous cross-section.

The number of interpolated cross-sections added to each profile may vary with discharge. Therefore, it is advisable not to run the programme for multiple profile because the analysis will require the same cross-section data.

2.2.8 Tributary Stream Profile

Subcritical profile may be computed for tributary stream system for single or multiple profiles in a single execution of the program. In a single run, a profile for a stream system with second order tributary may be drawn. This may be accomplished if data for the tributary, with the tributary, is treated as a portion of the main stream. Then the main stream beyond the junction of the two stream is treated as a tributary. This is illustrated in figure 5; numbers 1 through 8 locate cross section on the main stream, number 11 through 16 are cross section on the first order tributary and number 21 through 22 are cross section on the second order tributary. The arrangement of cross section data for the stream system in figure 5 for a tributary in a single execution of the program is as follows : 1,2,3,4,11,12,13,14,15,16, -4,5,6,7,8, -14,21,22.

Tributary stream profile should not be calculated simultaneously with

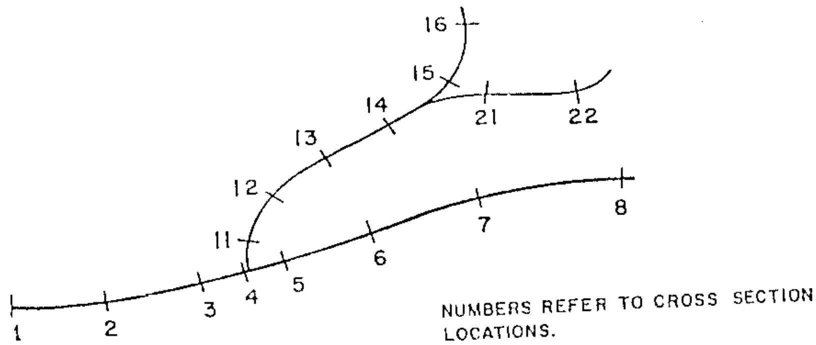


Figure 5 : Second Order Stream System

encroachment methods (3 to 6) and split flow option.

2.2.9 Solving For Manning's 'n'

HEC-2 can compute 'n' values automatically from high water data if the discharge, relative ratios of the 'n' values for the channel and over banks and water surface elevation at each cross section are known. The best estimate of 'n' for the first cross-section must be entered, since it is not possible to compute 'n' value for this cross-section. The relative ratio of 'n' values between channel and over bank is set by the first section and will be used for all subsequent cross-section unless changed by another ratio. The average friction slope is utilized by the profile to solve for 'n' values. When an adverse slope is encountered, computations restart using 'n' values from the previous section.

Another method is to specify the discharge and an assumed set of 'n' values and have the program compute a water surface profile which can be compared with the high water profile.

2.2.10 Storage Flow Data

HEC-2 storage outflow option can be used to generate input data for hydrograph routing using the Modified Puls Method which requires stream storage and corresponding discharge. Stream storage should be determined for a range of discharge which cover the anticipated range of flows for routed hydrographs.

The storage volume computed by the profile does not include any volume blocked out as ineffective flow. If the reach for which storage discharge data is being generated has ineffective flow areas, the storage data should be adjusted accordingly. In some cases, it is convenient to use high roughness coefficient to block out these ineffective flow areas. This approach retains the storage volume associated with these areas.

2.2.11 Split Flow Option

This option provides for the automatic determinations of channel discharges and profiles in situations where flow is lost from the main channel. The split flow option can model flow over levees or weirs, overtopping of watershed divides and flow splits created by diversion structure. This option allows the user to determine flow splits with weir or normal depth analysis or by direct input of rating curves.

2.2.12 Ice Covered Streams

HEC-2 has capability to determine water surface profile for streams with stationary floating ice covers. In addition to hydraulic analysis, the option determines the potential for ice jams.

2.3 THEORETICAL BACKGROUND

The concept and equations used for basic profile calculation, cross section subdivision for determining conveyance and velocity distribution, friction loss evaluation, iterative procedure for solving the basic equations and critical depth determination are discussed in the following sections :

2.3.1 Equations for basic profile calculations

The following two equations are solved by an iterative procedure of standard step method to calculate an unknown water surface elevation.

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = WS_1 + \frac{\alpha_1 . V_1 .^2}{2g} + h_e \quad \ldots\ldots (3)$$

$$h_e = L \bar{S_f} + C \left| \left[\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 . V_1^2}{2g} \right] \right| \quad \ldots\ldots (4)$$

Where : WS_1, WS_2 = Water surface elevations at ends of the reach.

V_1, V_2 = Velocity Co-efficient for flow at ends of the reach.

g = Acceleration due to gravity.

h_e = Energy head loss.

α = discharge weighted reach length

$\bar{S_f}$ = Representative friction slope for each reach

C = Expansion or contraction loss Coefficient

The discharge weighted reach length, L is calculated as :

$$L = \frac{\bar{L}_{lob} Q_{lob} + \bar{L}_{ch} Q_{ch} + \bar{L}_{rob} Q_{rob}}{\bar{Q}_{lob} + \bar{Q}_{ch} + \bar{Q}_{rob}} \quad \ldots\ldots (5)$$

Where $\bar{L}_{lob}$, $\bar{L}_{ch}$, $\bar{L}_{rob}$ = Reach lengths specified for flow in the left over bank, main channel and right over bank, respectively.

$\bar{Q}_{lob}$, $\bar{Q}_{ch}$, $\bar{Q}_{rob}$ = Arithmetic average of flows at the ends of the reach for the left over bank main channel and right over bank respectively.

Friction loss is evaluated as the product of $\bar{S}_f$ and L where S_f is the respective friction slope for reach and L given by equation (5). In HEC-2, $\bar{S}_f$ can also be obtained by any of the equations given below :-

Average Conveyance Equation

$$\bar{S}_f = \left[\frac{Q_1 + Q_2}{K_1 + K_2} \right]^2 \qquad \ldots\ldots (6)$$

Average friction slope equation

$$\bar{S}_f = \frac{S_{f1} + S_{f2}}{2} \qquad \ldots\ldots (7)$$

Geometric Mean Friction Slope Equation

$$\bar{S}_f = \sqrt{(S_{f1} \cdot S_{f2})} \qquad \ldots\ldots (8)$$

Harmonic Mean Friction slope Equation.

$$\bar{S}_f = \frac{2S_{f1} \cdot S_{f2}}{S_{f1} + S_{f2}} \qquad \ldots\ldots (9)$$

Equation (6) is the "default equation" used by the program, that is used automatically unless a different equation is specified.

The friction loss between adjacent cross-section is compared as the product of the representative rate of friction loss (friction slope) and the weighted reach length. Any of the above four friction loss equations will produce satisfactory estimate provided that the reach lengths are not too long.

The important feature of the capability of the program is that it selects the most appropriate of the preceding four equations on a reach basis depending on flow conditions (e.g. M1, S1, etc.) within the reach.

2.3.2 Cross section Sub-Division

For determining total conveyance and the velocity coefficient for a cross section, the flow should be subdivided into units for which the velocity is uniformly distributed. HEC-2 program sub divide the flow in the over bank areas using the input cross section stations as the basis for subdivision. Conveyance is calculated within each subdivision by the following equation (in English Unit)

$$K = \frac{1.486\ a\ r^{2/3}}{n} \quad \text{...... (10)}$$

where K = Conveyance for subdivision

n = Manning's 'n' for sub-division

a = Flow area for sub division

r = Hydraulic radius for sub division

The total conveyance for the cross section is obtained by summing the incre mental conveyance.

2.3.3 **Velocity Coefficient**

The velocity coefficient is computed by the following equation basec the conveyance in the three flow elements : left over bank, main channel, right over bank;

$$\alpha = \frac{(A_t)^2 \left[\frac{(K_{lob})^3}{(A_{lob})^2} + \frac{(K_{ch})^3}{(A_{ch})^2} + \frac{(K_{rob})^3}{(A_{rob})^2} \right]}{(K_t)^3} \quad \text{..... (11)}$$

where At = Total flow area of the cross section.

Alob, Ach, Arob = Flow area for left over bank, main channel and right over bank.

Klob, Kch, Krob = Conveyance of left over bank, main channel and right over bank.

The important feature of the capability of the program is that it selects the most appropriate of the preceding four equations on a reach basis depending on flow conditions (e.g. M1, S1, etc.) within the reach.

2.3.4 Critical Depth Determination

Critical depth for a cross section will be determined if any of the following conditions are satisfied :

(1) The super critical flow regime has been specified.

(2) Calculation of critical depth has been requested.

(3) This is the first cross section and critical depth starting conditions have been specified.

(4) The critical depth check for a subcritical profile indicates that critical depth needs to be determined to verify the flow regime associated with the balanced elevation.

The total energy head for a cross section is defined by :

$$H = WS + \alpha\, V^2/2g \qquad \text{...... (12)}$$

where H = Total energy head

WS = Water surface elevation

$\alpha V^2/2g$ = velocity head

The critical water elevation is the elevation for which the total energy head is a minimum. The critical elevation is determined with an iterative procedure whereby values of WS are assumed and the corresponding values of H are determined with equation (12) until a minimum values for H is reached.

2.4.0 METHODOLOGY ADOPTED

The unknown water surface elevation at a cross section is determined by an iterative solution of the equation 1 and 2. The computational procedure is as follows :

1. Assume a water surface elevation at the upstream cross section in the case of subcritical flow (or downstream cross section in the case of a super critical flow profile).

2. Using the assumed elevation, total conveyance K and velocity head are

computed.

3. With values from step 2, the energy slope is calculated from which head loss h_e is calculated.

4. With values from step 2 and 3, WS_2 is computed using the equation 1.

5. A new water surface elevation is assumed as an average of the computed value obtained from step 4 and the originally assumed value.

6. Step 2 to 5 are repeated until assumed value agrees within 0.01 m to get a balanced water surface elevation.

2.5 HEC-2 LIMITATIONS

The following assumptions are implicit in the analytical expressions used in the program :

(1) Flow is steady.

(2) Flow is gradually varied.

(3) Flow is one dimensional (i.e. velocity components in directions other than the direction of flow are not accounted for).

(4) River channels have small slopes, say less than 1:10.

The program does not have the capability to deal with movable boundaries (i.e. sediment transport) and requires that energy losses be definable with the terms contained in equation 3.

2.6 Flow Through Bridges

2.6.1 Normal Bridge Method

The normal bridge method handles a bridge cross section in the same manner as a natural river cross section except that the area of the bridge below the water surface is subtracted from the total area and the wetted perimeter is increased where the water is in contact with the bridge structure. The bridge deck is described either by entering the constant elevations of the top of the roadway and low chord as variables or by specifying a table of roadway stations and elevations and corresponding low chord elevations. Pier losses are accounted for by the loss of area and the increased wetted perimeter of the piers as described in terms of cross section co-ordinates.

2.6.2 Special Bridge Methods

The special bridge method computes losses through the structure for either low flow, pressure flow, weir flow or for a combination of these. The profile through the bridge is calculated using hydraulic formulas to determine the change in energy and water surface elevation through the bridge.

2.6.2.1 Low Flow

The procedure used for low flow calculation in Special bridge method depends on whether the bridge has piers. Without piers the flow solution is accomplished by Standard Step calculation as in the normal bridge method because the equations used in the special bridge method for low flow are based on the obstruction width due to piers. Without piers the Special Bridge Method would indicate that no losses would occur. For a bridge with piers, the program goes through a momentum balance for cross sections just outside and inside the bridge to determine the class of flow.

2.6.2.2 Class A low flow occurs when the water surface through the bridge is above the critical depth i.e. subcritical flow. In the momentum equations, a trapezoidal approximation of the bridge opening is used to determine the area

$$H_3 = 2 K (K + 10 W - 0.6) (\alpha + 15 \alpha^4) \frac{V_3^2}{2g} \quad \ldots\ldots (13)$$

where

H_3 = Drop in water surface from upstream to downstream side of the bridge

k = Pier shape coefficient

W = Ratio of the velocity head to depth downstream from the bridge.

α = Obstructed area/Total unobstructed area.

The computed water surface elevation is simply the downstream water surface elevation plus H_3. When the upstream surface elevation is known, the program computes the corresponding velocity head and energy elevation for the upstream section.

2.6.2.3 **Class B low flow** can exist for either a subcritical or super-critical profile. For either profile, class B low flow occurs when the profile passes through critical depth in the bridge constriction. For a **subcritical profile,** critical depth is determined in the bridge, a new downstream depth (below critical) and the upstream depth(above critical) are calculated by finding the depths whose corresponding momentum flux equals the momentum flux in the bridge for critical depth. With this solution the hydraulic jump occurs downstream as the downstream elevation becomes the super-critical elevation. The program does not provide the location of the hydraulic jump. A super critical profile could be computed starting at the downstream section with the super critical water surface elevation given by the computation of class B low flow. For **super critical profile,** the bridge is acting as a control and is causing the upstream water surface elevation to be above critical depth. Momentum equations are again used to recompute an upstream water surface elevation (above critical) and a downstream elevation below critical depth. For this solution a new upstream elevation is given by the program with a noting that new backwater is required, indicating a subcritical profile should be calculated upstream from the bridge starting at elevation given by this computation.

