Problem of Navigation and Probable Remedies at Selected Locations In the Ganga River

A DISSERTATION

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

MASTER OF TECHNOLOGY

in

WATER RESOURCES DEVELOPMENT

(CIVIL)

By

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JUNE, 2008

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CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in the dissertation entitled **"Problem of Navigation and Probable Remedies at Selected Locations In the Ganga river"** in partial fulfilment of the requirement for award of the degree of **Master of Technology** and submitted in the **Department of Water Resources Development and Management (WRD&M)**, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out during the period from July 2007 to June 2008 under the supervision of **Prof. Nayan Sharma,** IIT Roorkee, Roorkee (India).

The matter presented in this dissertaion has not been submitted by me for the award of any other degree of this or any other institute.

Date 23 May 2008

Place : Roorkee

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

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ACKNOWLEDGEMENT

I wish to record my sincere thanks with profound sense of respect and gratitude for my supervisors **Prof. Nayan Sharma.** The words prove to be insufficient to express my deep feelings for his benevolence and elaborative guidance throughout the study period.

I am grateful to Prof. R.P. Singh, Professor and Head, WRD&M. I express my heartfelt thanks and gratitude to all the Professors of this department, WRD&M: . Prof.Devadutta Das, Prof. Gopal Chauhan, Prof. Ram Pal Singh, Prof. Raj Pal Singh Prof. G.C Mishra, Prof.B.N Asthana, Prof. M.L. Kansal, Prof. D.Khare, Dr. S.K. Mishra, Dr. Asish Pandey for their salient contribution during the course to fill the capacity in me in meeting the inputs required by this dissertation work. I am also thankful to the staff members of WRD&M for their help and gentleness in other formalities.

I must be thankful to O/o The Chief Engineer, Designs & Research, Water Resources Government of India for giving me an opportunity to pursue the course in this prestigious Institute.

My sincere gratitude goes to the Commissioner cum Secretary, Department of Water Resources, Government of Orissa for offering me a chance to enhance my career. I also gratefully acknowledge the various authors and publications from where relevant references have been drawn in this dissertation.

I am also thankful to all my co-trainee officers and friends for their cooperation and friendship in the last two years here at IIT Roorkee

Finally, I offer my hearty respect to my family for their blessings on me throughout the study.

Anupama Nayak

June, 2008 **(Anupama Nayak)**

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NOTATIONS

Symbol	Description
δ_{np}	= error signal of neuron p in the hidden layer;
$\bar{d}$	= mean of the desired output;
σ	= statistical standard deviation;
$\frac{VS}{w}$	= non-dimensional unit stream power;
γ_w	= specific weight of water;
σy_j	= standard deviation of the model output y_j;
$\bar{y}$	=mean of the model output;
σ_{dj}	=standard deviation of the model output d_j;,
$\propto$	= momentum factor;
ϕ	= sigmoidal activation function;
α	= slope parameter;
γ	= unit weight of fluid ;
η	= learning rate
ΔE	=change in error function;
C_{yjdj}	= covariance between the model output (y_j) and desired output (d_j);
f	=activation function;
g	= acceleration due to gravity;
k	= number of neurons in the single hidden layer;
k_i	= of neurons that feed forward to neuron I;
L_{cmax}	= mid channel length of the widest channel through the reach;
L_d	= length of the down stream reach used in control volume computation;
L_m	= thalweg length of channel;
L_r	= length of the reach measured midway between the banks of the channel belt

L_R = overall length of the meander belt reach measured along a straight line;

P = modified sinuosity parameter, wetted perimeter;

q = water discharge per unit width;

Q = water discharge;

Q_b = bank full discharge;

R =correlation co-efficient;

S = longitudinal bed slope;

Ω = stream power;

W = water surface width of stream / channel;

W_h = weights of the h^{th} input node and p^{th} hidden layer neuron;

w_{ij} =weight of the network;

W_o = weights of the connection between p^{th} hidden layer neuron and q^{th} output neuron;

Y_o = observed value;

Y_p = predicted value;

y_q = output;

SYMBOLS FOR UNITS

Cumec	Cubic metre per second
ha. m.	Hectare-meter
Kg	Kilogram
Km	Kilometre
km^2	Square kilometre
m	Meter
MCM	Million cubic meter

ABBREVIATIONS

HEC	:	Hydrologic Engineering Center
IRS	:	Indian Remote Sensing
LISS	:	Linear Imaging Self Scanner
NDVI	:	Normalised Difference Vegetation Indices
NRSA	:	National Remote Sensing Agency
WRD&M	:	Water Resources Development and Management

ABSTRACT

In this preliminary phase of the study, a prima-facie analysis was conducted to get an insight into the basic causes of insufficient navigation draught in the six locations on the Ganga river falling between Varanasi & Farakka.

