CASE STUDY CS (AR) 156

FLOOD PROTECTION STUDIES USING HEC-2 MODEL ON RIVER TAWI NEAR JAMMU BRIDGE SITE

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PREFACE

The state of J & K is blessed with abundant water resources from both rain and snowmelt fed rivers. These rivers often bring floods and misery. Therefore the river hydraulics/ hydrology need to be understood properly before undertaking flood protection measures. Ever since the establishment of the Western Himalayan Regional Centre of the National Institute of Hydrology, the scientists of the centre have been closely interacting with the senior officers of various departments in the state, on hydrological problems.

The Tawi river in the Jammu region of the state is also prone to floods in the Monsoon season. During Sep 1988, an unprecedented flood estimated at 12000 cumecs (4.2 lac cusecs) passed through Jammu, breaching and overtopping the right bank upstream of city bridge, submerging low lying areas in the old city and causing extensive damage to life and property. In the present study, the flood affected reach of about 1.5 Km near the bridge site was selected to analyse the river hydraulics, estimate the flood (1988), its profile and flood plain using HEC-2 model of the US Army Corps of Engineers USA). A model developed rating curve at the bridge (GD site of CWC) is presented. Channel improvement methods using the model are also suggested to contain the flood in general.

The report was prepared by Sh S V N Rao, Scientist C, under the guidance of Dr. K S Ramasastri, Sc F and Coordinator. Sh Puran Singh, and Sh V K Agrawal, Research Assistants helped in the study.

(S.M. SETH)
Director

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ABSTRACT

The low lying areas on the right bank of river Tawi (Chenab river system) upstream of bridge site at Jammu have been often affected by floods. Existing protection works are not adequate to accommodate the floods of magnitude of Sep 1988 estimated at 12000 cumecs (4.2 lac cusecs) or 100 year flood (design capacity of bridge) which is 9600 cumecs (3.4 lac cusecs). Water surface profiles using recent cross section data along a reach of 1.5 Kms near the bridge site at Jammu were computed using the model. The model capability for design of protective works such as embankments, levees, flood walls is illustrated. Channel improvement to confine the floodwaters to the river channel are suggested. Model developed rating curve at the gauge site (Jammu bridge) is also presented.

1.0 INTRODUCTION

River Tawi is one of the major tributaries of the river Chenab in the Sivaliks of the Himalayas. The river with a catchment of over 2000 sqkms. flows through the city of Jammu for nearly 2 Kms. A large part of the catchment is either degraded or barren (Jain, 1992) resulting in floods laden with heavy loads of sediment/ debris that cause extensive damage in the flood plains along the banks of the river. During Sep 1988 an estimated flood of 4.6 lac cusecs passed through the bridge at Jammu almost touching the bottom of the bridge deck endangering the structure and causing extensive damage to life and property of the inhabitants along both the banks and/in the low lying areas with in the city.

In the present study HEC-2 model of the US Army Corps of Engineers, (1990) is utilised to study the hydraulics of the river and simulate the profiles of a 100 yr flood and an unprecedented flood during Sept 1988 along a reach near Jammu, estimate their flood plains and suggest channel improvement measures to contain the flood, besides ensuring the bridge safety. The model being versatile offers several options to help in the design of protective works for effective flood control and flood management. A model developed rating curve for the control structure (bridge site) presently maintained by CWC Jammu is also presented.

2.0 REVIEW

Our current knowledge of river hydraulics is largely based on the theory of uniform flow and Manning's equation. River streams with gradual change in depth and velocity (i.e on relatively flat gradients) may however be analysed numerically. The energy equation is applied to a differential control volume and the resulting equation relates change in depth to distance along the flow path. Thus a nonuniform flow problem is assumed to be approximated by a series of uniform flow segments. This is generally referred as the standard step or the backwater curve method used by most numerical models including the HEC-2. Other similar models for profile computations include stepbackwater model J635 (Sherman, 1987) and SCS's WSP2 (1976) etc.

Most step backwater flow modelling procedures require input parameters of energy loss coefficients, stage and discharge. The initial stage choice is of little consequence, since profiles generated at equal discharge will converge after a few computed steps. Moreover several studies have demonstrated that step backwater flow modelling of relatively deep, rare flood is remarkably insensitive to reasonable errors in estimating either Mannings N or the expansion/ contraction coefficients (Dawdy and Moteyed, 1979).

Jarret and Malde (1987) used the backwater computation technique to estimate the paleo flood and flood plain at Bonneville flood, of the Snake river, Idaho (USA). Mishra and Ramasas-

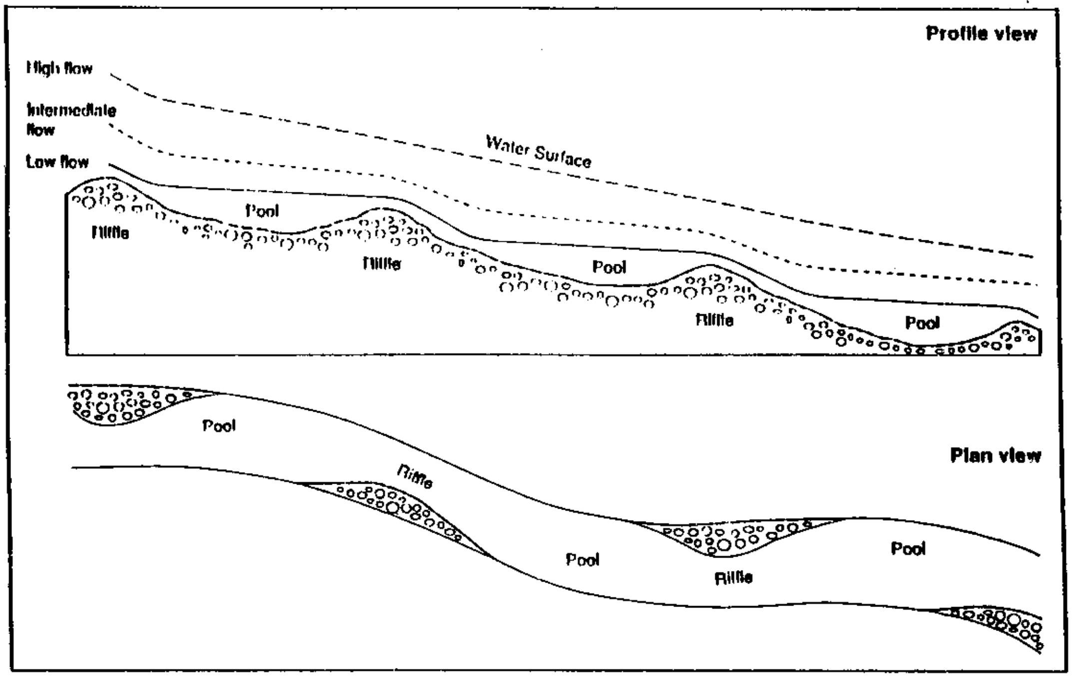

Figure 1: Diagram of longitudinal profile and plan view of a pool-riffle sequence. Water surface profiles in upper figure represent high, intermediate, and low flow conditions.

tri (1992) have used HEC-2 model to determine paleo floods in river Narmada.

Natural stream profile can not have a constant energy gradient along the reach. This is due to changes in cross section and roughness characteristics of the underlying bed material. The profile may be diagrammatically represented for various flood magnitudes as in fig 1 (after Grant, 1992). The Mannings coefficient for various bed conditions have been discussed at length by Chow (1959), Dalrymple and Benson (1967), Barnes (1967) and Alridge and Garret (1973). Several researchers have also tried to relate bed material particle size distribution with Mannings N and other hydraulic characteristics (Thorne and Zevenberger, 1985), Jarrel (1990) etc. Extensive research work in this direction has been carried out by Jarret (1984, 1987, 1990) of the USGS on natural streams in USA. He found that Mannings n varies inversely with depth and directly with slope and that flow conditions were subcritical even in steep bed slopes (upto 0.02). He established a relation between hydraulic radius, energy slope and Mannings 'n'. Dobbie & Wolf (1953) and Jarret (1987) recommended critical depth method rather than slope area method for peak flood estimation at sections where critical depth would have most likely occurred. Critical sites could be relatively deep constricted sites such as bridges or other control structures.

3.0 THEORETICAL BASIS OF HEC-2 MODEL

3.1 General:

The flood profile of a given recurrence interval is obtained by the solution of one dimensional energy equation. The details are explained in HEC-2 users manual (1990a), Bedient (1989).

3.2 Equation for basic profile calculation:

The following equations are solved iteratively, by assuming starting condition of subcritical (or supercritical) flow for upstream (subscript 1) and downstream (subscript 2) sections.

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = WS_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \quad \text{.............. (1)}$$

$$h_e = L\bar{S}_f + C\left|\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right| \quad \text{........... (2)}$$

Where,

WS_1 , WS_2 = Water surface elevations at ends of reach

V_1 V_2 = Mean velocities (total discharge/ total flow area) at ends of reach,

α_1 , α_2 = Velocity coefficients for flow at ends of reach,

g = gravitational constant,

h = Energy head loss,

L = Discharge weighted reach length

$\bar{S}_f$ = representative friction slope for reach, C = Expansion or contraction loss coefficient.