2.6.2.4 **Class C low flow** is computed for a super critical profile where the water surface profile stays super critical through the bridge constriction. The downstream depth and the depth in the bridge are computed by the momentum equations based on the momentum flux in the constriction and the upstream depth.

2.6.2.5 Pressure Flow

The pressure flow computes the flow as orifice equations given by

$$Q = A\sqrt{(2gH/k)} \qquad \text{...... (14)}$$

where

H	=	Difference between the energy gradient elevation upstream and tail water elevation downstream.
k	=	Total loss coefficient
A	=	Net area of the orifice
g	=	Gravitational acceleration
Q	=	Total orifice flow

The total loss coefficient k, for determining losses between the cross sections immediately upstream and downstream from the bridge is equal to 1.0 plus the sum of the loss coefficients for intake, intermediate piers, friction and other minor losses.

2.6.2.6 Weir Flow

Flow over the bridge and the roadway approaching the bridge is calculated by standard weir equation:

$$Q = CLH^{3/2} \qquad \text{...... (15)}$$

Where

C	=	Coefficient of discharge.
L	=	Effective length of weir controlling flow.
H	=	Difference between the energy grade line elevation and the roadway crest elevation.

Q = Total flow over the weir.

The approach velocity is included in using the energy grade line elevation in lieu of the upstream water surface elevation for computing the head, H. Values for coefficient of discharge 'C' are presented in the section on loss coefficient. Where submergence by tail water exists, the coefficient 'C' is reduced by the program. Submergence corrections are based on a trapezoidal weir shape or optionally an Ogee spillway shape. A total weir flow, Q, is computed by subdividing the weir crest into segments, computing L,H, a submergence correction and Q for each segment, and summing the incremental discharge.

2.6.2.7 Combination Flow

Sometimes combinations of low flow or pressure flow occur with weir flow. In these cases, a trial and error procedure is used, with the equations just described, to determine the amount of each type of flow. The procedure consists of assuming energy elevations and computing the total discharge until the computed discharge equals, within 1 percent, the discharge desired.

2.7 General Modeling Guidelines for flow through bridge

Figure 6 shows the basic configuration of cross sections for computing losses through bridges.

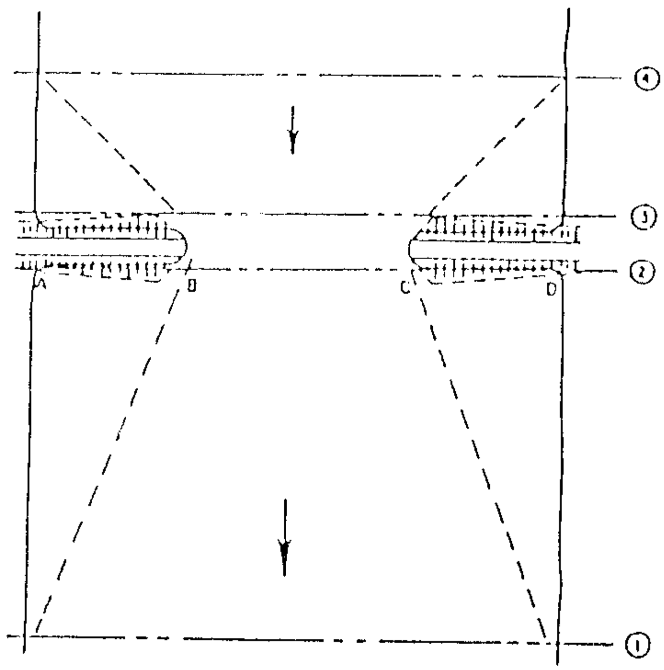


Figure 6 : Cross Section Locations in the Vicinity of Bridges

Cross section 1 is sufficiently downstream from the bridge where the flow has fully expanded, not affected by the bridge. The basic input problem is to determine how far downstream from the bridge the cross section should be located. As a thumb rule it should be located about four times the average length of the side constriction caused by the bridge abutments. Therefore, cross section 1 would be located downstream from the bridge four times the distance AB or CD as shown in the figure 6. Because the constriction of flow may vary with the discharge, the downstream reach length should represent the average condition if a range of discharge is used in the model.

Cross section 2 is a river cross section immediately (within a foot or two) downstream from the bridge. The cross section should represent the effective flow (effective flow is that portion of flow where the main velocity is normal to the cross section and in the downstream direction) area just outside the bridge and its location could be considered as the downstream face of the bridge. It is important to work with effective flow area because it is assumed in the application of the energy equation that the mean downstream velocity for each subsection can be determined from Manning's equation. The standard step solution at cross section 2 would include determination of the expansion loss from cross section 2 to cross section 1.

Cross section 3 represents the effective flow area just upstream from the bridge. The reach length from cross section 2 to cross section 3 is generally equal to the width of the bridge. A standard step solution from a cross section in the bridge to this cross section provides the energy elevation. The energy loss between cross section 2 and 3 represents the loss through the bridge structure itself.

Cross section 4 is an upstream cross section where the flow lines are approximately parallel and the full cross section is effective. Because the flow contraction can occur over a shorter distance than the flow expansion, the reach length between cross-sections 3 and 4 can be about one time the average bridge opening between the abutments (distance B-C in figure 6). However, this criterion of locating the upstream cross-section may result in too short a reach length for situations where the ratio of the width of the bridge opening to the width of the flood plain is small. An alternative criterion would be to locate the upstream cross section at a distance equal to the bridge contraction (distance AB or CD in fig. 6). The program will compute the contraction portion of the loss over the reach length by standard step calculations.

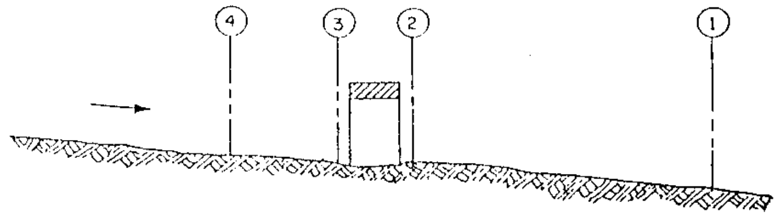
2.7.1 EFFECTIVE AREA OPTION

A basic problem in setting up the bridge routines is the definition of effective flow area near the bridge structure. In figure 6, the dashed line represent the effective flow boundary for low flow and pressure flow condition. Therefore, for cross section 2 and 3, ineffective flow areas to either side of the bridge opening (along distance AB and CD) should not be included for low flow or pressure flow. The elimination of the ineffective over bank area can be accomplished by redefining the geometry at cross-sections 2 and 3 as shown in part C of figure 7 or by using the natural ground profile and requesting the program's effective area option to eliminate the use of over bank area.

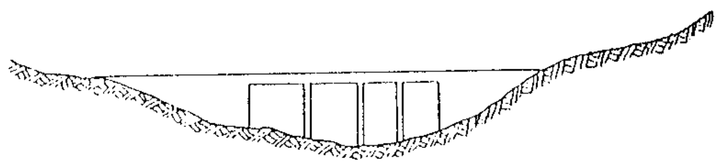
By redefining the cross section, a fixed boundary is used at the sides of the cross-section to contain the flow, when in fact a solid boundary is not physically there. The use of the effective area option does not add perimeter to the flow boundary above the given ground profile.

The effective area option of the program is used to keep all the flow in the channel until the elevations associated with the left and/or right bank stations are exceeded by the computed water surface elevations.

The bridge example shown in figure 7 is a typical solution where the bridge spans the entire floodway and its abutment obstruct the natural floodway. The program converts the input profile of cross sections at 2 and 3 to that part shown in part C of figure 7.



A. Channel Profile and Section Locations



B. Bridge Cross Section on Natural Floodway

C. Portion of Cross Sections 2 & 3 Effective for Low Flow and Pressure Flow

Figure 7 : Cross Sections Near Bridges

The program allows controlling elevations on the left and right bank stations to be specified by the user. If these elevations are not read in, elevations specified on the ground records data for the left and right bank stations are used. These elevations would correspond to an elevation where weir flow would just start over the bridge.

The effective are option applies to the left and right bank station. Therefore, those stations should coincide with the abutments of the bridge. For cross section 2 and 3, the left and right bank station should line up with the bridge abutment. For the downstream cross section, the threshold water surface elevation for weir flow is not usually known on the initial run, so an estimate must be made. For a reasonable estimate, average of elevation of low chord and top-of-road could be used. The entire over bank becomes effective as soon as the effective area elevation is exceeded. The assumption is that under weir flow conditions, the water can generally flow across the whole bridge length and the entire over bank in the vicinity of the bridge would be effective for flow up to and over the bridge.

Cross-section 3 is put same as the cross-section 2. The only difference, generally, is the elevations to use for the effective area option. For the upstream cross section, the elevation would be the low point of the top-of-road.

Effective area option provides for a constricted section when all of the flow is going under the bridge. When the water surface elevation is higher than the controlled elevations, the entire cross-section is used. The program user should check the computed solutions on either side of the bridge section to ensure that they are consistent with the type of flow. That is, for low flow or pressure flow, the printout should show the effective area restricted to the main channel. When the bridge data indicates weir flow, the solution would show that the entire cross section is effective.

2.7.2 LOSS COEFFICIENTS

Referring to figure 6, the new values of coefficients should be read in just before section two and changed back to the original values after section four. Typical values are shown below :

TABLE 1

CONTRACTION AND EXPANSION COEFFICIENTS

	Contraction	Expansion
No transition loss computed	0.0	0.0
Gradual transition	0.1	0.3
Bridge sections	0.3	0.5
Abrupt transition	0.6	0.8

The maximum values for the expansion coefficient would be 1.0

2.7.2.1 Special Bridge Coefficients

When using the special bridge methods, coefficients must be read in for the orifice equation and the weir equation.

Pier Shape Coefficient is used for computing the change in water surface elevation through a bridge for class A low flow. Because the calculation is based on the presence of piers, both the coefficients and a total width (BWP) must be specified. The table 2 gives values of coefficients for various pier shape.

TABLE 2

PIER SHAPE COEFFICIENTS

Pier Shape	Coefficient
Semi circular nose and tail	0.90
Twin cylinder pier with connecting diaphragm	0.95
Twin cylinder piers without diaphragm	1.05
90 degree triangular nose and tail	1.05
Square nose and tail	1.25

Loss coefficient is used in the orifice flow equation

$$Q = A \sqrt{(2\ g\ H/K)} \qquad \text{...... (16)}$$

Coefficient of Discharge is used in the standard weir equation

$$Q = C\ L\ H^{3/2} \qquad \text{...... (17)}$$

Under free flow condition, 'C' ranges from 2.5 to 3.1 (1.39 - 1.78 metric) for broad crested weirs depending primarily on the gross head on the crest ('C' increases with head). Increased resistance to flow caused by obstruction such as trash on bridge, railings, curbs and other barriers decreases the value of 'C'. For submerged flow (discharge affected by tail water), the 'C' should be reduced.

Assumptions of a rectangular weir for flow over the bridge deck, assuming the bridge can withstand the forces, a coefficient of 2.6 is appropriate. If the weir flow is over the road approaching to the bridge, a value of 3.0 seems appropriate.

3.0 APPLICATION

3.1 Description of the Study Area

Plenty of water resources is available in North Eastern Region. It is necessary to utilize this water resource for generation of power, irrigation, water supply and other developmental activities. For these purposes, the basic hydrological data and investigation will be required for planning, investigation and design purposes.

The mighty river of North East, Brhamputra covers an area of about 5,80,000 sq. km. (approximately) right from its origin to the outfall in the Bay of Bengal. It traverses a distance of about 1,700 km. in Tibet, 900 km. in India and 300 km. in Bangladesh totaling length of 2,900 km.

In the upper reaches it is joined by a number of tributaries which originate at different elevations in the hills encircling the catchment forming watershed. Among the tributaries Subansiri, Manas, Jia-Bharali, Pagladiya, Puthimari and Sankosh etc. are snow fed.

The river Brahmaputra originating from Himalayan lake MANAS SAROVAR in Tibet flows eastwards. Initially known as "TSANGPO". The watershed area is mostly on the northern side of the river in this region. After traveling a distance of about 1700 km. eastward of the river "TSANGPO", the river turns changing the courses from east to south and then entering the Indian territory. Its name also changes from "TSANGPO" to Dehang and Siang in Arunachal Pradesh. The river then flows almost in southern direction for another distance of 200 km. upto Passighat before touching plains where it is joined by major Himalayan tributaries like Lohit. The combined flow of these rivers is known as the Brahmaputra and passes through the plains of Assam and Bangladesh before falling into the Bay of Bengal.

Geographically, the Brahmaputra valley is long and narrow. This valley is bounded by the Lower Himalayan range on the north running along Assam, Burma on the east and by Garo, Khasi and Naga hill ranges on the south. A number of major tributaries on both the sides join in India while the southern tributaries viz. Burideding, Dikhow and Desang etc. originate from Patkai, Mikir, Khasi, Garo and Naga hills. The northern tributaries originate mainly from the lower Himalayas. Most of the northern bank tributaries of the river Brahmaputra are having very steep slope, shallow braided channels for a considerable distance from the foothills and carry heavy silt, discharge and also carry flash floods.