This initial preliminary study phase has brought out the occurrence of channel flow bifurcation and shoal formation behaviour due to heavy sedimentation at these sites as the root cause of navigation bottlenecks. Stream bank erosion and heavy sediment deposition due to inadequate transport capacity could be identified as major underlying causes leading to channel instability and resultant reduction in draught.

Basic prerequisite to development of reasonably durable navigation channel in the Ganga river can be cited as effective sediment management both in the stream as well as in the watershed areas and effectively control the predominant channel instability processes to achieve concentration of stream flow to generate desired draught. The problem of reduced draught in several locations of the Ganga river can be primarily attributed to channel instability phenomenon triggered by heavy sediment inflows and channel widening process due to erosion which invariably cause erratic drifting of the thalweg channel.

From the preliminary analysis of the satellite data of the Ganga river along with available information on LAD, discharge etc., it could be surmised that in the identified six locations the draught reduction problem is primarily an off-shoot of the stream flow division into multiple channels with resulting braided configuration. This river behaviour of stream channel bifurcation in the Ganga is actuated by bank erosion accompanied with random deposition of sediments in heaps during the receding stages of flood flows when sediment transport capacity in the stream sharply declines.

In the above backdrop, the strategy to develop cost-effective durable fairway in the Ganga river warrants adoption of suitable flow channelisation measures to be supported by commensurate river sediment management along with compatible stream bank protection system in place to provide the required stream bank stability.

Chapter1
Introduction

1.1 General

Water based transport is effective as generally ,operating cost of fuel are low and environmental pollution is lower than for corresponding volumes of movement by road ,rail or air. A major advantage is that main infrastructure; the waterway is often naturally available which is then has to be trained, maintained and upgraded. Moreover the order of ratio's between water, railway and road transportation is within the ranges of 1 :2:5 in cost and 1:1.5:4 in energy consumption. So, this mode of transport continues to be competitive for transportation of bulk products in the present age of fast communication even in fully developed countries such as European Countries and United States. In some developing countries presently land routes are non existent in some areas and the simple roads or trails that do exist are inadequate for commercial transportation, especially in rainy season.. In such areas inland waterways are extremely important as transport routes of people and supplies. In short improved commercial IWT can support the economic development of a region and therefore, of the nation.

1.2 Findings of Study

In this preliminary phase of the study, a prima-facie analysis was conducted to get an insight into the basic causes of insufficient navigation draught in the six locations on the Ganga river falling between Varanasi & Farakka.

This initial preliminary study phase has brought out the occurrence of channel flow bifurcation and shoal formation behaviour due to heavy sedimentation at these sites as the root cause of navigation bottlenecks. Stream bank erosion and heavy sediment deposition due to inadequate transport capacity could be identified as major underlying causes leading to channel instability and resultant reduction in draught.

A preliminary package of cost effective measures have been tentatively identified for tackling the problem of navigation bottleneck caused due to flow division, shoal formation and channel instability. This tentative package includes submerged vane system, board fencing & permeable spur. A pilot channel to be supported by appropriate layout of submerged vanes and board fencing is proposed for Digha site.

In the next phase detailed analysis will be conducted for introducing refinements and fine tuning on these preliminary measures, which will be taken up by IWAI for field trials on pilot basis in the coming low flow season.