The discharge weighted reach length L is computed by weight lengths is the left overbank, channel and right overbank with their respective flows at the end of the reach. A representative friction in HEC-2 is usually expressed as follows, although alternative equations can be used;

$$\bar{S}_f = \left(\frac{Q_1 + Q_2}{K_1 + K_2}\right)^2 \qquad \text{..........} \quad (3)$$

where K and K and Q and Q represent the conveyance and discharge at the beginning and end of reach. Conveyance is defined from Manning's equation as

$$K = 1/N * A\ R^{2/3} \qquad \text{............} \quad (4)$$

Where,

K = Conveyance or Hydraulic capacity

N = Manning's Coefficient

A = Cross sectional area

R = Hydraulic radius.

The total conveyance for a cross section is obtained by summing the conveyance from the left and right overbanks and the channel. The velocity coefficient is obtained from the equation:

$$\alpha = \left(\frac{A_T^2}{K_T^3}\right)\left(\frac{K_{LOB}^3}{A_{LOB}^2} + \frac{K_{CH}^3}{A_{CH}^2} + \frac{K_{ROB}^3}{A_{ROB}^2}\right) \qquad \text{...........} \quad (5)$$

Where the subscript T is for cross sectional total, LOB is for left overbank CH is for channel, and ROB is for the right overbank.

3.3 Computational procedure:

The computational procedure for iterative solution of equations (1) and (2) is as follows;

1. Assume a water surface elevation at the upstream cross section (or downstream cross section if a supercritical profile is being computed).
2. Based on the assumed water surface elevation, determine the corresponding total conveyance and the velocity head.
3. With the values from step 2, compute representative friction slope $\bar{S}_f$ and solve equation 2 for head loss h_e
4. With values for steps 2 and 3, solve equation 1 for WS_2
5. Compare the computed values of WS_2 with the values assumed in step 1; repeat steps 1 through 5 until the values agree to within 0.1 metres.

The first iterative trial is based on the friction slope from the previous two cross sections. The second trial is an average of the computed and assumed elevations from the first trial. Once a balanced water surface has been obtained from a cross section, checks are made to be sure that the elevation is on the correct side of the critical water surface elevation. If otherwise, critical depth is assumed and a message to that effect is provided. The occurrence of critical depth in the program is usually the result of a problem with reach lengths or flow areas unless a critical condition actually occurs.

3.4 Model Assumptions:

The following assumptions are implicit in the equations and procedures used in the program.

1. Flow is steady
2. Flow is gradually varied
3. Flow is one dimensional
4. River channels have small slopes (less than 1 : 10).

If any of these assumptions are violated, the results from the HEC-2 program may be in error.

3.5 Model Capabilities:

The HEC-2 is a versatile model for flood studies. Several options available may be utilised to exploit the models capabilities. The important ones include:

1. Computation of multiple water surface profiles and hence the rating curves at any desired cross section.
2. Modelling existing hydraulic structures such as bridges and culverts to estimate head losses by special and normal bridge/ culvert methods in a given reach.
3. Carrying out channel improvement to lower flood peaks.

4. Compute Mannings's N in a given reach.

5. Design of protection works using several encroachment options for floodway analysis.

6. Estimation of Paleofloods.

4.0 PROBLEM DEFINITION

The river Tawi reach near Jammu upstream of bridge site is not adequately protected along its banks. Severe floods submerge the low lying areas (especially the right bank) affecting life and property. An unprecedented flood estimated at 4.6 lac cusecs passed through Jammu during Sept 1988 causing extensive damage due to a breach in the right bank and submerged the lowlying areas including the old city. The present study is intended to obtain a realistic estimate of this flood by simulating the flood profile using HEC-2 model, determine its flood plain and suggest channel improvement methods to contain the flood for effective flood control and flood management.

The bridge site acts as a control structure (CWC, Jammu) with rating curves developed using velocity measurements by float method. Hence their is a need for more reliable rating curve which could be developed using the model.

5.0 DESCRIPTION OF STUDY AREA

5.1 General:

The Tawi river catchment upto the bridge point at Jammu encompasses an area of over 2000 sq kms and lies between 74° - 35' and 75°- 45' east longitudes and 32°35' and 33° - 5' north latitudes (fig 2). The entire catchment is mountainous comprising hills in the upper, middle and lower Sivaliks of the Himalayan ranges. The entire river profile is steep with altitudes ranging from 4000 m to nearly 400 m near Jammu. From Jammu the river is relatively on flat river plains that are contagious upto Punjab . The catchment receives a rainfall of over 100 cms mostly during monsoon season and the remaining during snow season in the upper reaches of the catchment. The discharge in the river at the bridge site is 300 - 400 cusecs perennially due to snow melt. Monsoon flows have been recorded often above 1 lac cusecs. High flows from 2.2 to 2.4 lac cusecs were experienced during 1950 and 1957. An all time high record estimated at 4.6 lac cusecs exceeding the design capacity of bridge (100 yr flood of 3.4 lac cusecs) was observed during the storm of Sep 1988. The various aspects of this storm have been dealt in detail by Ramasastri (1992). The hydrology, geomorphology, landuse and network requirement for Tawi basin have been discussed at length by Patwary (1991, 1993), Jain (1992) and Ramasastri (1993) respectively.

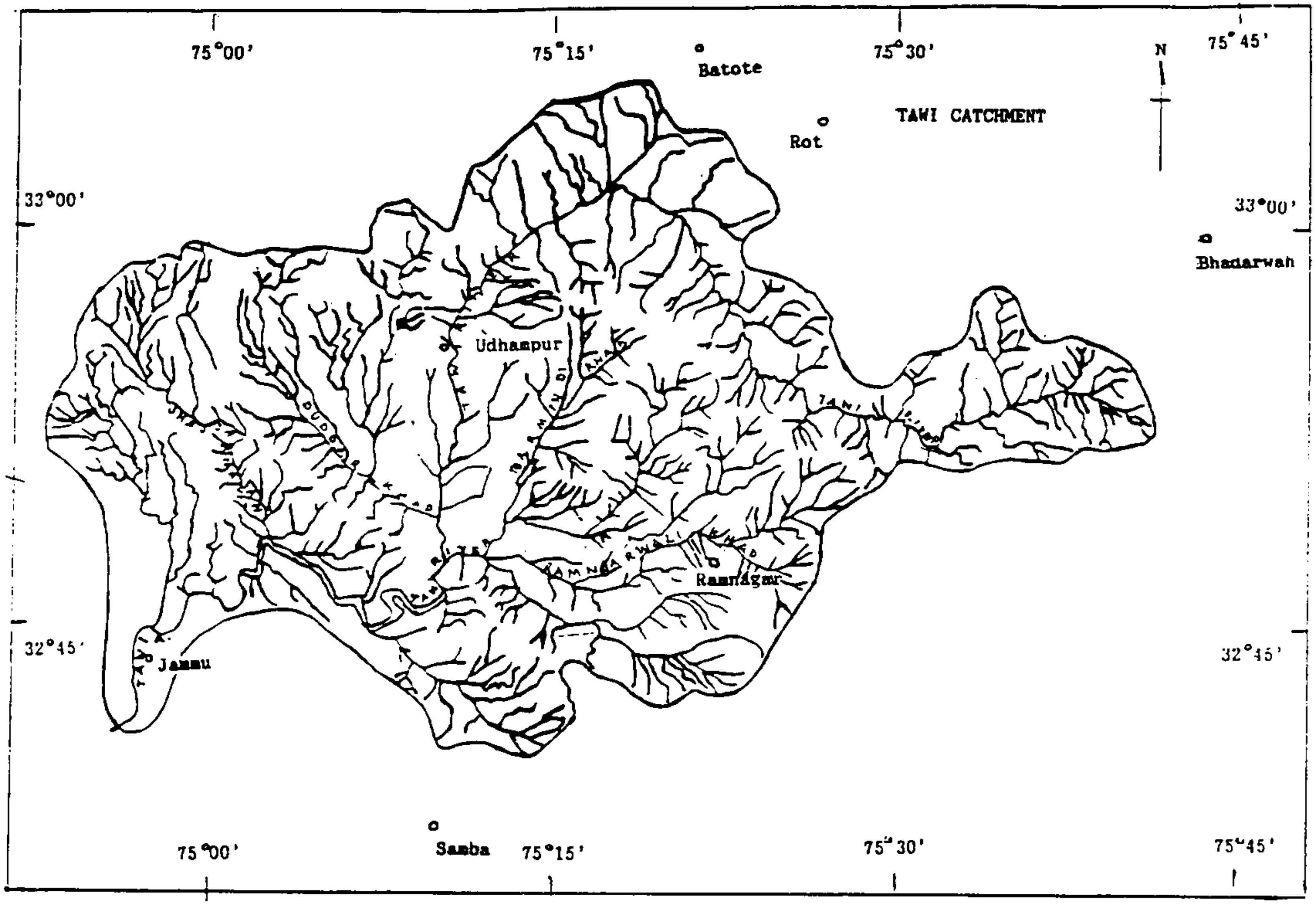

Figure 2 : Catchment map of Tawi river and streams

5.2 Catchment topography, land use and Flood problem:

The catchment consist mostly of broken country of low Shivaliks composed of boulders and debris and sparsely vegetated lands consisting of thorny bushes and brush wood. Jain (1992) has reported more than 60 % land either degraded or barren using remotely sensed data (see fig 3). The catchment by this very nature offers little or no resistance to active denudation. This is further aggravated by merciless cutting of bushes and trees for fuel and also uncontrolled grazing by the cattle and the extension of cultivation on the slopes. The denudation of the soil cover results in quick erosion and formation of deep gullies on the hill slopes. The sediment load carried by the stream is quite heavy. The heavy load of the sediment is safely carried down by the stream in the initial stages where the grades are steeper. These include silt, sand, gravel, cobble, shingle and boulders. However, when the streams emerge out of the gorges on to the plains, slopes flatten and velocities get reduced and the stream deposits the heavy debris chocking their normal water ways. The subsequent freshlets cannot push down the heavy debris and erode the softer soil on the banks to make room for stream flows. The streams therefore get widened, flow becomes braided with channels getting shifted with each new flood. The mean width of river Tawi near jammu is about 350 meters. It will thus be seen from what is said above that flood problem lies mainly in the lower portions near Jammu city due to increase in width, topography and easing of grades (I & FC report, 1988a).