The portion of the Brhamputra under consideration is bounded by the eastern Himalayas to the north, its covered extension as Patkai Hills to the east and Garo, Khashi, Mikkir and Naga hills to the south. All the tributaries have mainly hilly catchment with main rivers flowing narrow plains having opening to the west.

In the plain also the river Brhamputra and its major northern tributaries have steep slope, narrow braided channel for a considerable distance from the foot-hills. They have coarse sandy bed and heavy silt discharge. The tributaries generally have flash floods and frequent lateral shifting of channels.

The Brhamputra is a braided channel through out its length in Assam and in a number of channels and cross channels between its high banks. There is a constant silt movement resulting in shifting of these channels and sandy shoale. In monsoon, all the channels combine together to form a vast sheet of water. The average width of Brhamputra valley between foot hills is about 80 Km. of which the river itself has a width of 6 to 10 Km. in most places. There are many islands in the river, Majuli with an area of about 875 sq. km. is the biggest river island in the world

The Digaru river is a small tributary of river Brhamputra which joins the river brhamputra downstream of Sonapur. The road bridge at Sonapur is located on Guwahati-Shillong National Highway No.37 at about 20 km. south west of Guwahati. Catchment area of river Digaru is 1210 sq. km. Geographical location of the gauge and discharge site is longitude of 91^{o}-59'-05" E and latitude of 26^{o}-06'-55" N., located at right bank of river. In the vicinity of the discharge site, the average height of the bank is 3.00 m and the maximum water width is 121.50 m. Width of the river from high bank to high bank is 61.00 m. The gauge and discharge site can be assessed either by rail or road from Guwahati.

Data on daily discharge and water elevation at Sonapur, observed suspended sediment was obtained from Central Water Commission. Maximum and minimum values of water level, discharge and silt for the discharge site at Sonapur is given in Table (3).

The location map of the site is shown in figure 8.

Table 3

MAXIMUM AND MINIMUM VALUES OF WATER LEVEL, DISCHARGE AND SILT

Site : Sonapur, River : Digaru, Basin : Brhamputra
Standard Bank : Right, Zero of Gauge : 50.000 m.

Sr. No.	Particulars	On record up to 12/91	During the year 1992	Remarks
1.	Maximum Water Level (m)	55.500	54.420	
	Date	15-10-91	01-09-92	
	Times (Hrs.)	10:00	06:00	
2.	Corresponding discharge (Cumecs)	428.00*	217.0 *	'*' Indic-ted
3.	Mimimum Water Level (m)	50.260	50.140	
	Date	15-03-91	12-03-92	
	Times (Hrs.)	08:00	08:00	
4.	Corresponding discharge (Cumecs)	9.284	6.570	Computec
5.	Maximum Discharge(Cumecs	353.30	216.100	
	Date	04-07-89	28-06-92	
	Times (Hrs.)	08:00	08:00	
6.	Corresponding Waterlevel (m)	54.740	54.090	Values
7.	Minimum Discharge(Cumecs	9.284	6.570	
	Date	15-03-91	12-03-92	
	Times (Hrs.)	08:00	08:00	
8.	Corresponding Waterlevel (m)	50.260	50.140	
9.	Total Annual Silt Load in (MT) Max	328314	269719	
	Year	1989		
	Min	252520		
	Year	1191		
10.	Maximum one day silt Load (MT)	7537	3753	
11.	Corresponding runoff (in Thousand H.M.)	1.6004	1.867	
12.	Minimum one day silt Load (MT)	20.600	44.95	
	Date	23-03-91	09-05-92	
13.	Corresponding run-off (in Thousand H.M.)	0.111	0.078	
14.	Maximum intensity of Silt (g/L)	0.538	0.273	
	Date	05-07-90	04-09-92	
15.	Minimum intensity of Silt [illegible]	0.025	[illegible]	
	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	607		
	Year	1991	550	
	Min.	606		
	Year	1990		
18.	Annual yield of water depth over the catchment area (m). Max.	1.677	1.380	
	Year	1991		
	Min.	1.480		
	Year	1990		
19.	Annual sediment yield over the catchment area (mm)		0.159	

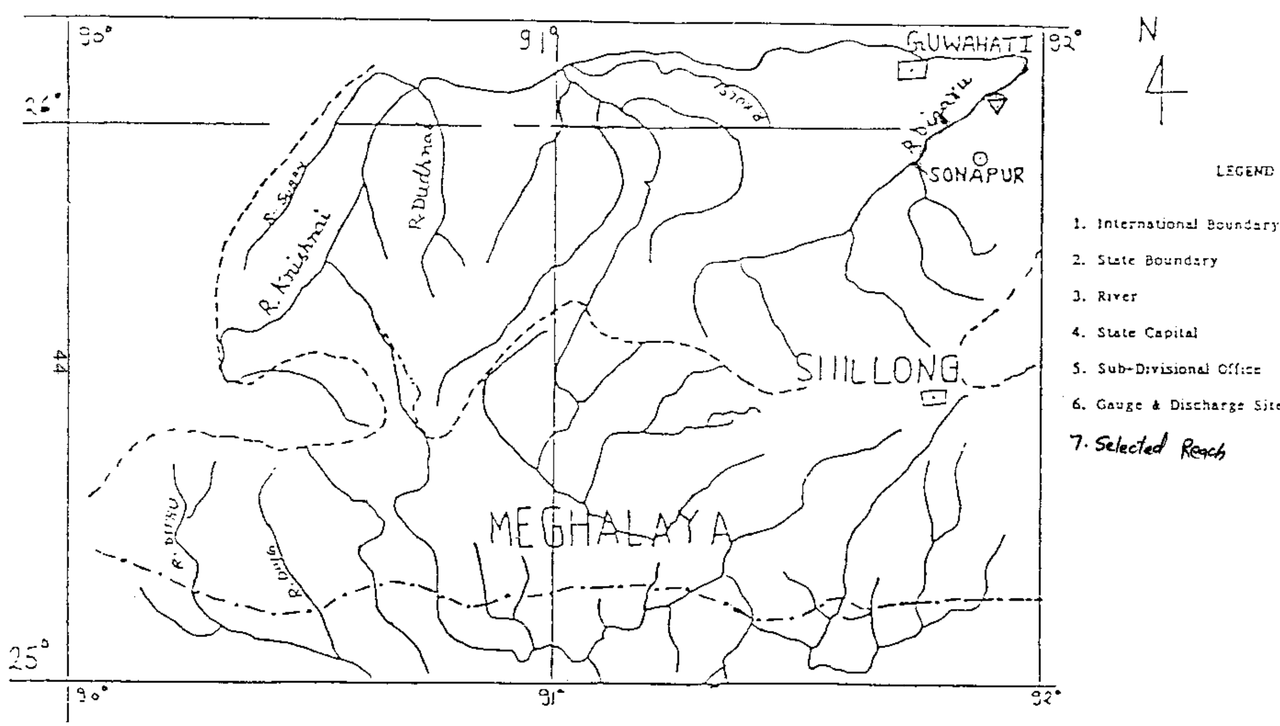


Figure 8 : Index Map of the Study Area

3.2 Data Used

Data of Digaru river, a tributary of river Brhamputra, has been used for the purpose of this study. Information on cross-sections of the river at six sites at different interval was obtained from the field survey. Slope of the reach of the river was obtained from the longitudinal distance along the river section obtained by field survey. The cross sectional distances along the reach actually taken during field survey is shown in the table 4.

Table 4

Summary of Reach Lengths

Reach	River Sta.	Left	Channel	Right
		Reach Length (m)		
1	20	580	550	520
1	18	360	340	320
1	16	65	60	55
1	14	Bridge		
1	12	470	460	450
1	8	85	80	75
1	7	Bridge		
1	6	320	300	280
1	4	750	700	650
1	2	0	0	0

Manning "n" values used at each cross section is given in table 5 and contraction and expansion coefficients used in the program are given in table 6.

Table 5

SUMMARY OF MANNING'S N VALUES

Reach	River Sta.	n1	n2	n3
1	20	.0671	.0442	.0671
1	18	.0671	.0442	.0671
1	16	.0672	.0442	.0672
1	14	Bridge		
1	12	.0672	.0442	.0672
1	8	.0672	.0442	.0672
1	7	Bridge		
1	6	.0672	.0442	.0672
1	4	.0672	.0442	.0672
1	2	.0672	.0442	.0672

Table 6

SUMMARY OF CONTRACTION & EXPANSION COEFFICIENTS

Reach	River Sta.	Contr.	Expan.
1	20	.3	.5
1	18	.3	.5
1	16	.3	.5
1	14	Bridge	
1	12	.3	.5
1	8	.3	.5
1	7	Bridge	
1	6	.3	.5
1	4	.3	.5
1	2	.3	.5

3.3 Analysis and Results

The observed discharge and the computed Manning' 'n' from Stage Measurements are plotted as shown in figure 9. This figure shows different pattern of variation; one for discharge less than about 30 cumecs and other for higher discharges. This plot is used to estimate the Manning' 'n' to be input to the programme as given in table 5. The location of the cross-sections in the reach is shown in figure 10. The cross-sectional drawings were discretised and the necessary reduced distances, reduced levels were tabulated. They are given as input to the programme in the required format.

Initial Conditions

Out of the several options as detailed in section 2.1.2 of this reoprt the water surface elevation for the beginning cross section has been specified by the slope area method. For beginning by the slope area method, estimated slope of the energy grade line is set equal to the slope of the ground and initial estimate of the water surface elevation is used. The slope at hte downstream reach used in the computation is given below:

```
* Reach          Profile *     Upstream              Downstream
*****************************************************************************
* 1                    1 *                           Normal S =  .00042 *

* 1                    2 *                           Normal S =  .00042 *

* 1                    3 *                           Normal S =  .00042 *
*****************************************************************************
```

The profiles 1,2 and 3 were computed for discharge of 9.284 cumecs, 100 cumecs and 250 cumecs respectively.

The flows computed for the fixed slope is adjusted until the computed flow is within one percent of the starting flow. The water surface elevation thus determined get used as the starting water surface elevation for subsequent iteration for water surface profile computation by the program.

The program uses the following computational criteria

Water surface calculation tolerance = 0.01

Critical depth calculaton tolerance = 0.01

Maximum number of interations = 20

Maximum difference tolerance = 0.3

Flow tolerance factor = 0.001

Computational Flow Regime: Subcritical Flow

Encroachment Data: None

Flow Distribution Locations: None

FLOW DATA (m^3)

Reach	Riv Sta	PF#1	PF#2	PF#3
1	20	9.284	100	250

Energy loss coefficients has been expressed as Manning's "n" taken from tabel 5 for each cross section and contraction & expansion loss coefficients for each cross section taken from table 6.

Bridge Data

Data for the bridges used in the analysis is given below :

Description: BRIDGE nO. 1 (WOODEN BRIDGE)

River Station = 14 of reach 1

Distance from Upstream XS = 25 m.

Deck/Roadway Width = 10 m.

Weir Coefficient = 2.6

Bridge Deck/Roadway Skew = 0.0

Elevation at which weir flow begins = 56.6

Maximum allowable submergence for weir flow = .95

Submergence criteria :Broad Crested

Upstream Deck/Roadway Coordinates, No. = 7

	Elevation (m)			Elevation (m)			Elevation (m)	
Sta.	Hi Cord	Lo Cord	Sta.	Hi Cord	Lo Cord	Sta.	Hi Cord	Lo Cord
0	56.6	48.55	13	56.6	54.113	13	56.6	55.1
73	56.6	55.1	73	56.6	51.9	78	56.6	53.8
83	56.6	48.55						

Downstream Deck/Roadway Coordinates, No. = 7

	Elevation (m)			Elevation (m)			Elevation (m)	
Sta.	Hi Cord	Lo Cord	Sta.	Hi Cord	Lo Cord	Sta.	Hi Cord	Lo Cord
0	56.6	48.55	13	56.6	54.113	13	56.6	55.1
73	56.6	55.1	73	56.6	51.9	78	56.6	53.8
83	56.6	48.55						

Pier Data

Number of Piers = 3

Pier Station	Upstream	=	23m.	Downstream =	23m.
		=	43m.	=	43m.
		=	63m.	=	63m.

Pier No.	Pier Station	Width Elevation (m)	Width (M)
1	23	48.5	1
		55.1	1
2	43	48.5	1
		55.1	1
3	63	48.5	1
		55.1	1

Number of Bridge Coefficient Sets = 1

Low Flow Methods

Energy

Momentum Cd = 2

Yarnell KVal = 1.25

Selected Low Flow Methods = Highest Energy Answer

High Flow Method
Submerged Inlet + Outlet Cd = .8

Only low flow method has been used in the computation

Additional Bridge Parameters
Add Friction component to Momentum
Do not add Weight component to Momentum
Class B flow critical depth computations use critical depth inside the bridge at the downstream end
Criteria to check for pressure flow = Upstream water surface

Description: BRIDGE NO. 2 (G & D SITE BRIDE)

River Station = 8 of reach 1

Distance from Upstream XS = 20 m.