1.3 Organisation of the thesis

The main objective of this study is centered on evolving in-depth analysis of the river behaviour and the reasons causing the aforesaid cause with the help of the recent-most multi-band satellite data of latest sensor LISS-IV, in conjunction with hydrological & hydrographic data of the six identified locations on the Ganga. It has been tried in chapterII. In chapterIII with the help of these multi-band satellite data of latest sensor LISS-IV it is tried to measure the channel changes analytically. In chapterIV, it is envisaged to use wherever feasible (by available data) the mathematical modeling technique to be supplemented by studies based on analytical approaches on channel equilibrium conditions. In the light of the above approach, preliminary designs along with tentative layout of the river training structures have been evolved which have been presented in the chapterV in this report. The next phase of study is centered on evolving further refined and more objectively based channel development measures which will be taken up for implementation as field trials on pilot study basis in the coming low flow season. In this preliminary phase of evolving design for suitable river training works for the Ganga, it has been envisaged to undertake a prima facie laboratory experiment at the first instance.

Then the conclusions and suggestions for further study, references and appendices are enumerated in the following pages.

Chapter 2

Literature Review

2.1 General

Basic prerequisite to development of reasonably durable navigation channel in the Ganga river can be cited as effective sediment management both in the stream as well as in the watershed areas and effectively control the predominant channel instability processes to achieve concentration of stream flow to generate desired draught. The problem of reduced draught in several locations of the Ganga river can be primarily attributed to channel instability phenomenon triggered by heavy sediment inflows and channel widening process due to erosion which invariably cause erratic drifting of the thalweg channel. From the preliminary analysis of the satellite data of the Ganga river along with available information on LAD, discharge etc., it could be surmised that in the identified six locations the draught reduction problem is primarily an off-shoot of the stream flow division into multiple channels with resulting braided configuration. This river behaviour of stream channel bifurcation in the Ganga is actuated by bank erosion accompanied with random deposition of sediments in heaps during the receding stages of flood flows when sediment transport capacity in the stream sharply declines.

In the above backdrop, the strategy to develop cost-effective durable fairway in the Ganga river warrants adoption of suitable flow channelisation measures to be supported by commensurate river sediment management along with compatible stream bank protection system in place to provide the required stream bank stability.

For achievement of above objectives, at the first instance the following relatively cost-effective river training measures have been identified for field trials on pilot study basis.

1. Submerged Vane system (for desired flow modification, sediment redistribution & channelisation)
2. Board Fencing (for direct stream bank protection)
3. Wooden permeable spur (to slow down sediment-laden flows for inducing siltation and for breaking-up approaching erosive flow vortices)

The submerged vanes will be installed by water-jetting technique using balli piles for the sites having low flow water depths less than 2 m, while Geo-tubes may be required to be used for the sites with higher water depths to facilitate ease of putting in place these Vanes.

In this preliminary phase of evolving design for suitable river training works for the Ganga, it has been envisaged to undertake a prima facie identification of the required river training structures at the six locations after duly taking into account the local channel configurations depicted in the satellite images at the first instance.

In the light of the above approach, preliminary designs along with tentative layout of the river training structures have been evolved which have been marked in the satellite images enclosed in this report.
In the next phase of the study, in-depth analysis of the river behaviour have been undertaken with the help of the recent-most multi-band satellite data of latest sensor LISS-IV, in conjunction with hydrological & hydrographic data of the six identified locations on the Ganga. Towards this end, it is envisaged to use wherever feasible (by available data) the mathematical modeling technique to be supplemented by studies based on analytical approaches on channel equilibrium conditions. The main objective behind the next phase of study is centered on evolving further refined and more objectively based channel development measures which will be taken up for implementation by IWAI as field trials on pilot study basis in the coming low flow season. The brief description on the location-wise local navigation problems along with the preliminary package of measures with tentative dimensions are enumerated in the following pages.

2.2 Purpose

In Navigable waterways, where vessel traffic is expected to make use of the full water depth and width, it is necessary to ensure that a careful balance is achieved between the need to accommodate the user and the paramount need to maintain adequate safety allowance. This involves analyses and full account of the inter-relation between the parameters of the vessels, the waterway and weather factor. In addition, engineering measures to regulate the channel to get the objective of acquiring adequate navigable channel cross-section are to be dealt elaborately as far as possible.

2.3 Channel Depth

Obviously the actual depth required by vessels must be larger than the draught of the vessels. A reason for extra depth is that most vessels have squat which is defined as the tendency of a vessel to sit lower in the water when in motion. The second reason is that keeping some under keel clearance can make for easy and avoid risk of grounding caused by extra sinking due to wind, wave etc. Minimum water depth for safe navigation is calculated from the sum of the draught of design vessel as well as a number of allowances and requirements as per following formula.