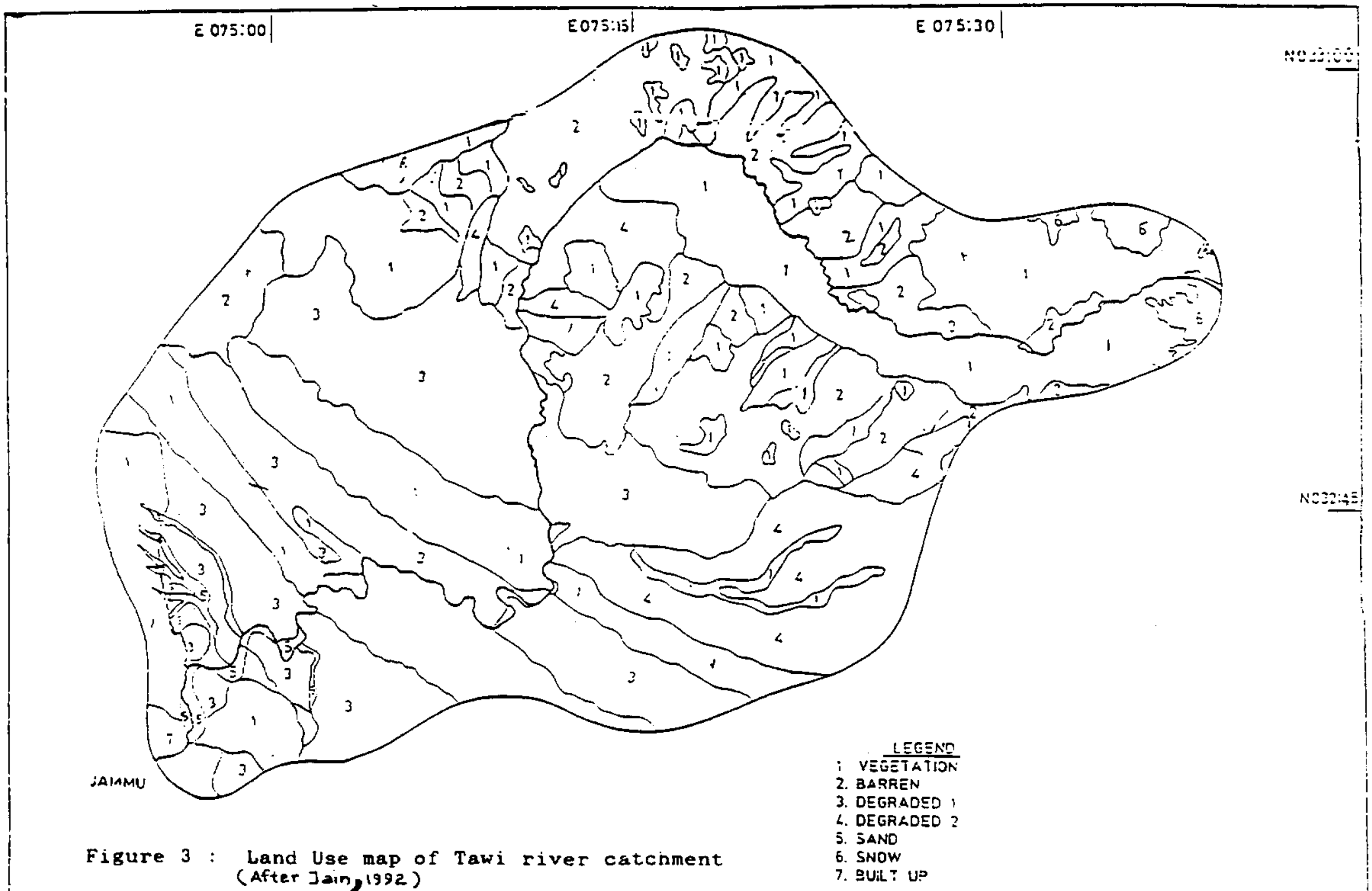

Figure 3 : Land Use map of Tawi river catchment
(After Jain, 1992)

5.3 The Flood of Sept 1988:

An unusual storm during 24 & 25 Sep 1988 resulted in a flood estimated at 4.6 lac cusecs in Tawi river at Jammu near bridge site. This flood exceeded the design capacity of bridge of 3.4 lacs cusecs and almost touched the bottom of the bridge endangering the structure. The HFL of the flood has been recorded as 304.3 metres at the bridge point by CWC Jammu. The flood also inundated the low lying areas causing extensive damage to life and property along both the banks. The right bank 400 M upstream of bridge breached in a length of 230 M which apart from destroying Qasim nagar habitation submerged large areas of exhibition, Vinayak bazar, Jewel chowk, Krishnanagar, University (old Campus) where heavy damages to establishments and residential areas took place. Ground floors of most of the houses and shops remained under 1.1 M of water. Huge treasures of rare library books and sophisticated machinery and equipments in various departments of University were spoiled. Similarly left bund breached in a length of 290 M but due to high banks not much damage to public property took place. Toe part of the embankment along both banks of river down stream of bridge got partly damaged upto canal crossing point about 1 Km down stream of bridge. The island portion carrying open canal section was over run and made flush with river bed completely disrupting the irrigation supplies of large cultivated area in R S Pura and Bishnah tehsils (Irrigation and Flood Control report, 1988b).

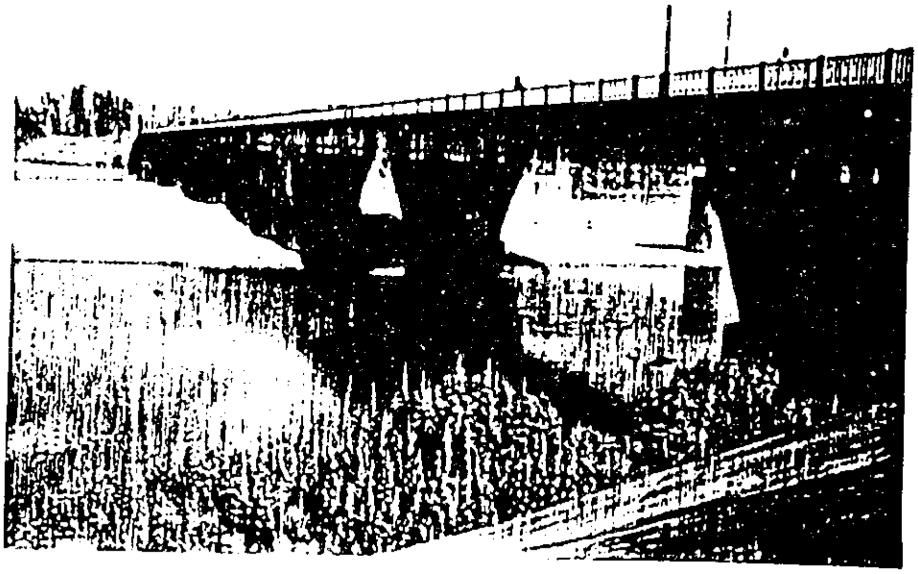

Figure 4: CWC gauge discharge site at Jammu

5.4 Study Area:

The Tawi river (see fig 4) reach of about 1.5 kms (one km U/S and half a km D/S of bridge site) adjoining the city of Jammu which directly affects the urban population due to floods was selected for study. The average width of river here is about 350 meters. The bed material includes silt, sand, gravel and boulders but mainly consists of silt and sand. The size of largest boulder was observed to be about 15 cms and that of gravel as 5cms (average size is 3 - 4 cms). Due to high ground in the mid of the river bed near bridge site (2-lane bridge supported on 7 piers) the river flows as two divided channels for low flow conditions (see fig 5).