Deck/Roadway Width = 20 m.

Weir Coefficient = 2.6

Bridge Deck/Roadway Skew = 0.0

Elevation at which weir flow begins = 62.78 m.

Maximum allowable submergence for weir flow = .95

Submergence criteria :Broad Crested

Upstream Deck/Roadway Coordinates, No. = 7

	(Elevation (m)			Elevation (m)			Elevation (m)	
Sta.	Hi Cord	Lo Cord	Sta.	Hi Cord	Lo Cord	Sta.	Hi Cord	Lo Cord
0	62.78	49	20	62.78	54.35	20	62.78	59.28
120	62.78	59.28	120	62.78	54.2	135	62.78	55.53
147	62.78	49						

Downstream Deck/Roadway Coordinates, No. = 7

	Elevation (m)			Elevation (m)			Elevation (m)	
Sta.	Hi Cord	Lo Cord	Sta.	Hi Cord	Lo Cord	Sta.	Hi Cord	Lo Cord
0	62.78	49	20	62.78	54.35	20	62.78	59.28
120	62.78	59.28	120	62.78	54.2	135	62.78	55.53
147	62.78	49						

Pier Data

Number of Piers = 2

Pier Station	Upstream =	48m.	Downstream =	48m.
	=	99m.	=	99m.

Pier No.	Pier Station	Width Elevation (m)	Width (M)
1	48	50.5	2
		59.28	2
2	99	50.5	2
		59.28	2

Number of Bridge Coefficient Sets = 1

Low Flow Methods
- Energy
- Momentum Cd = 2
- Yarnell KVal = 1.25

Selected Low Flow Methods = Highest Energy Answer

High Flow Method
- Submerged Inlet + Outlet Cd = .8

Additional Bridge Parameters
- Add Friction component to Momentum
- Do not add Weight component to Momentum
- Class B flow critical depth computations use critical depth inside the bridge at the downstream end
- Criteria to check for pressure flow = Upstream water surface

The results obtained are depth of flow at various sections along the river, cross section plots as opted and also water surface profile plot at each cross section. These results were transferred to Graphic software and plots were made. These plots are shown in figures 11 (a) to 11 (j). Longitudinal view of the water surface profile is shown in figure 12. Rating curves at each cross section of the reach are shown in figures 13(a) to 13(I). Some of the direct outputs of HEC-2 are given in Table-7, 8. A quick glance of water surface elevation for different discharge at each cross section is given in Table-9. Water surface elevations only at the bridge site are given in Table-10. Comparison of water surface profile and flow parameters at the two bride is given in Table-11. The perspective flood plain of the reach is shown in figure 14 and perspective flood plain with profiles for discharge of 9.284, 100 and 250 cumecs is shown in figure 15.

Comparison of the computed and observed Rating Curves

Stage versus Discharge curve for river Digaru at G & D bridge site at Sonapur as computed by Central Water Commission, Guwahati and as computed by HEC-2 programme is shown together in figure 16. Stage Vs Discharge curve as computed by HEC-2 matches with that compute by CWC. Henceforth results of HEC-2 as applied on river Digaru can be applicable for subsequent reaches of the river.

Sensitivity of the profiles to varying "n"

The calculated water surface profile is very much sensitive to the variation in the Manning's "n" values. This is explicit in the relation of the depth of flow with "n" as given in the equation below:

$$Y = (Q. n/S^{1/2} T)^{3/5} \quad \text{....(18)}$$

for a wide natural river where top with is at least ten times the depth of flow which is quiet valid for most of the natural rivers in India.

Here, Q is the discharge passing through the reach.

T is the top width of the river

S is the ground slope of the reach.

After differentiating the above equation we find that

$$dY = C. n^{-2/5} dn \quad \text{..... (19)}$$

where C involves the term of Q, S and T because they are not function of "n". In incremental form differential equation can be written as

$$Y = C. n^{-2/5} \quad n \quad \text{..... (20)}$$

Variation of "n" with discharge is given in figure 9. Since discharge is a function of depth of flow Y. Hence the equation 20 also holds good for discharge versus "n". As the "n" values are increased the slope of the curve of discharge versus "n", or depth of flow versus "n" reverses. Therefore in the analysis of flow by using equation 18, a reverse slope is encountered and the computation starts again. This computation will keep on alternating and the stable profile will not be obtained. For the given site value of "n" with respect to depth of flow was obtained from the available information on discharge versus "n" and discharge versus Y curve. These values are given below:

Stage of Flow D (m)	Manning's "n"
54.4	0.04419
53.7	0.04568
52.8	0.04598
51.5	0.04741
50.6	0.0517
50.2	0.06716

Covariance of depth of flow with respect to "n" values were found to be -0.011.