Actual Waterway Depth = Target Vessel Static Draught + Trim + Squat + Exposure allowance + Fresh Water Adjustment + Bottom Material

Allowance + Over depth Allowance

+ Depth Transition - Tidal Allowance,

In the determination of the design draught, it should be realized that the depth does not necessarily have to be available 100 percent of the time. This may require the deepest draught vessel to schedule passage during high water levels. Selection of the design depth should be based on an economic analysis of the cost of vessel delays, operation and light load, compared with construction and maintenance costs.

2.3.1 Target Vessel Static Draught

The draught of the target vessel that will be using the waterway is based on the anticipated ship traffic for the proposed waterway. These dimensions are selected by an economic evaluation of the ship traffic for the waterway.

2.3.2 Trim

Trim is generally defined as the longitudinal inclination of a ship, or the difference in draught from the bow to the stem. It is controlled by loading. In general, at low speed, a ship underway will squat by the bow. The practice is to counteract this squat by trimming the ship by the stem when loading. The rule of thumb is to provide an allowance of 0.31 m to account for trim in waterway design.

2.3.3 Tidal Allowance

It is not of much significant in case of inland navigation. But still allowance for tidal effect should be derived from examination of statically significant sample of tidal records during the navigation season to determine tidal records during the navigation season to determine tidal height above the chart datum.

2.3.4 Squat

Squat refers to the increase of a ship's draught as a result of its motion through water. It is a hydraulic phenomenon whereby the water displaced creates an increase in current velocity past the moving hull causing a reduction in pressure resulting in a localized reduction of the water level and, consequently, in a settling of the vessel deeper in the water.

2.3.5 Depth Allowance for Exposure

The selection of the exposure allowance should take into account the movements of Heaving, pitching and rolling caused by local conditions, and should be based on available information on the local wave climate and vessel traffic considerations. If a substantial allowance is required for a minimal reduction in delays or the delay problems are minimal with low traffic, the allowance can be omitted. However, for other cases, the supplementary depth can be based on the information given below.

Exposure Depth Allowance	
Unexposed	0M
Medium Exposure (Minor Vessel Heaving)	.15 m
Fully Exposed	.30m

2.3.6 Fresh water Adjustment

Salinity increases the density of water, in turn reducing the draught of the vessel in the waterway. Design of the waterway depth should account for fluctuations in the salinity that may occur in an estuary exposed to tidal influences and river discharges. An adjustment for fresh water should account for the decreased buoyancy of the vessel.

A rule of thumb to determine the additional loading allowance for vessels in fresh water is to set it at 2-3% of the salt water draught.

2.3.7 Bottom Material Allowance

This allowance, also known as the Net Under keel Clearance, is by definition the minimum safety margin between the keel of the vessel and the project waterway depth. The value is a function of the nature of the bottom, the handling characteristics of the vessel and the operational character of the waterway.

Bottom Material Depth Allowance

Soft	0.25m
Medium (Sand)	0.60m
Hard Bottom (Rock)	0.90m

2.3.8 Maneuverability Margin

The Maneuverability Margin is made up of the allowance for bottom material (or the Net Under keel Clearance) and the exposure allowance. This margin is a measure of the minimum required to allow the vessel to man oeuvre adequately in the waterway. A minimum margin of 1.0 m is generally used for the operation of large vessels. Therefore, the sum of the Bottom Material Allowance and Exposure Allowance should be at least 1.0 m to accommodate the Maneuverability Margin for vessels of 250,000 DWT and greater.

2.3.9 Over Depth Allowance

Over depth Allowance refers to an allowance to account for waterway siltation between dredging and tolerance of sounding and dredging. The dredging tolerance varies with the type of dredging plant employed and the bottom conditions. The average acceptable tolerance is 0.3 m. If the bottom material is soft and can be displaced by a ship, no tolerance allowance is necessary.

2.3.10 Depth Transition

If the transition between adjacent reaches is large, the sudden change in under keel Clearance will have an effect on the current velocities and hydrostatic pressure on the hull. The result will be a change in the ship's performance, maneuverability and draught. Vessel squat in a transition area is presently being evaluated by Waterways Development. The preliminary analysis shows that the squat would increase by 15% to 20% when the transition is from deep water to shallow water.