The bridge site acts as a control section due to constriction and hydraulic jump (critical depth) is evident during high

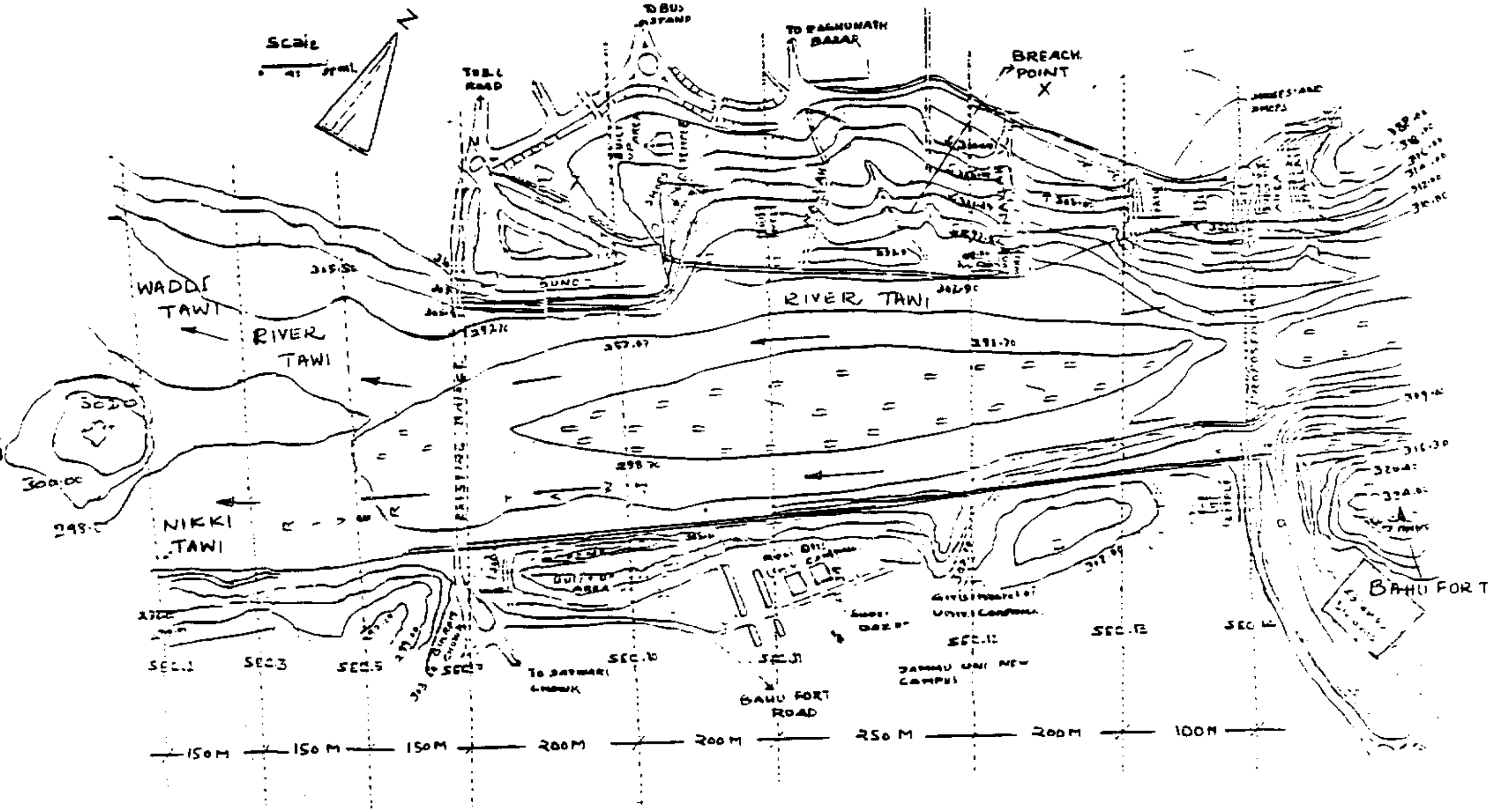

FIG : 5. RIVER TAWI NEAR BRIDGE SITE JAMMU
(STUDY AREA)

flows. The CWC has therefore located its gauge scale on the down stream side of the piers. The state Irrigation and Flood Control deptt also maintains a gauge discharge site about 50 M upstream of the bridge. Both the GD sites are supported on stable concrete embankments of uniform sections with little scour. The inhabitants of the low lying areas (including the old city, University upto canal road) adjacent to right side of the bank upstream of bridge are most vulnerable to floods. These areas were badly inundated during the Sep 1988 flood.

6.0 METHODOLOGY AND DATA COLLECTION:

The methodology and field investigations for the study were designed to keep consistency between data requirements of the model, actual data available and the objectives of the study. These are briefly discussed here:

6.1 Study Objectives:

1. Develop a rating curve at the Jammu bridge control point.
2. Obtain a realistic estimate of the Sep 1988 flood and estimate its flood profile and flood plain. Also obtain the profile of 100 Yr flood.
3. Suggest channel improvement/ protection measures to contain floods.

6.2 Model requirements:

These include the following:

1) Discharge data, for which profile (s) is/ are required.
2) Rating curve: In case a control structure exists but not essential
3) Observed flood marks for simulation and verification
4) Cross section survey data at representative points along the reach.
5) Bridge dimensions and details.
6) Mannings's coefficients along the reach.

6.3 Existing data:

The data available includes daily discharge for nearly 15 years. Cross section and rating curves at the bridge site since 1988 with some gaps are also available.

6.4 Field investigations:

The field investigations for cross section surveys along the reach of 1.5 Kms were made near the bridge site. Eight cross sections at representative points were taken with 3 on the downstream and 5 on the upstream side of the bridge (fig 5). Water surface levels along the reach at these cross sections during high and low stage during Monsoon season were also recorded.

6.5 Data Assembly

The data was assembled to meet the format required by HEC-2 model. Considering the bed profile of the reach, subcritical option was selected. Initially Manning's N along the reach were adopted from Chow (1964) and later were modified by trial and error to match recorded water surface elevations. Loss coefficients for contraction and expansion were taken at 0.3 and 0.5 respectively (HEC-2 manual, 1990). The cross section data were fed starting from the most downstream point. Several intermediate cross sections were inserted by simply measuring the channel width, where the ground level did not vary significantly (see fig

5). The existing bridge (control structure) was modeled using the normal bridge method since all flows were to pass under the bridge (no pressure flow). The pier thickness were taken as 1 Metre with an average width (along flow) of 5 Mts. The special bridge option was not considered to represent the bridge structure for two reasons. Firstly several bridge openings had to be taken as one trapezoidal opening with one equivalent pier, which did not represent the actual conditions and is not recommended in the manual (HEC-2 manual, 1990b). Secondly no pressure flow was to be considered. Typical cross sections at downstream point, bridge point and upstream point are shown in fig 6. The input file for a multiple profile run is presented in appendix I.

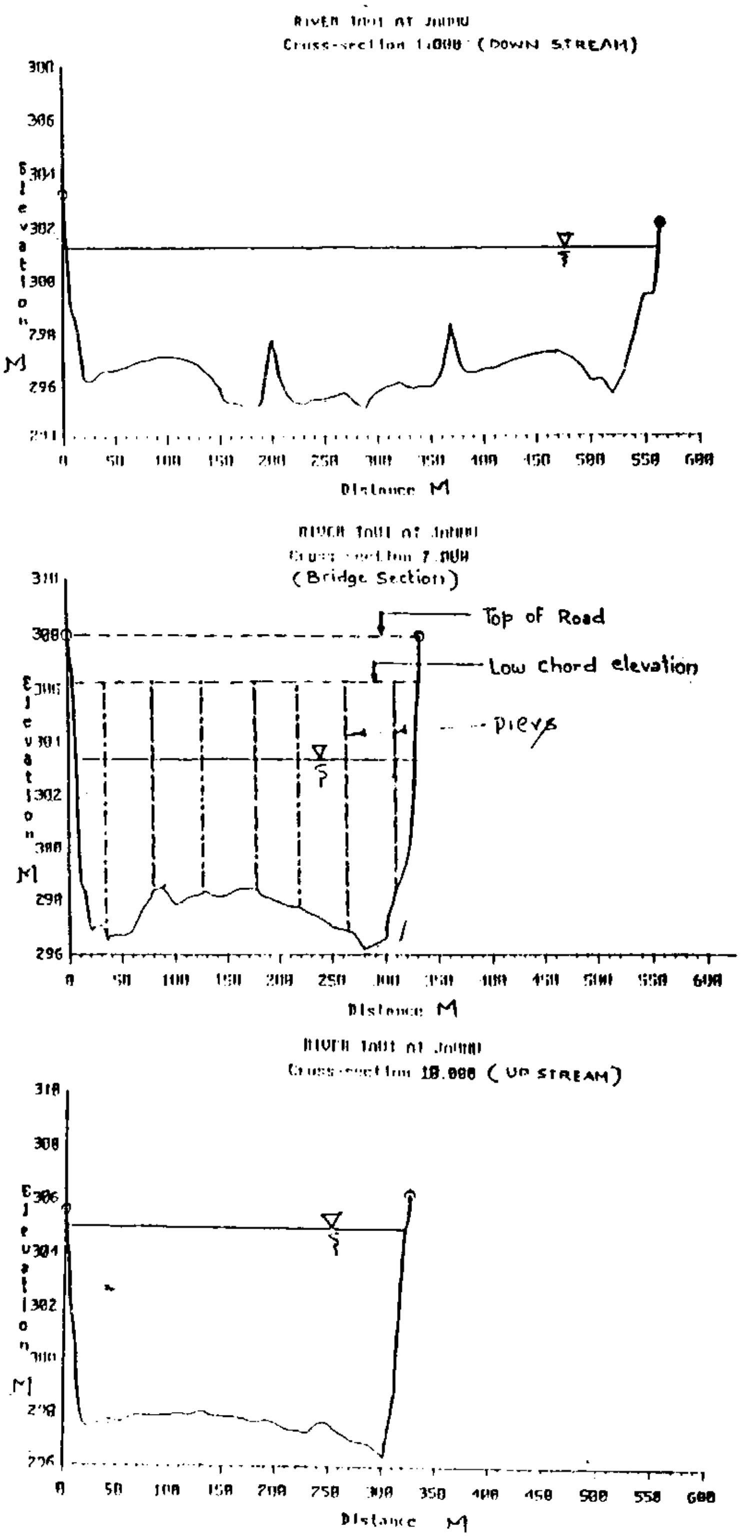

FIG : 6 TYPICAL CROSS SECTIONS

7.0 MODEL APPLICATION:

7.1 General

The model was applied using the data discussed in the previous section. Under low flow conditions (< 500 Cumecs) the river flows in two branches for nearly 400 Mts upstream of the existing bridge. The split flow option was however not considered since the reach was very small with apparently no difference in water elevation.