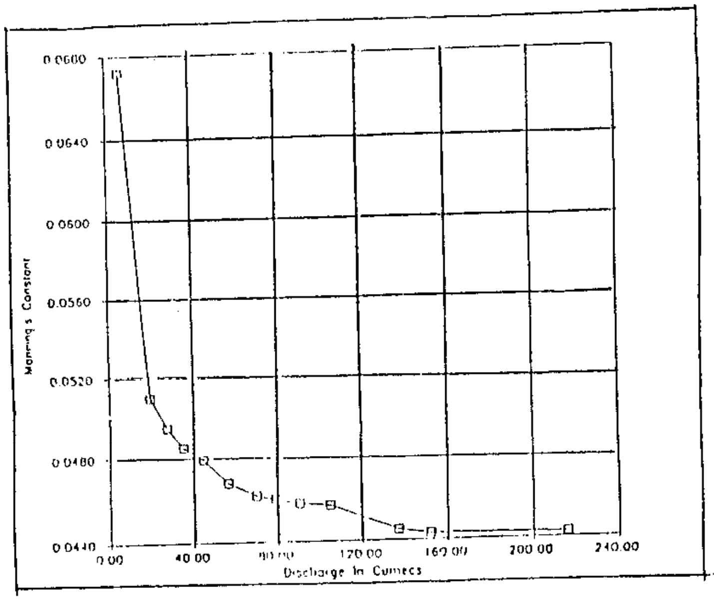


Figure 9 : Stage Vs Manning's Constant

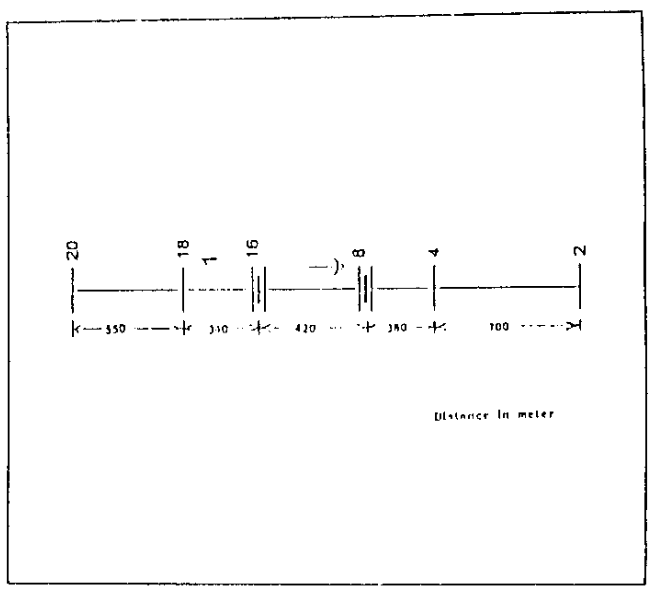


Figure 10 : Cross Sectional Location for the Reach

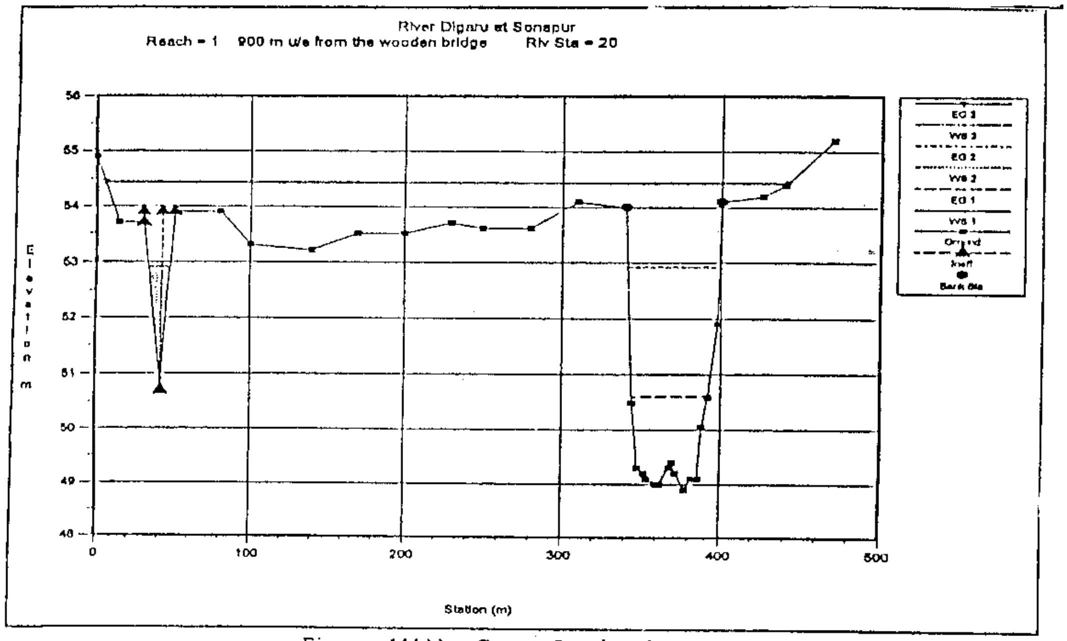


Figure 11(A) : Cross Sectional Plot

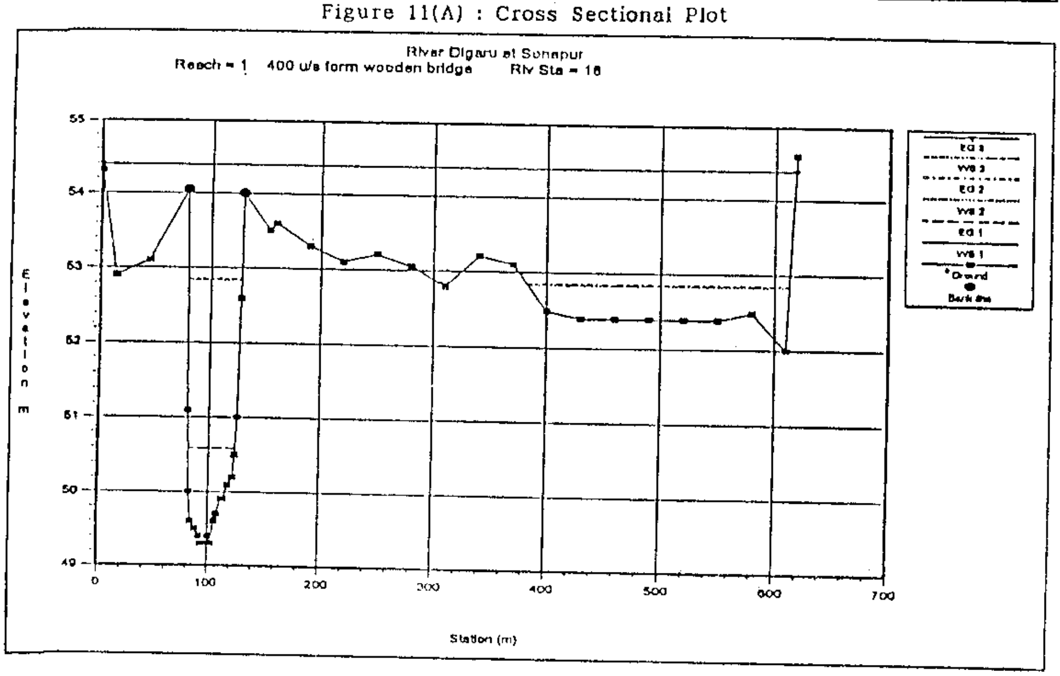


Figure 11(B) : Cross Sectional Plot

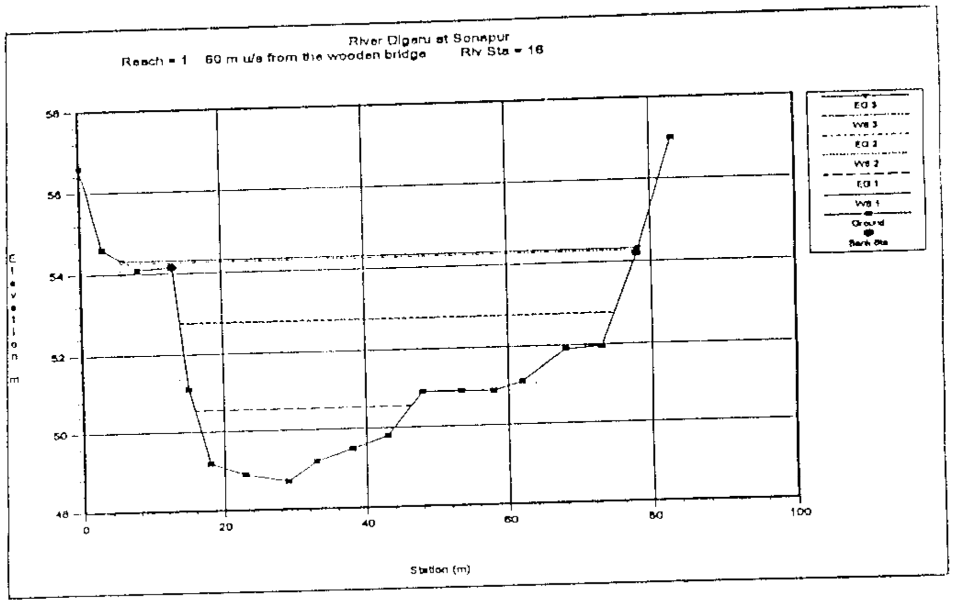


Figure 11(C) : Cross Sectional Plot

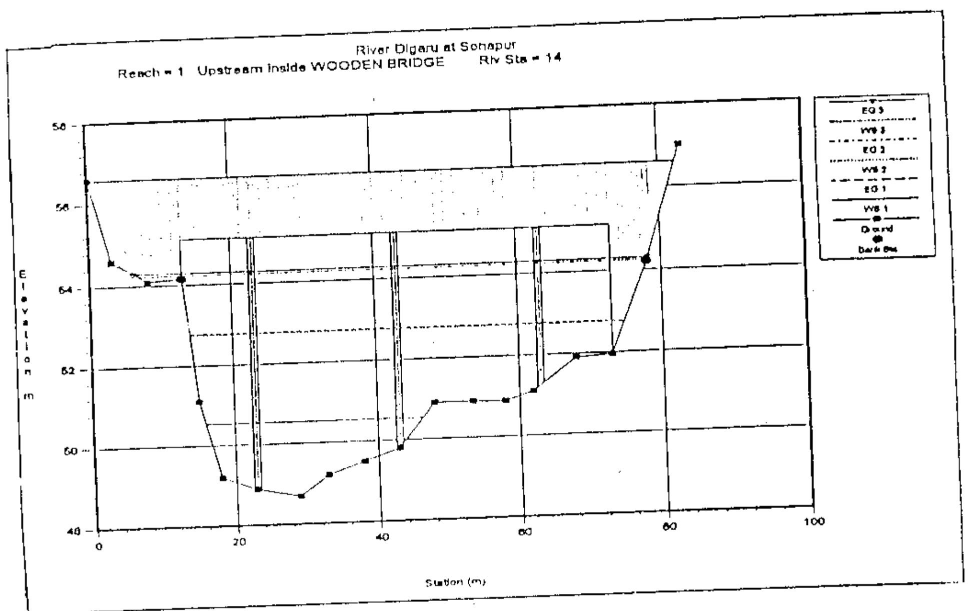


Figure 11(D) : Cross Sectional Plot

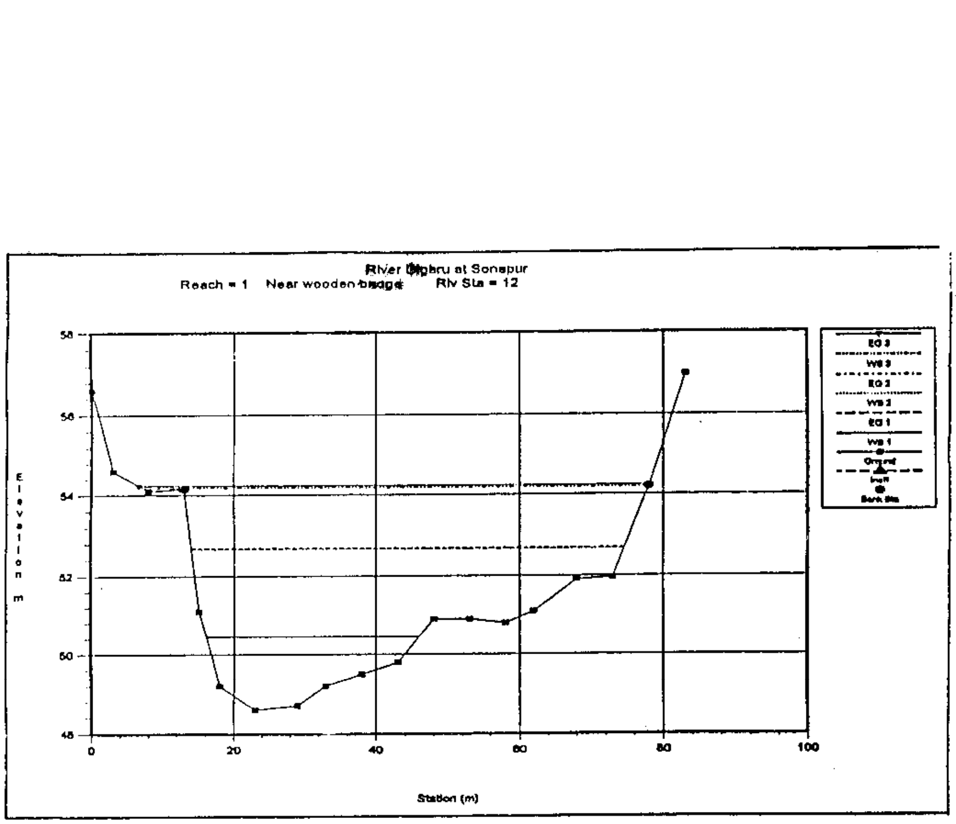


Figure 11(E) : Cross Sectional Plot

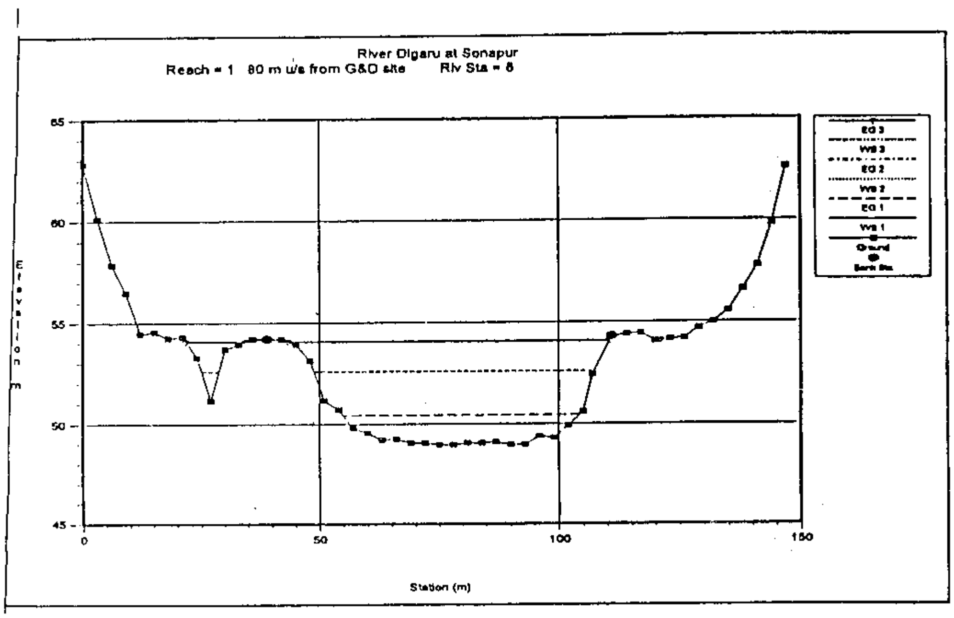


Figure 11(F) : Cross Sectional Plot

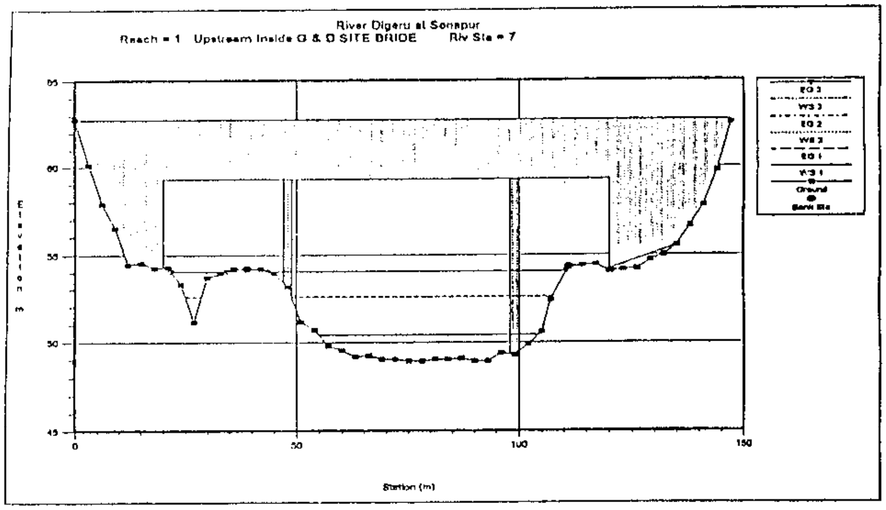