2.4 Channel Width

The total channel width refers to the horizontal distance measured from the toe-to-toe side slopes at the design depth. Total width is expressed as:

Total Width = Design Width + Allowances

Design Width refers to the summation of width requirements for:

1) Ship maneuvering;
2) Hydrodynamic interactions between meeting and passing vessels in two-way Traffic;
3) Counteracting crosswinds and cross current;
4) Counteracting bank suction; and
5) Navigational aids (including **pilots).**

Allowances refer to additional width increases to compensate for bank slumping and erosion, sediment transport and deposition, as well as the type of bank material.

2.4.1 Maneuvering Lane

The maneuvering lane is the width required to allow for the oscillating track produced by the combination of sway and yaw of the vessel. Maneuvering lane widths should be calculated for the largest of the most frequently expected vessel type, and the resulting largest lane should be adopted as the required maneuvering lane width. In some cases, depending on the traffic structure, the channel width may accommodate two-way traffic for a certain range of vessel sizes and one-way traffic for a larger range of traffic. Frequency of channel use by vessel classes can be used to determine the probability of the width that would be required.

Depending on the type of target vessel, a "maneuverability coefficient" is multiplied by the target vessel's beam (B) to determine the maneuvering lane width.

Vessel Type	Maneuverability	Maneuverability Coefficient	Lane Width
Naval fighting Vessels, Victory Class freighters	Excellent	1.3	1.3 B
Tankers, new ore Ships .liberty Class freighters	Good	1.5	1.5 B
Old ore ships, Damaged vessels	Poor	1.8	1.8B

Where B = target vessel beam

2.4.2 Hydrodynamic Interaction Lane (Ship Clearance)

As two vessels pass, there are strong interaction forces between them, giving rise to path deviations and heading changes. Even though the interaction forces are quite large, the magnitudes of the path deviations and heading changes during the actual passing of the vessels are small. The real danger lies after the vessels have passed when the dynamic disturbances imparted to the vessels during passing can combine with bank effects and lead to oscillating diverging motions if not properly controlled.

The minimum hydrodynamic interaction width desired is 30 meters (100 feet). The recommended approach is:

Vessel Clearance = 1 **B,** if B > 30 m

Vessel Clearance = 30 m, if B < 30 m

Encounter traffic density should also be considered in two-way traffic channels. Additional width is required for channels with heavy traffic density. The requirements for traffic density are shown below.

Table: 2.1- Requirement for Traffic Density

Traffic Density*	Width Requirement
Light (0 - 1.0 vessel/hour)	0.0 B
Moderate (1.0 - 3..0 vessel/hour)	0.2 B
Heavy (> 3.0 vessel/hour)	004 B

2.4.3 Wind and Current Affects

Wind forces on a vessel produce two effects: a sideways drift and a turning moment. The former is overcome by steering a course to counteract it, and the latter is overcome by applying a certain amount of helm. Counteracting the drift will induce vessel yaw; this requires a widening of the channel.

The degree to which wind affects a vessel depends on the relative direction of the wind, the ratio of wind speed to vessel speed, the depth to draught ratio and whether the vessel is loaded or in ballast.

Winds from the bow are generally not a concern for wind speeds less than 10 times the vessel speed. However, winds become a greater concern as the wind shifts abeam. The maximum effect occurs perpendicular to the ship's beam.

The yaw angle caused by wind is most severe for a vessel in ballast. Therefore, it is the ballast condition that is used to determine the additional channel width required for wind effects. The width requirement for wind effects is shown in table below.

Table: 2.2- Width Requirement for vessel Manoeuvrability

Wind Severity	Width Requirement for vessel Manoeuvrability		
	Excellent	Good	Poor
Low « 15 knots)	0.0 B	0.0 B	0.0 B
Moderate (1.5-.33 knots)	0.3 B	0.4 B	0.5 B
Severe (> 33 knots)	0.6 B	0.8 B	1.0 B

Where B = target vessel beam

2.5 Case studies

2.5.1 Inland water way classification in India

Historically, atleast one some geographical section, it has been a viable mode of transport, but it has become a marginal part (.15%) of overall transport movement. The main passenger movement by inland water way that are viable are ferry operation across river (at numerous location on all waterway), on short stretch along river and tourism based passenger traffic (in Goa, Kerala, Sunder bans, Northen regions). So, inland waterway development is in nascent stage. A number of central and state agencies plau a role in the regulation, operation and substance of inland waterway transport. Some actors in these sectors are IWAI, CIWTC, state government, port authorities, transport development authorities.