7.2 Mannings N

To obtain more realistic values of Mannings N along the reach, initial runs were made to match computed and observed water elevations by trial and error for medium flow conditions (1850 cumecs). The value of N ranged from 0.03 to 0.05. Higher flows were therefore expected to have N values in the range of 0.02 to 0.03 as already discussed in section 2.0

The variation of Mannings N along stage at the control section (bridge site) using data from CWC is presented in table

1. The values were computed using the slope area method and by Jarrets equation (1987). The two values compare reasonably.

Table 1. Computation of Manning's N

S.No	Gauge Mts	Slope	Velocity* Mts/sec	Discharge Cumecs	N^1	N^2
1.	297.53	0.00086	0.78	23.7	0.0265	0.0246
2.	297.65	0.00086	0.82	28.4	0.0294	0.0244
3.	297.68	0.00086	0.85	35.3	0.0276	0.0255
4.	298.02	0.00029	0.95	147.4	0.0291	0.0158
5.	298.25	0.00099	1.17	229.8	0.0267	0.0268
6.	298.65	0.00131	2.04	688.5	0.0192	0.0278
7.	298.93	0.00329	2.26	960.0	0.0316	0.0380
8.	294.35	0.00329	2.80	1541.0	0.0301	0.0363
9.	294.45	0.00395	2.90	1684.0	0.0330	0.0387
10.	294.65	0.00493	3.16	2167.0	0.0369	0.0406

Note: 1. N^1 Using Manning's equation

$$V = 1/N * R^{0.67} S^{0.5}$$

V = Velocity* by float method and reduced by a factor 0.8
R = Hydraulic radius
N = Mannings N
S = Slope of water surface

2. N^2 Using Jarret's equation (1987) for Manning's N
Jarrets equation is given by

$$N = 0.32 R^{0.38} S^{-0.16}$$

7.3 Development of Rating Curve:

The use of multiple profile option was made to cover a range of discharges including the estimated largest flood. While low flows were slightly affected by change in Mannings N and contraction/ expansion coefficients, in terms of computed water surface elevation and hence the rating curve, the higher flows indicated almost identical depths. The rating curve at the CWC gauge site (bridge site) is presented in Fig 7. A comparison of existing (CWC) and model developed rating curves is presented in fig 8.

RIVER TAWI AT JAMMU

CROSS-SECTION 7.0(BRIDGE SECTION)

RATING CURVE

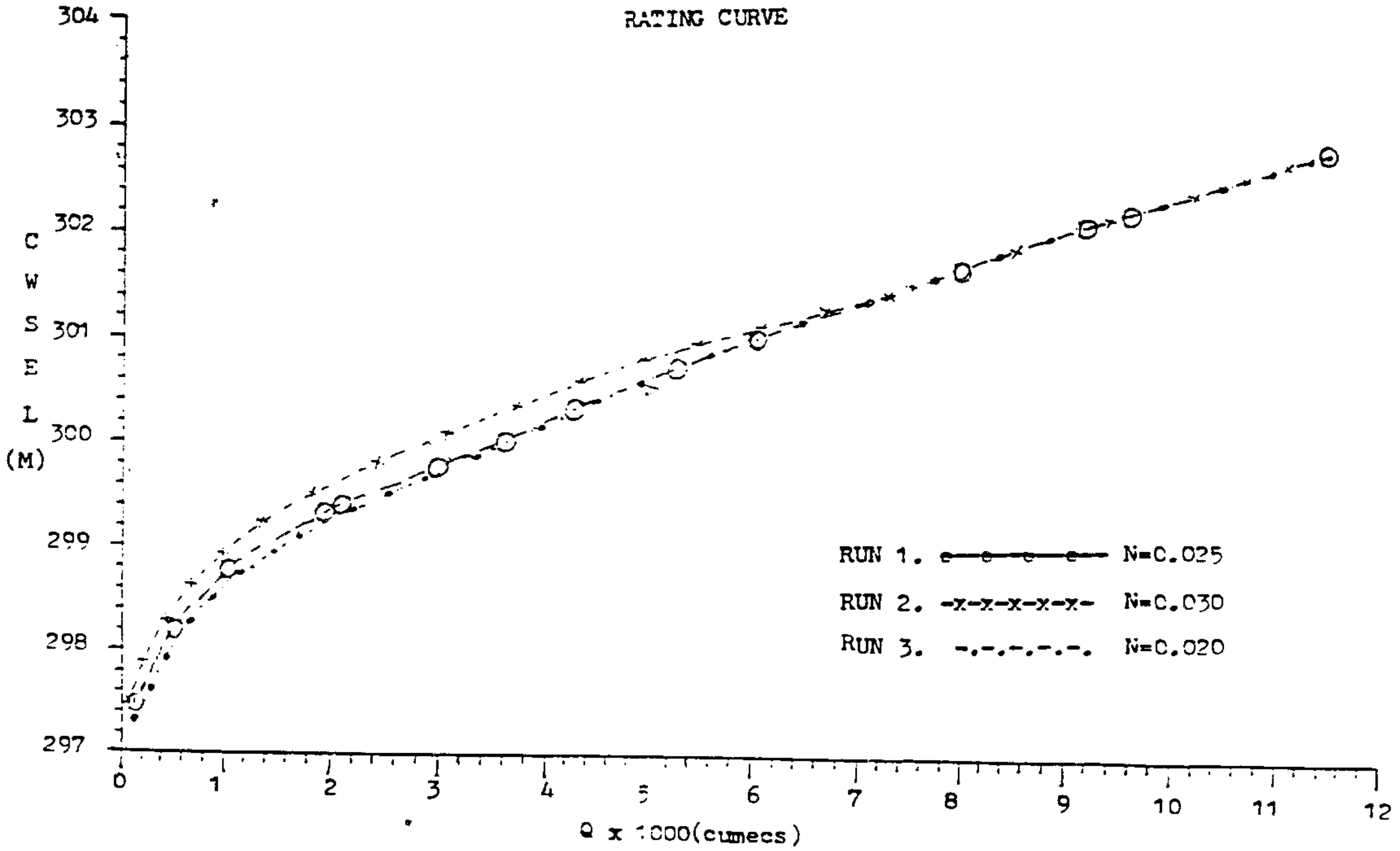

FIG. 7

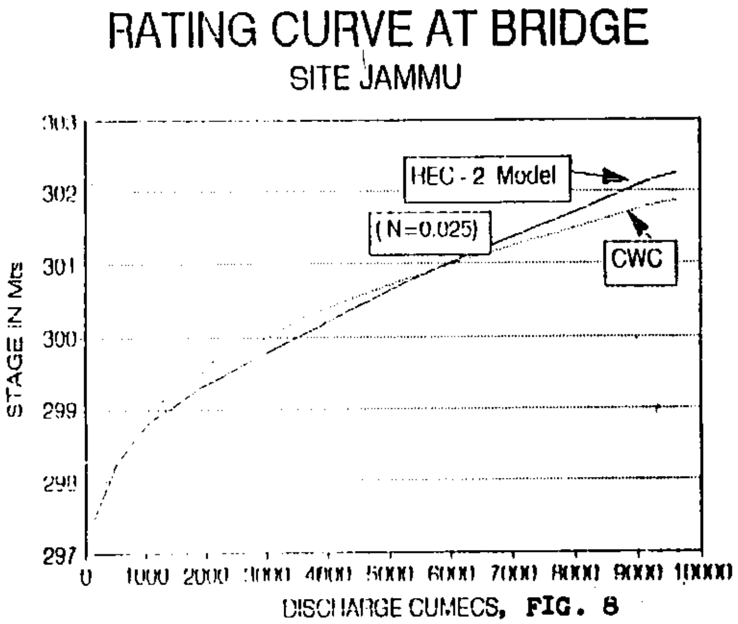

FIG. 8

7.4 Analysis of Sept. 1988 flood

The details of the unprecedented flood were discussed in section 5.3. To estimate this peak flood two approaches were adopted. These are as under:

1. Since the model computed water surface elevations which were relatively insensitive to any reasonable variations in loss coefficients, for high flow conditions the only variable left for manipulation was discharge Q. The peak flood of Sep 88 estimated at 4.3 lac cusecs (by state Flood control deptt) was varied in a range the computed profiles were compared with the observed flood marks (HFL) reported by the local residents at a few points along the reach. The computed profile at nearly 12000 cumecs (4.2 lac Cusecs) agreed closely. It was however assumed that the current cross sections were more or less the same as they were during

Sep 1988.

2. Alternatively the peak discharge may be computed from the recorded HFL at the bridge site (304.3 M) assuming critical depth at the section (Dobbie & Wolf (1953), Jarret (1984)). The model also indicates critical depth for most high flow conditions at the bridge site. This leads to an estimated discharge of about 5.6 lac cusecs.

Since critical depth is an unstable depth and that recorded flood level may also be prone to errors the most probable discharge considering the two approaches could be or the close to 12000 cumecs (4.2 lac cusecs).