Figure 11(G) : Cross Sectional Plot

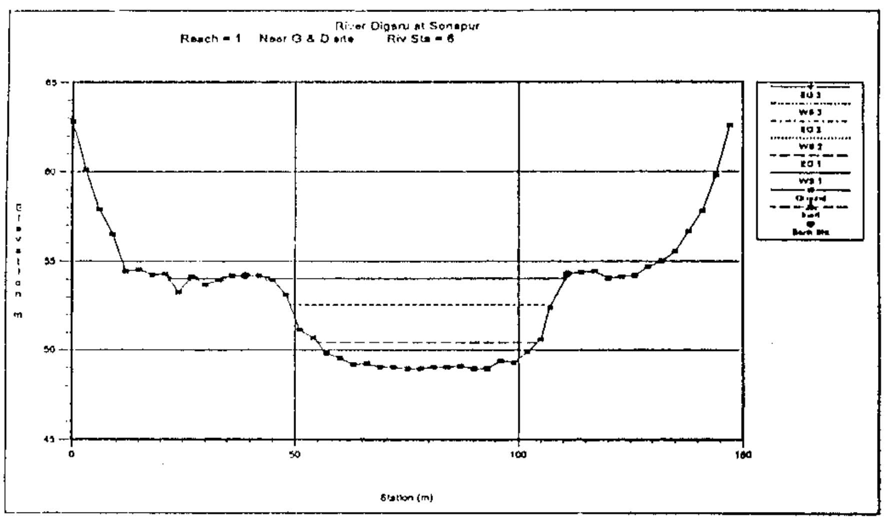


Figure 11(H) : Cross Sectional Plot

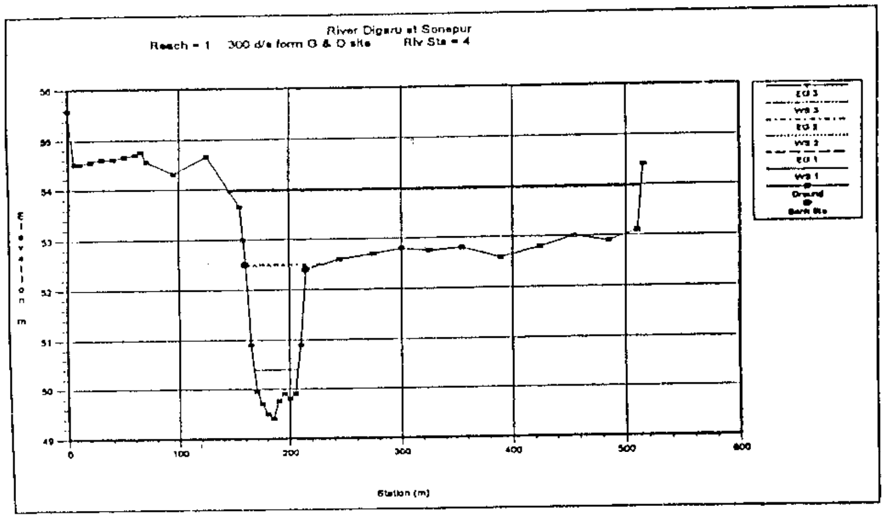


Figure 11(I) : Cross Sectional Plot

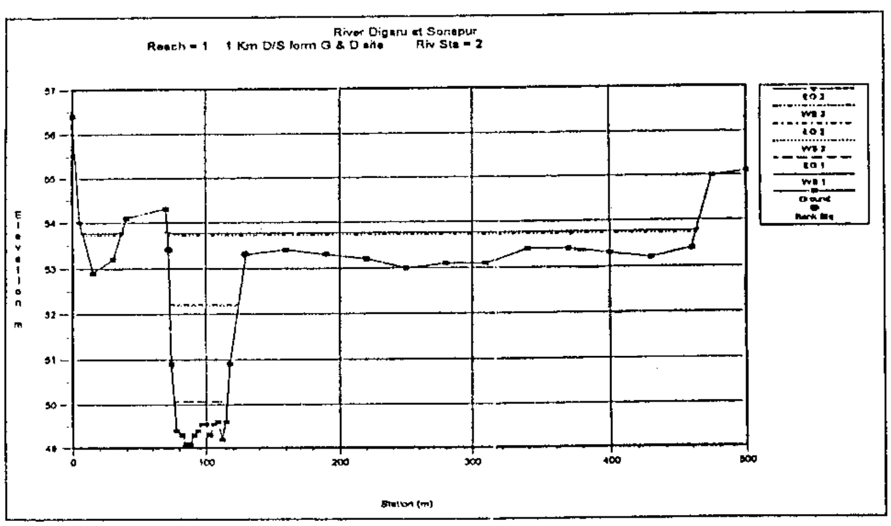


Figure 11(J) : Cross Sectional Plot

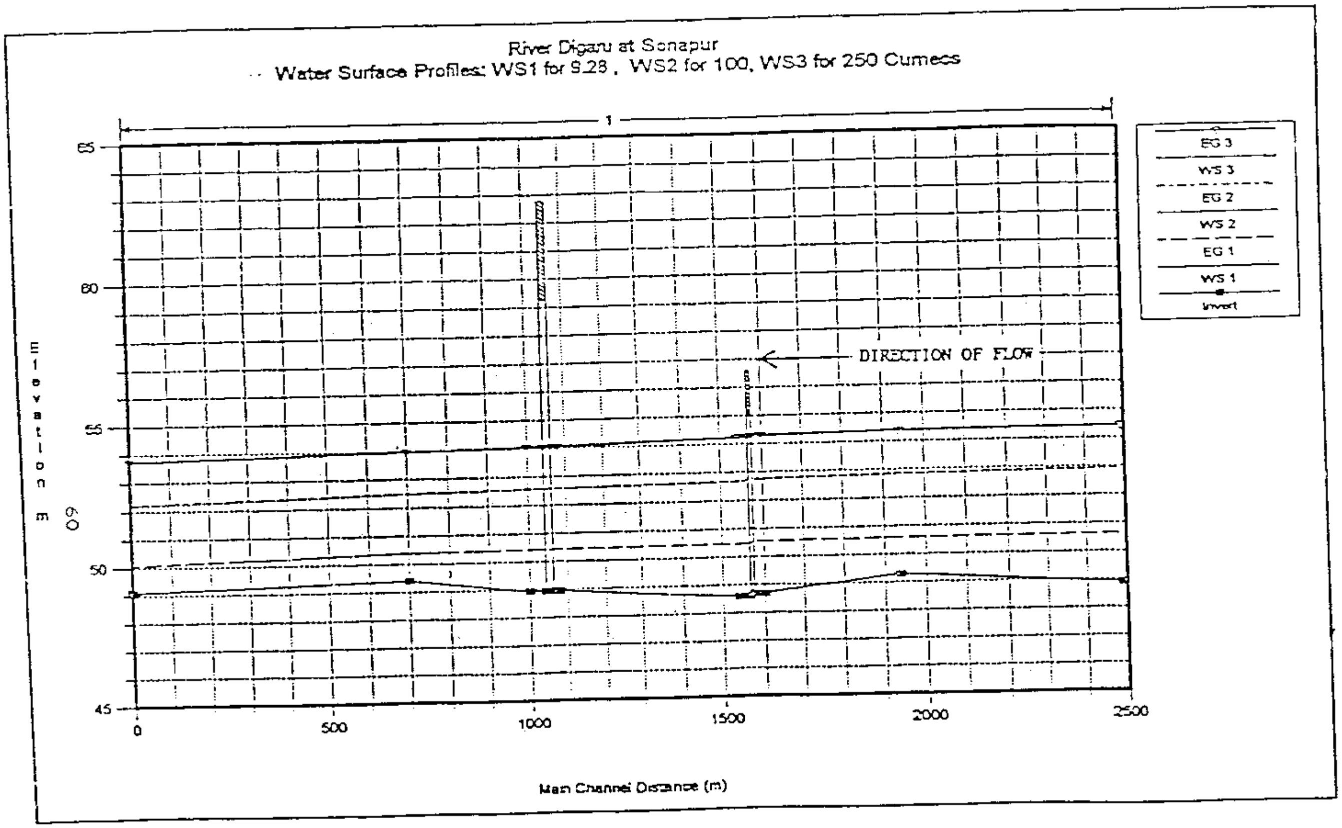


Figure 12 : Plot of Water Surface Profiles for the Reach

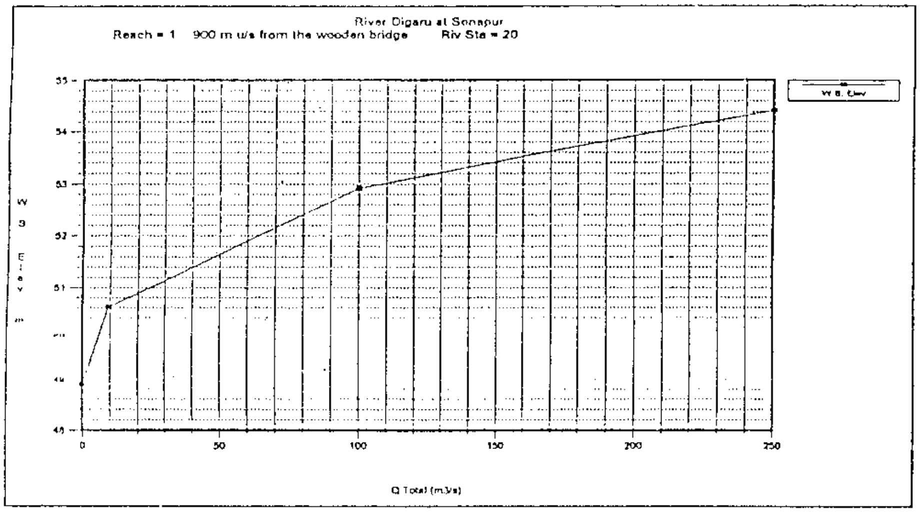


Figure 13(A) : Stage Vs Discharge Curve

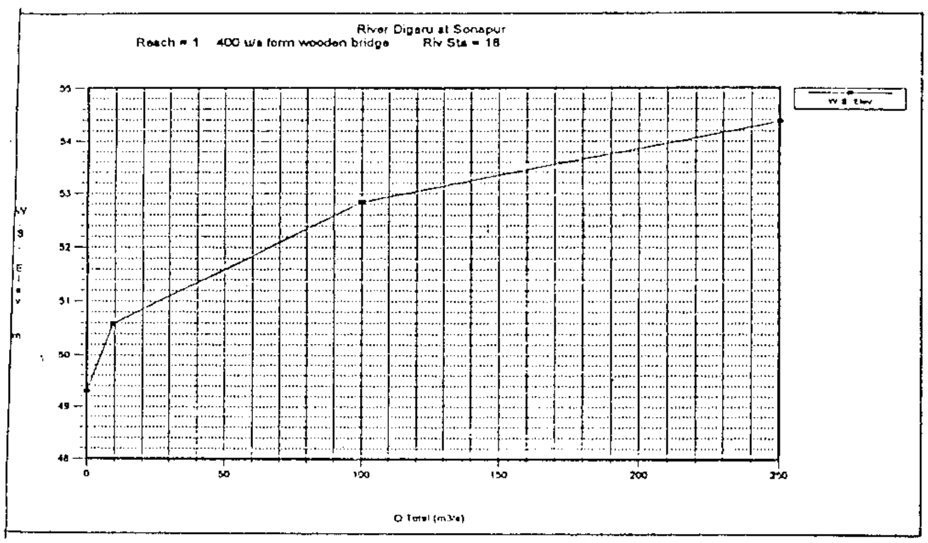


Figure 13(B) : Stage Vs Discharge Curve

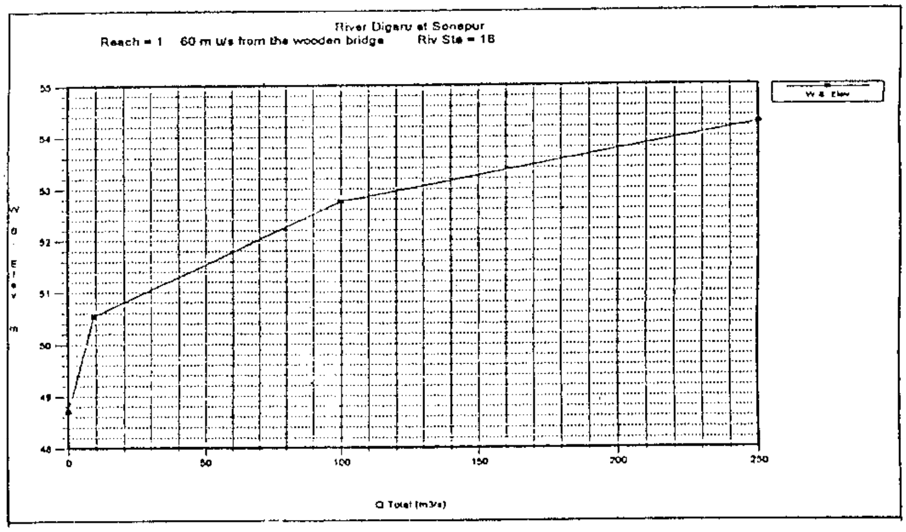


Figure 13(C) : Stage Vs Discharge Curve

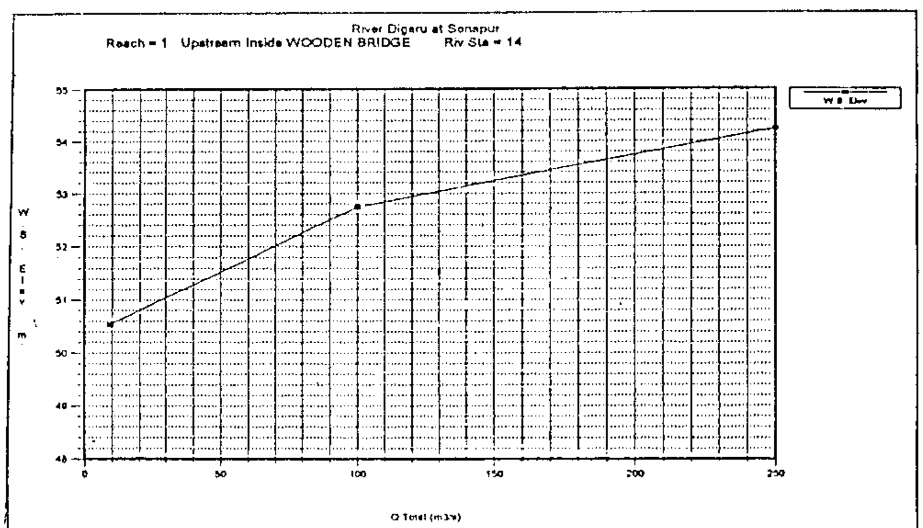