To estimate the flood plain corresponding to this flood a dumpy level survey was carried out for 500 Mts on either side of the banks of the river along the reach. Contours were plotted and the flood plain corresponding to a profile of estimated 12000 cumecs. The flood plain and the profile are shown in fig 9. These are in reasonable agreement with reported flood levels by local residents. The flood plain must have extended beyond the breach as indicated in fig 9 (marked X) on the right bank, 500 Mts (approxi) upstream of the bridge due to a breach (right bank) as reported by local residents and in the Flood control deptt report (1988b). The flood waters after the breach reportedly inundated low lying areas from Jewel Chowk (topographic details beyond the breach are not available to extend the flood plain) upto the old campus of Jammu University. The profile of 100 year flood esti-

FLOOD PROFILE & FLOOD PLAIN NEAR BRIDGE
SITE JAMMU

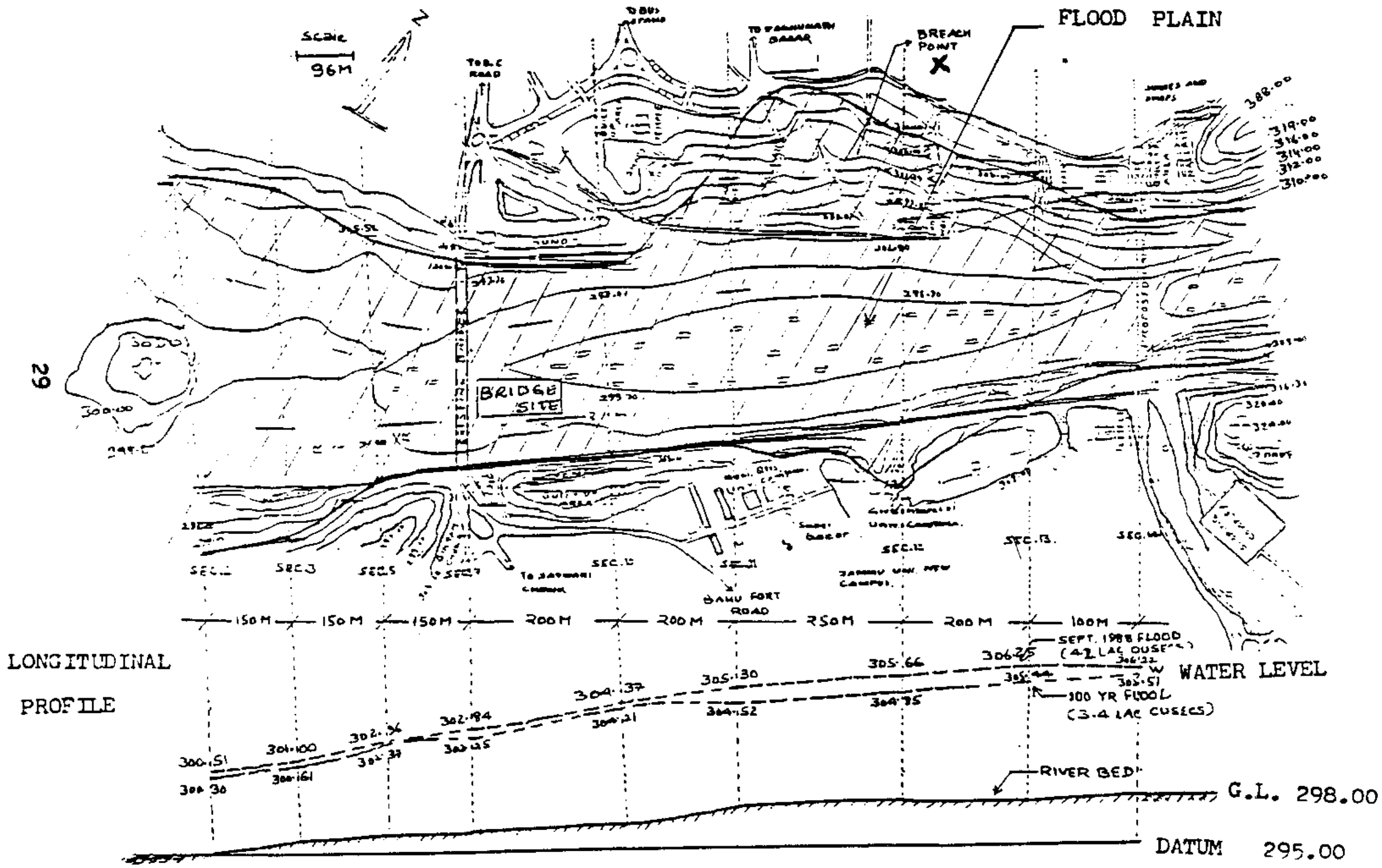

FIG 9.

mated at 9600 Cumecs (3.4 lac cusecs - design capacity of existing bridge) is also indicated in the same figure 9.

7.5 Flood Control and Channel Improvement:

The flood control techniques generally involve confining the estimated largest flows by means of embankments, levees, flood walls etc and lowering the peak stage through channel improvement methods (Linsley et al, 1979). Other methods involve construction of diversion structures or acquiring the flood plain areas at an early date under a master plan so as to avoid expensive construction of flood protection works.

The Irrigation and Flood control deptt, Jammu had reportedly taken several flood protection measures (I & FC report, 1988b) during Mar 1988 (before occurrence of flood) under the master plan for Jammu city. These included strengthening/ improving existing embankments, construction of spurs, studs and other river training works. After the Sep 1988 flood, most of these were either washed away or badly damaged.

At present the embankment on both the sides of river upstream of bridge upto 200 Mts is in a good condition with a uniform trapezoidal section. However, beyond this point and upto the proposed new bridge site and slightly beyond are low lying areas of Vinayak bazaar, Qasim nagar etc. on the right bank side and are not having adequate protection as evident from the profiles of 12000 cumecs (4.2 lac cusecs - flood of Sep 1988) or even the 3.4 lac cusecs (100 year flood) in fig 9.

The HEC-2 model has floodway analysis (encroachment option) and channel improvement (CHIMP option) to confine the flood to any desired bank station (or elevation) and keep the peak stage low by increasing the flow velocities. To illustrate this capability of the model a reach length of 400 Mts upstream of the bridge was assumed to be straightened with a uniform trapezoidal section by cutting (dredging) the bottom and sides of the channel with a more uniform slope. The existing embankments along the reach were assumed to be raised vertically to accommodate the flood of 4.2 lac cusecs (the model automatically does that when the banks cannot accommodate the computed water surface elevations for any given discharge). The resulting reduction in peak stage along the reach with and without channel improvement (Mannings N unaltered, but conveyance is increased) is indicated in table 2. The same is shown at a typical cross section in fig 10. Alternative methods of channel improvement that could be evaluated, include lining the channel, using rip rap etc, so as to reduce the value of Mannings N.

Several options or a combination must be examined before arriving at any particular flood control methodology. More so in terms of benefit cost analysis. This is however outside the scope of the present study.

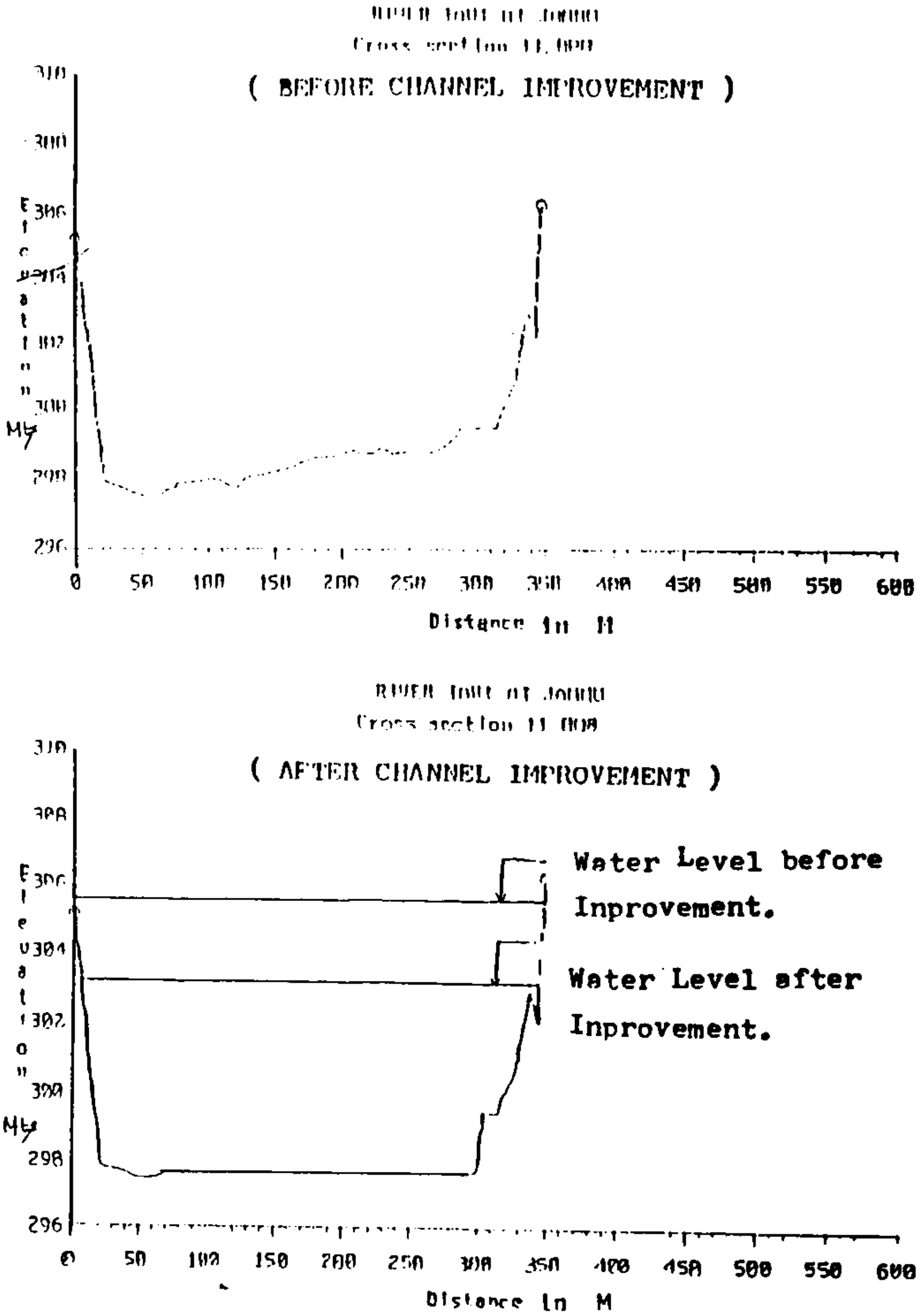

FIG : 10 EFFECT OF CHANNEL IMPROVEMENT

Table 2. Effect of Channel Improvement

S. No	Section No	Computed water surface elevation in Mts	
		Before Improvement	After Improvement
1.	2	300.73	300.61
2.	3	301.04	300.66
3.	5	303.03	301.44
4.	7	302.85	301.51
5.	9	304.49	302.69
6.	10	305.23	302.93
7.	11	305.54	303.13
8.	12	305.62	303.26
9.	13	306.59	304.45
10.	14	306.25	304.51

Note: See figs 9 and 10.

8.0 CONCLUSIONS:

Following conclusions may be inferred from the study:

1. The rating curve developed by CWC Jammu by actual measurement of discharge (float method) is in reasonable agreement with that developed through the model.The model developed curve may not be very accurate for low and medium (< 2000 Cumecs) conditions as a constant value of Manning's N has been used along the stage.
   The variation of N along the reach is (for high flows) not very significant as was evident from several simulation runs. Hence the model developed curve should be considered better (especially for high flows) compared with actual one owing to error in discharge computations using float method of velocity measurements and the extrapolation of rating curve on log-log graph. Also the model developed curve is based on actual cross sections and hence the conveyance.

2. The variation in Manning's N along stage lies between 0.02 and 0.03 (table 1) at the bridge section for low flow conditions (with exception for very low flows). Jarrets equation should be useful for estimating Manning's N.

3. The flood of Sep 1988 may be estimated close to 12000 cumecs (4.2 lac cusecs).

4. The flood profiles along with channel improvement suggested should be useful in planning flood control and flood management programmes. These measures should at at least be able to confine the 100 year flood of 3.4 lac cusecs. Following measures are recommended subject to structural and economic justifications.

   i) Acquiring the land in the flood plain (right bank) and /or

   ii) construction of flood wall, levees etc, along with channel improvement, straightening and bank stabilisation using rip rap. Each option or a combination of options which maximises the hydraulic capacity (conveyance) and reduces the peak stage.

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REFERENCES

Alridge B N and J M Garétt. 1973. Roughness coefficients for coefficients stream channels in Arizona. USGS open file report.

Barnes Harry H Jr. 1967. Roughness characteristics of natural channels. U S G S water supply paper 1849.

Bedient Philip B and Wayne C Huber, 1989. Rice University University of Florida. Hydrology and Flood plain analysis. Addison Wesley publ. pp 421-484.

Chow Ven Tee, 1964. Handbook of Applied Hydrology. Mc Graw Hill publication.

Dalrymple, Tate and M A Benson. 1967. Measurement of peak discharge by the slope area method. Techniques of water resources investigations of the USGS Book 3. Chap A2

Dawdy, D.R. and A K Motayed. 1979. Uncertainties in determination of flood profiles In: Inputs for risk analysis in water resources . Water resources publications, Fort Collins, Colorado. pp 193-208.

Grant Gordon E et al, 1992. XSPRO: A channel cross section analyser (An interactive program) Technical note 387. Bureau of land management service center Bldg 41 Denver Federal Center, Denver, CO, USA.

HEC-2, 1990a. User manual, Water surface profiles, US Army Corps of Engineers. A genralised computer program.p 3-11

HEC-2 1990b. User manual, Water surface profiles, US Army Corps of Engineers, pages III-33.

I & FC deptt, 1988a. Irrigation and flood control deptt Jammu report after the Sep 1988 flood. Chap 1.

I & FC deptt, 1988b. Irrigation and flood control deptt Jammu report after the Sep 1988 flood. Chap 7.

Jarret, Robert D. 1984. Hydraulics of high gradient streams Jr of Hydraulic Engg Am Soc Civil Engr, Vol 110, No 11 pp 1519 - 1539.

Jarret, Robert D. 1985. Determination of roughness coefftients for streams of Colarado USGS investigations report 85-4004, 54pp.

Jarret, Robert D. 1987. Errors in slope area computation of peak discharges in mountain streams. Jl of Hyd, 96 pp 59.

Jarret, Robert D and Harold E Malde, 1987. Paleodischarge of the late Pleistocene Bonneville Flood, Snake river, Idaho, computed from new evidence. Geo soc of Ameri Bull v.99 pp 127-134.

Jarret, Robert D. 1990. Paleo hydrologic techniques used to define the spatial occurrence of floods. Geomorphology, 3 pp 181-195.

Jain S K, 1992. Landuse mapping of Tawi catchment using satellite data. NIH case study publication. Roorkee.

Linsley Ray K and Joseph B Franzini, 1979. Water resources Engineering. Chap 20, Mc Graw Hill Int edition.

Mishra S K and K S Ramasastri, 1993. Application of HEC-2 model for paleoflood studies on Narmada. Ptd in the Int sym on Hyd and Water resources, New Delhi.

Patwary B C et al, 1991. Technical report on Hydrological aspects of the river Tawi. NIH publ 131.

Patwary B C et al, 1993. Hydrogeomorphological study of Tawi catchment, J & K. A case study NIH Publ 114.

Ramasastri K S, 1993. Hydrological network for Tawi - J & K Technical report 161.

Ramasastri K S, 1992. Hydrometeorological aspects of Sept 1988 storm over western Himalayas. Proc of the Int Sym on Hydrology of Mountainous areas, Simla. pp 29-38.

Sherman J O., 1976. Computer applications for stepback-water and floodway analysis, USGS, Open file report 76-499.

Thorne Collin R and Lyle W Zevenbergen. 1985. Estimating mean velocity in mountain rivers. Jr of Hydraulics Engg Am Soc Civil Engr Vol 111 No 4 pp 612-624.

Dobbie C H and Wolf P O, 1953. The Lynmouth flood of Aug 1952. Proc Inst Civ Eng, London, 2: 522-588.

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APPENDIX 1
(Input file)