Figure 13(D) : Stage Vs Discharge Curve

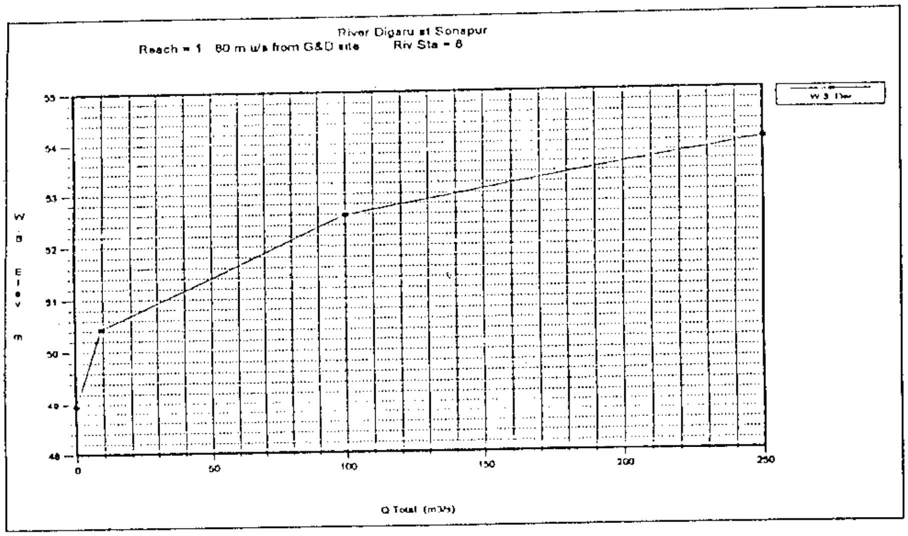


Figure 13(E) : Stage Vs Discharge Curve

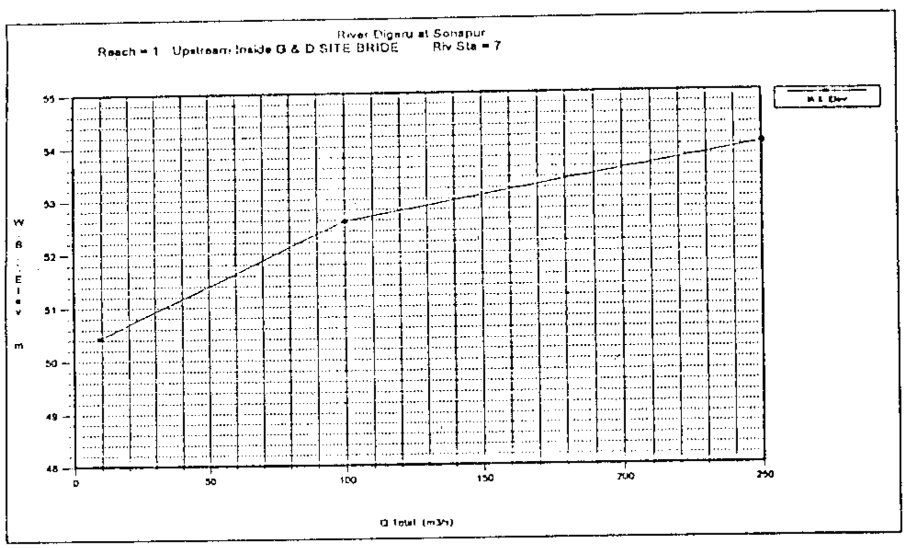


Figure 13(F) : Stage Vs Discharge Curve

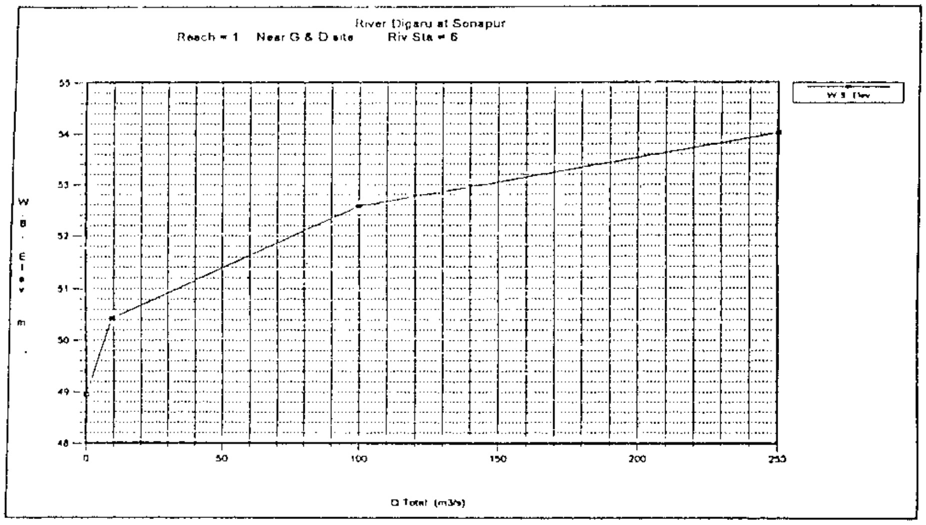


Figure 13(G) : Stage Vs Discharge Curve

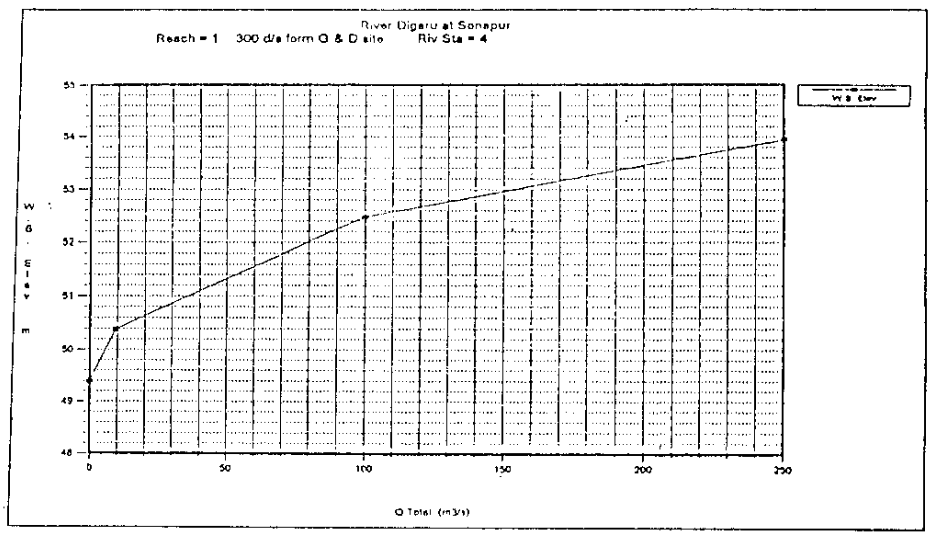


Figure 13(H) : Stage Vs Discharge Curve

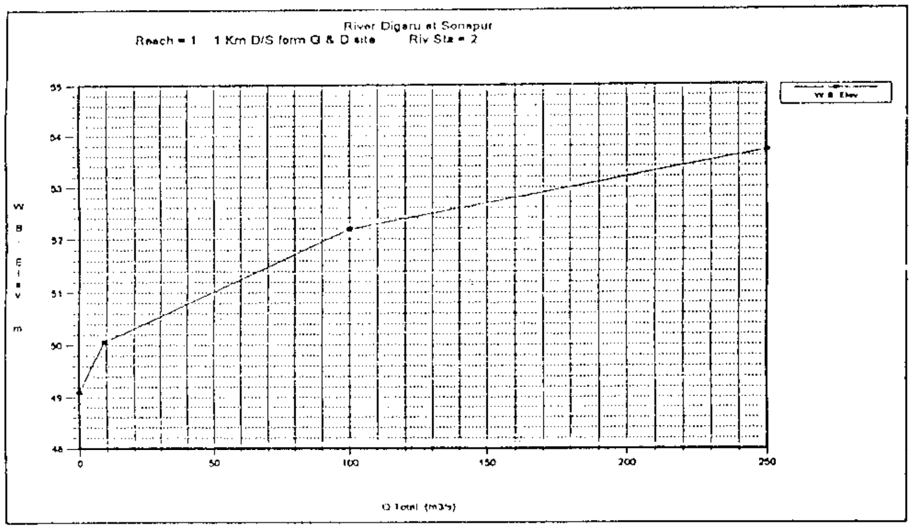


Figure 13(I) : Stage Vs Discharge Curve

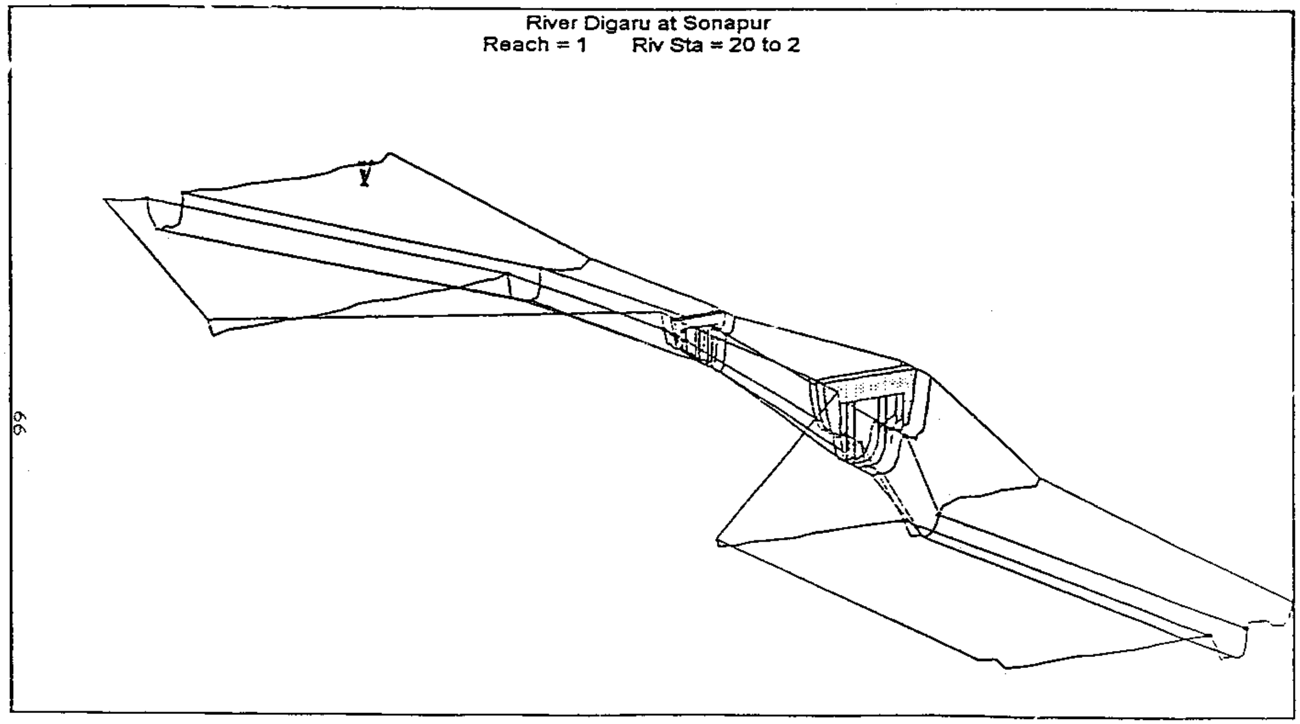


Figure 14 : Perspective Flood Plain of the Reach

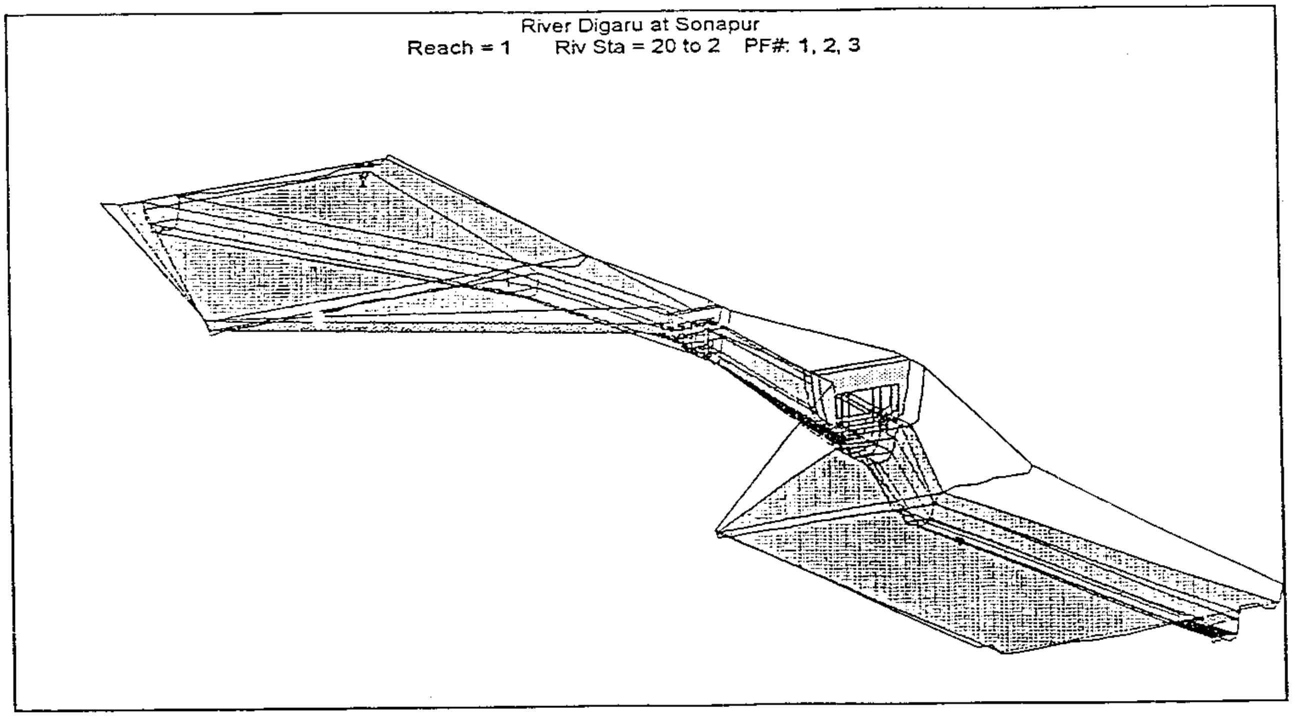
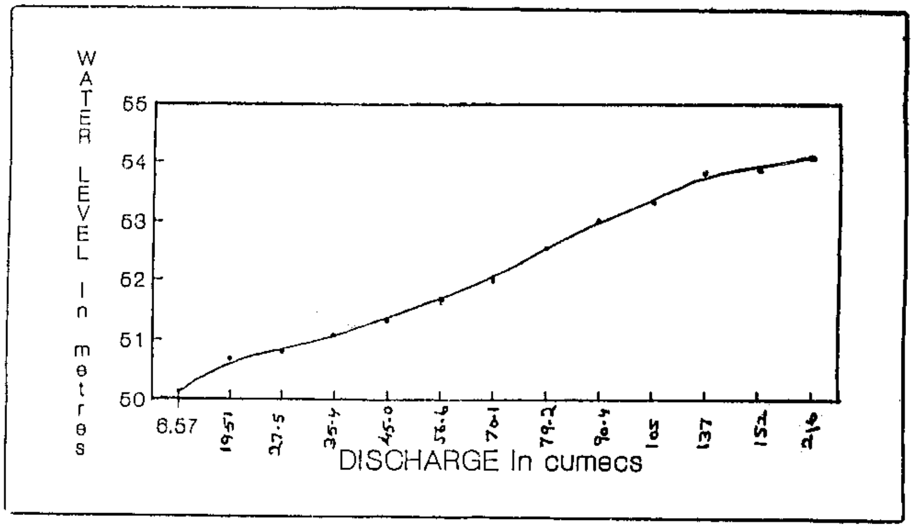
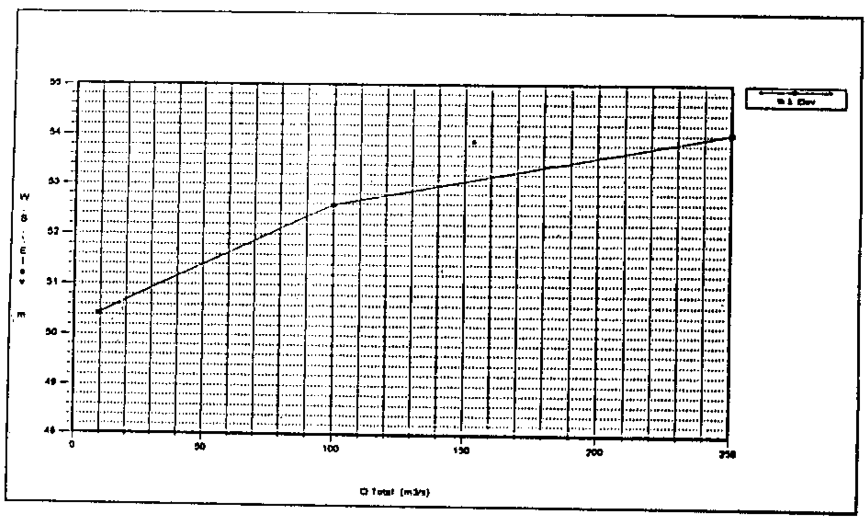