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T1   RIVER PROFILE ANALYSIS 25-7-94
T2   EXISTING RIVER REACHES
T3        RIVER TAWI AT JAMMU            (BRIDGE SITE)
J1   -10      2      0      0      0      1    .30           300.5
J2            0                                 -1
J6
NC                 0.025    .3     .5
QT     1  11500
X1     1     81      0  565.7
X2      297.305
GR 303.3      0  302.6      1  301.9      2  301.3      3  300.6      4
GR 300.0      5  299.0      5  299.5      7  299.0      9  295.8      9
GR 298.7     10  298.6     11  299.5     12  297.9     13  297.7     14
GR 297.4     15  297.2     16  297.2     17  296.6     18  296.6     19
GR 296.2     20  295.2     30  296.5     40  296.6     50  296.7     60
GR 296.9     70  297.0     80  297.1     90  297.1    100  297.1    110
GR 297.0    120  295.9    130  296.4    140  295.0    150  295.5    153
GR 295.3    160  295.3    170  295.0    180  295.3    190  297.8    200
GR 295.3    210  295.4    220  295.3    230  295.5    240  295.5    250
GR 295.5    260  295.9    270  295.2    280  295.1    290  295.7    300
GR 295.0    310  295.2    320  295.3    330  295.0    340  296.0    350
GR 296.3    360  299.4    370  295.7    380  295.5    390  296.7    400
GR 295.7    410  295.0    420  297.0    430  297.1    440  297.2    450
GR 297.3    460  297.4    470  297.2    480  295.9    490  296.2    500
GR 296.3    510  295.7    520  295.4    530  297.9    540  299.5    550
GR 299.5    560  300.0    551  300.5    552  300.9    553  301.4    564
GR 302.2  565.7
NC                 0.025
X1     2                                75     75     75    .95    .15
X2       299.00
NC                  .025
X1     3     78      0    439     75     75     75
X2       298.95
GR 303.7      0  303.3      1  303.0      2  302.1      3  302.0      4
GR 302.0      5  301.5      6  300.7      7  300.5      8  300.0      9
GR 299.7     10  299.3     11  299.9     12  299.6     13  298.5     14
GR 299.4     15  299.4     15  299.3     17  299.1     18  298.0     19
GR 299.0     20  297.3     30  297.3     40  297.4     50  297.6     60
GR 297.4     70  297.5     80  297.2     90  295.9    100  296.6    110
GR 296.7    120  296.5    130  295.6    140  296.3    150  296.0    160
GR 297.0    170  296.9    180  297.0  188.3  297.1    190  297.0    200
GR 295.7    210  296.5    220  296.3    230  295.2    240  296.0    250
GR 295.9    260  295.8    270  295.0    280  295.2    290  295.4    300
GR 296.7    310  296.9    320  295.9    330  295.1    340  297.2    350
GR 297.4    360  297.6    370  297.7    380  297.9    390  298.0    400
GR 298.0    410  298.1    420  298.0    430  298.1    440  299.0    450
GR 298.1    460  297.9    470  299.5    480  300.1    485  301.2    490
GR 302.0    491  302.0    492  302.2    493  302.8    494  303.3    495
GR 303.8    496  304.3    497  305.5    499
NC                 0.025
X1   3.5                                35     35     35    .95    .05
X1     4                                40     40     40    .95    .10
X2       299.00
NC                 0.025
X1     5     72      0  453.5     75     75     75
X2       299.15
GR 303.7      0  303.3      1  302.8      2  302.4      3  301.9      4
GR 301.4      5  301.0      6  300.6      7  300.1      8  299.7      9
GR 299.5     10  299.4     11  299.4     12  299.3     13  299.3     14
GR 299.1     15  298.9     16  298.8     17  299.0     18  297.9     19
GR 297.9     20  299.0     30  299.0     40  297.5     50  297.3     60
GR 297.1   61.7  297.1     55  295.3     75  297.0     95  296.6     95
GR 295.6    105  295.7    115  295.9    125  295.9    135  296.7    145
GR 297.2    170  297.2    190  297.2    190  297.1    200  297.0    210
GR 295.9    220  295.3    230  295.3    240  295.5    250  295.4    260
GR 295.3    270  296.1    280  295.3    290  295.2    300  296.3    310
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GR	235.3	320	235.4	330	235.4	340	235.4	350	236.6	360
GR	297.0	370	297.4	380	237.9	390	238.5	400	299.0	410
GR	299.4	420	299.8	430	299.7	440	299.7	450	301.0	452
GR	301.7	453	302.0	454	302.3	455	302.7	456	303.4	457
GR	303.9	458	305.4	458.5						
NC		0.021	0.025							
X1	6	53	0	335	150	150	150			
X2		299.55								
X3	10							306.3	306.3	
GR	309	0	305.0	5	298.8	10	298.3	15	296.9	20
GR	297.1	30	295.7	34.5	295.5	35.5	296.7	40	296.7	50
GR	296.8	60	297.8	70	299.2	80	295.4	80.5	298.4	81.5
GR	299.4	90	297.8	100	298.0	110	298.2	120	298.2	126.5
GR	298.2	127.5	298.3	130	299.1	140	299.2	150	298.4	160
GR	299.4	170	299.4	177.5	299.4	178.5	299.2	180	298.1	190
GR	298.0	200	297.9	210	297.8	218.5	297.9	219.5	297.7	220
GR	297.5	230	297.3	240	297.1	250	235.9	250	298.8	264.5
GR	235.9	255.5	235.8	270	298.2	280	235.4	290	295.6	300
GR	237.5	310	239.5	310.5	239.5	311.5	299.9	315	239.2	320
GR	300.2	325	304.6	330	309	335				
NC		0.021	0.025							
X1	7				1	1	1			
X3	10							306.3	306.3	
BT	-67	0	309	309	5	309	305.3	10	308	306.3
BT		15	309	305.3	20	309	305.3	30	308	306.3
BT		34.5	309	305.3	34.5	309	296.7	35.5	308	236.7
BT		35.5	309	305.3	40	309	305.3	50	308	305.3
BT		60	309	305.3	70	309	305.3	90	309	308.3
BT		80.5	309	305.3	80.5	309	299.4	81.5	309	298.4
BT		81.5	309	305.3	90	309	305.3	100	308	306.3
BT		110	309	305.3	120	309	305.3	126.5	309	305.3
BT		126.5	309	299.2	127.5	309	299.2	127.5	308	306.3
BT		130	309	305.3	140	309	305.3	150	309	306.3
BT		160	309	305.3	170	308	305.3	177.5	308	306.3
BT		177.5	309	299.4	178.5	309	299.4	178.5	309	306.3
BT		190	309	305.3	190	309	305.3	200	309	306.3
BT		210	309	306.3	218.5	309	305.3	218.5	309	297.8
BT		219.5	309	297.9	219.5	308	305.3	220	308	305.3
BT		230	309	305.3	240	309	305.3	250	308	305.3
BT		250	309	305.3	254.5	309	305.3	254.5	308	295.8
BT		255.5	309	235.8	255.5	308	305.3	270	309	306.3
BT		290	309	305.3	290	309	305.3	300	309	306.3
BT		310	309	306.3	310.5	308	305.3	310.5	309	298.5
BT		311.5	309	293.5	311.5	309	305.3	315	308	306.3
BT		320	309	306.3	325	309	306.3	330	308	308.3
BT		335	309	309						
X1	8				5	5	5			
X2		299.69					1			
X3	10							305.3	305.3	
NC			0.025							
X1	2	33	0	335	1	1	1			
X2		299.72								
X3	10							306.3	305.3	
GR	305.8	0	305.3	5	299.8	10	299.3	15	236.9	20
GR	297.1	30	295.7	40	295.7	50	295.8	60	297.6	70
GR	299.2	80	299.4	90	297.8	100	298.0	110	299.2	120
GR	299.3	130	299.1	140	233.2	150	295.4	160	239.4	170
GR	299.2	180	298.1	190	295.0	200	297.9	210	297.7	220
GR	297.5	230	297.3	240	297.1	250	235.9	260	295.8	270
GR	295.2	280	295.4	300	295.6	300	295.5	310	299.2	315
GR	299.2	320	300.2	325	304.6	330	305.6	335		
NC			0.025							
X1	2.5				35	35	35	35	[illegible]	
NC			0.025							
X1	10	[illegible]	0	325	150	150	150			
X2		299.23								

GR 305.6	0	304.9	1	304.3	2	304.1	3	303.7	4
GR 302.9	5	302.4	6	301.9	7	301.5	8	301.0	9
GR 300.6	10	300.2	11	299.2	12	298.9	13	298.3	14
GR 298.0	14.9	297.5	20	297.5	30	297.8	40	297.7	50
GR 297.8	60	297.3	70	297.9	80	297.9	90	297.9	100
GR 298.0	110	297.9	120	298.1	130	297.9	140	297.9	150
GR 297.9	160	297.9	170	297.7	180	297.9	190	297.7	200
GR 297.4	210	297.4	220	297.3	230	297.7	240	297.6	250
GR 297.3	260	297.0	270	295.9	280	295.9	290	295.4	300
GR 296.4	301	296.4	302	296.5	303	297.2	304	297.2	305
GR 297.4	306	297.9	307	297.9	308	298.1	309	298.5	310
GR 298.8	311	298.9	312	299.6	313	300.2	314	300.7	315
GR 301.2	316	301.8	317	302.4	318	302.9	319	303.6	320
GR 304.8	321	304.9	322	305.2	323	305.6	324	306.1	325
GR 306.3	326								
NC		0.025							
X1 10.5				100	100	100	1.0	.15	
NC		0.025							
X1 11	52	0	351	100	100	100			
X2	301.010								
GR 305.1	0	304.2	1	304.0	2	303.9	3	303.5	4
GR 303.3	5	302.9	6	302.5	7	302.3	8	302.2	9
GR 301.5	10	301.3	11	300.7	12	300.3	13	299.8	14
GR 299.5	15	299.5	16	299.4	17	299.1	18	298.8	19
GR 298.0	20	297.8	30	297.7	40	297.5	50	297.5	60
GR 297.7	70	297.3	80	297.9	90	298.0	100	298.0	101.5
GR 298.0	110	297.7	120	298.1	130	298.1	140	298.2	150
GR 298.3	160	298.5	170	298.6	180	298.6	190	298.7	200
GR 298.8	210	298.7	220	298.9	230	298.7	240	298.8	250
GR 298.9	260	298.9	270	299.0	280	299.5	290	299.5	298
GR 299.5	315	300.0	320	300.9	330	302.3	335	303.0	340
GR 302.1	345	302.9	345	303.6	347	304.4	348	305.1	349
GR 305.9	350	306.3	351						
NC		0.025							
X1 12	72	0	350	200	200	200			
X2	301.58								
GR 302.6	0	302.5	1	302.2	2	302.0	3	301.9	4
GR 301.8	5	301.2	6	301.1	7	301.2	8	301.2	9
GR 300.8	10	300.2	11	299.3	12	299.3	13	299.1	14
GR 299.1	15	298.3	15	299.5	17	299.2	18	298.1	18.8
GR 297.3	20	297.7	30	297.5	40	297.5	50	297.8	60
GR 297.7	70	297.6	80	297.8	90	297.9	100	297.6	110
GR 297.9	120	299.1	129.5	298.5	130	298.4	140	298.7	150
GR 299.5	160	298.8	170	299.0	180	299.0	190	299.0	200
GR 299.1	210	299.0	220	299.0	230	299.9	240	298.9	250
GR 299.0	260	299.3	270	299.6	280	299.8	281	300.1	282
GR 300.3	283	300.4	284	300.4	285	300.5	286	301.1	287
GR 301.7	289	302.3	289	302.3	293	301.1	295	303	300
GR 303.3	305	303.8	310	304.3	315	304.5	320	304.5	325
GR 304.7	330	305.3	335	305.4	340	305.5	345	305.8	350
GR306.17	355	306.9	350						
NC		0.025							
X1 13				195	195	185		0.05	
X1 14	23	0	319	195	185	195			
GR 315.2	0	314.6	3.9	314.5	7	312.9	8.8	310.4	11.9
GR 303.5	14.55	307.5	21.45	302.8	24.5	300.8	30.8	299.8	33.2
GR 299.3	42.4	298	75	299.4	104.4	299.8	173.9	300.4	197.2
GR 301.2	227.2	302.4	257	305.0	261	309	271	310	294
GR 312.8	307	315	315	317	319				
EJ									

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