Figure 15 : Perspective Flood Plain of the Reach with Water Surface Profiles



(a) As Computed by CWC



(b) As Computed by HEC-2

Figure 16 : Stage Vs Discharge Curve for River Digaru at G & D Site at Sonapur

Profile Output : Table - 7

River Sta.	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # C
20	9.28	48.90	50.62	49.29	50.62	0.000030	0.15	63.02	48.21	0.
20	100.00	48.90	52.92	49.99	52.94	0.000121	0.53	187.51	72.13	0.
20	250.00	48.90	54.42	50.71	54.45	0.000145	0.73	575.51	435.29	0.
18	9.28	49.30	50.58		50.59	0.000134	0.24	38.50	43.39	0.
18	100.00	49.30	52.84		52.85	0.000186	0.61	241.87	286.00	0.
18	250.00	49.30	54.38		54.38	0.000074	0.50	1053.76	617.31	0.
16	9.28	48.70	50.54	49.27	50.55	0.000093	0.25	37.46	30.51	0.
16	100.00	48.70	52.75	50.33	52.78	0.000280	0.67	148.65	60.87	0.1
16	250.00	48.70	54.27	51.31	54.32	0.000372	1.02	244.97	71.79	0.1
14	Bridge									
12	9.28	48.60	50.46		50.46	0.000100	0.25	36.45	29.97	0.0
12	100.00	48.60	52.68		52.70	0.000296	0.68	146.00	60.64	0.1
12	250.00	48.60	54.18		54.24	0.000389	1.04	241.14	70.81	0.1
8	9.28	48.95	50.43	49.26	50.43	0.000042	0.16	57.64	49.37	0.0
8	100.00	48.95	52.59	49.97	52.60	0.000149	0.57	178.79	62.23	0.1
8	250.00	48.95	54.04	50.64	54.08	0.000278	0.93	277.85	78.84	0.1
7	Bridge									
6	9.28	48.95	50.43		50.43	0.000042	0.16	57.45	49.34	0.0
6	100.00	48.95	52.57		52.59	0.000152	0.57	175.26	58.49	0.1
6	250.00	48.95	54.01		54.05	0.000288	0.95	266.43	76.45	0.1
4	9.28	49.40	50.39		50.39	0.000517	0.37	24.78	39.74	0.1
4	100.00	49.40	52.48		52.51	0.000418	0.80	126.05	66.75	0.1
4	250.00	49.40	53.95		53.97	0.000216	0.80	561.93	367.13	0.1
2	9.28	49.10	50.06	49.54	50.07	0.000421	0.35	26.44	39.83	0.14
2	100.00	49.10	52.18	50.26	52.22	0.000420	0.81	122.85	51.45	0.1
2	250.00	49.10	53.71	50.99	53.76	0.000421	1.06	373.13	419.54	0.18

Profile Output : Table - 8

River Sta.	E.G. Elev (m)	W.S. Elev (m)	Vel Head (m)	Frctn Loss (m)	C & E Loss (m)	Q Left (m3/s)	Q Channel (m3/s)	Q Right (m3/s)	Top Width (m)
20	50.62	50.62	0.00	0.03	0.00		9.28		48.21
20	52.94	52.92	0.01	0.08	0.00		100.00		72.13
20	54.45	54.42	0.02	0.06	0.01	47.36	202.07	0.57	435.29
18	50.59	50.58	0.00	0.04	0.00		9.28		43.39
18	52.85	52.84	0.02	0.08	0.00		88.89	11.11	286.00
18	54.38	54.38	0.01	0.05	0.01	10.30	110.89	128.80	617.31
16	50.55	50.54	0.00				9.28		30.51
16	52.78	52.75	0.02				100.00		60.87
16	54.32	54.27	0.05			0.06	249.94	0.00	71.79
14	Bridge								
12	50.46	50.46	0.00	0.03	0.00		9.28		29.97
12	52.70	52.68	0.02	0.09	0.00		100.00		60.64
12	54.24	54.18	0.05	0.15	0.01	0.01	249.99		70.81
8	50.43	50.43	0.00	0.00	0.00		9.28		49.37
8	52.60	52.59	0.02	0.00	0.00	0.33	99.67		62.23
8	54.08	54.04	0.04	0.01	0.00	2.61	247.39		78.84
7	Bridge								
6	50.43	50.43	0.00	0.03	0.00		9.28		49.34
6	52.59	52.51	0.02	0.07	0.00		100.00		58.49
6	54.05	54.01	0.05	0.07	0.01	0.26	249.74		76.45
4	50.39	50.39	0.01	0.33	0.00		9.28		39.74
4	52.51	52.48	0.03	0.29	0.00		99.98	0.02	66.75
4	53.97	53.95	0.02	0.20	0.01	0.66	164.18	85.16	367.13
2	50.07	50.06	0.01	0.00	0.00		9.28		39.83
2	52.22	52.18	0.03	0.00	0.00		100.00		51.45
2	53.76	53.71	0.05	0.00	0.00	2.78	219.93	27.30	419.54

Profile Output : Table - 9

River Sta.	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)
20	9.28	48.90	50.62
20	100.00	48.90	52.92
20	250.00	48.90	54.42
18	9.28	49.30	50.58
18	100.00	49.30	52.84
18	250.00	49.30	54.38
16	9.28	48.70	50.54
16	100.00	48.70	52.75
16	250.00	48.70	54.27
14	Bridge		
12	9.28	48.60	50.46
12	100.00	48.60	52.68
12	250.00	48.60	54.18
8	9.28	48.95	50.43
8	100.00	48.95	52.59
8	250.00	48.95	54.04
7	Bridge		
6	9.28	48.95	50.43
6	100.00	48.95	52.57
6	250.00	48.95	54.01
4	9.28	49.40	50.39
4	100.00	49.40	52.48
4	250.00	49.40	53.95
2	9.28	49.10	50.06
2	100.00	49.10	52.18
2	250.00	49.10	53.71

Profile Output (Bridge Only) : Table - 10

River Sta.	E.G. Elev (m)	Min El Prs (m)	Opening Area (m2)	Prs O WS (m)	Q Total (m3/s)	Min Top Rd (m)	Q Weir (m3/s)	Delta EG (m)
14	50.55	55.10	272.82		9.28	56.60		0.09
14	52.78	55.10	272.82		100.00	56.60		0.08
14	54.32	55.10	272.82		250.00	56.60		0.08
7	50.43	59.28	769.58		9.28	62.78		0.00
7	52.60	59.28	769.58		100.00	62.78		0.01
7	54.08	59.28	769.58		250.00	62.78		0.03

Profile Output (Bridge Comparison) : Table - 11

River Sta.	E.G. Elev (m)	W.S. Elev (m)	Energy EG (m)	Momen. EG (m)	Yarnell EG (m)	WSPRO EG (m)	Prs O EG (m)	Prs/Wr EG (m)
14	50.55	50.54	50.47	50.55	50.46			
14	52.78	52.75	52.73	52.78	52.70			
14	54.32	54.27	54.28	54.32	54.25			
7	50.43	50.43	50.43	50.43	50.43			
7	52.60	52.59	52.60	52.59	52.59			
7	54.08	54.04	54.08	54.05	54.06			

4.0 DISCREPANCIES AND CORRECTION

The programme implemented on PC system and applied to a small river Digaru in Brhamputra Basin, is capable of computing water levels at various sections along the river. The pertinent hydraulic parameters determined are depth of flow, water surface width, water surface elevation, elevations of the total energy line, friction slope, flow velocity, critical depth and volume beneath the computed profile.

Flow under a bridge is a complex hydraulic process which is approximated by one dimensional equations and certain discharge coefficients. HEC-2 handles the complex situation by two SUB-ROUTINES. This has been tested for the case of Indian rivers as applied to river Digaru.

A secondary purpose of HEC-2 is to calculate the permissible encroachment width for specified discharges. This capability also needs to be tested so that its use can be made for flood plain zoning with additional information from topographic maps.

HEC-2 linked with sediment transport routine are useful for alluvial rivers. Further the programme HEC-2 (i) is very long, contains more than 8000 statements, (ii) computations using this general purpose programme is high for simple water surface profile computations, (iii) the input data preparation is time consuming and error prone which could be avoided by proper change in the programme. This change has been accomplished by the programme, "River Analysis System", micro soft window version of HEC-2. For the programme "River Analysis System" input of data is convenient, easy and error free as compared to HEC-2.

5.0 RECOMMENDATIONS

HEC-2 gives water surface profile by considering flood plain, roughness of main channel as well as flood plain. The computation can be down either for sub-critical or super critical flow. The computed results are more or less the same as the observed one. Hence HEC-2 can be directly applied in the field use.

6.0 REFERENCES

1. Eichat Bill S and John C. Peters, (1974), 'Computer determination of flow through bridges', Tech. paper 20, US Army Corps of Engineers.
2. HEC-2 (1981) Water Surface Profile, User's Manual.
3. HEC-2 (1981) Programmer's Manual.
4. HEC-2 (1974) Computations of water surface profiles through bridges using HEC-2.
5. Nagler, Floyd, A. (1978), 'Obstruction of bridge piers to the flow of water', A.S.C.E. Trans. Vol. 82, pp. 334-343.
6. William G. Westall, (1981), ' A back water programme for th G.E. 225 computer', paper no.5, Proc. of Seminar on computer applications in Hydrology. The HEC, Davis, California.
7. Yarnell, David L. and T.A> Nagler, (1930), Flow of flooa watei over Railway and Highway Embankments', Public Roads, Vol. 11, no. 2, Apl., pp. 30-34.
8. Miller, W.A. and Yevjenich, V., (1975), "Unsteady flow in open channels", Bibliography, Vol. II, Water Resources Publications, Fort Collins, Colaro-do.

9. Technical Report No. RN-21, 1985-86 of National Institute of Hydrology, "Effect of floodplain on flood routing"

10. Technical Report No. CS-13, 1985-86 of National Institute of Hydrology, "Application of muskingum-cunge method of flood routing".

7.0 ACKNOWLEDGEMENT

The'author express his gratitude to Dr. S.M. Seth, Director, National Institute of Hydrology for allowing me to work on this project. I wish to express my appreciation for Dr. K.K.S. Bhatia, Sc. 'F' and Coordinator, NERC, Guwahati and Sh. B.C. Patwary, Sc. 'E" and Head, NERC for their generous guidance and assistance in preparing this report. I owe debt to Central Water Commission at Guwahati for providing the data of river Digrau and helping in conducting cross sectional survey of river Digaru at Sonapur.

Thanks to Sh. Pradeep Kanti Sarkar, R.A. for conducting cross-sectional survey of the river and to Sh. C.S. Chauhan, Tech. for helping in preparing this report. Finally, the author thanks all his friends who helped in one way or other in my this endeavor.

DIRECTOR	:	Dr. S.M. SETH
DIVISIONAL HEAD	:	Sh. B.C. PATWARY
STUDY GROUP	:	Sh. V.K. DWIVEDI
ASSISTANTS	:	Sh. P.K. SARKAR