Currently, three major waterway in the country have been designed as **National waterway: NW-1, the Ganga-Bhagarathi-Hooghly stsyem,** from Allahabad to Haldia, **NW-2, the Brahmaputra system in Assam and NW-3, the west coast canal system** in Kerela. Commercially the most important sector in the small tidal riveine system in Goa, comprising the Zuari and Mandovi rivers and the Cumberjua canal. A number of possibilities do exists, in terms of in-principle navigable waterway , but the once that offer some potential (a mix of feasibility and some traffic possibilities) are the riverne inlet along the coast, especially the ones near ports and some of the canal system as part of large water resource development project.

Measures taken so far to develop IWT

- In National Waterway I, navigable depth of 2 m is maintained on the stretch from Haldia -Farakka -Patna (1020 km) for about 330 days in a year.
- In Natioal Waterway 2, depth of 2 meters is maintained between Dhubri and Pandu (260 km) for about 330 days a year.
- In National Waterway 3, capital dredging is in progress after completionj of which depth can be maintained
- If we compare navigable depth of 2.0 meter with corresponding carrying capacity, (refer table at previous page) then it comes out to be 500-750 ton.

Nevertheless, the major problem the barge operators report is the non availability of sufficient draught to operate vessels of sufficient size in an economical manner. Traffic appears to be a secondary concern although because of years of decline in this sector [Raghuram, 2006]. So emphasis needs to be given to engineering measures to maintain navigable depth of at least 2.0 m – 2.5m or more at all conditions thought out the year.

Table 2.3 The national water way system

	Details of national waterway	**Distance (Kms)**	**Cargo moved**		
			1998-99 Lakhs Tons	**1999-00 Lakhs Tons**	**2000-01(P) Lakhs Tons**
1	National waterway 1 (Allahabad-Haldia stretch of Ganga-Bhagarathi-Hooghly river system)	1620	8.52	7.31	3.52
2	National waterway 2(Sadiya – Dhubri stretch of Brahmaputra river system)	891	0.09	0.06	0.04
3	National waterway 3 (Kollam-Kottapuram stretch of west coast channel along with Champakara Canal and Udyog-Mandal Canal)	205	10.27	11.12	10.85
	Total	**2716**	**18.88**	**18.49**	**14.41**

Source: Inland waterways Authority of India.

In India, IWAI, in principle, commits to maintaining a year-round draught of 2 m along the National Waterway [Planning Commission. 2001]. This is not found to be the case, in practice. One portability is that it is strategically justified to provide this draught on appropriate channels, (through various measures as mentioned earlier) by an assessment of the commercial traffic potential on each waterway. The other option is for operation to plan fro a realistic draught of 1.5 m and see if that if operationally viable.

The ultimate requirement if that the river if trained and constantly provides sufficient depth vis-à-vis the draught of the vessels. Out are expected to ply on it. This is possible for some type of river beds and may require maintenance of banks and dredging of the river bed periodically, to maintain the required depth provided adequate flow of water is available. This will be discussed in the next section elaborately. The width requirement for safe use of waterways for commercial dipping is not of much concern as the traffic in inland Indian waterways is low as well as Indian risers , are wide with shallow depth.

2.5.2 Revival of inland navigation in Pakistan

In Pakistan, river Indus and its tributaries had been used as means of transport for a very long time. Navigation in rivers, canals and other water oourses (specially river Indus) had developed to a great extent by the end of fourteenth century primarily from defence point of view.

River indus is perennially navigable in its upper reach from Kalabagh to Sukkur but downstream of Sukkur in the Lower Indus region the river flows during the dry season are very low and the water has also to be diverted to canals in the region skking navigation in the river downstream of Sukkur almost impossible.

The problem, therefore, is to provide an inland water route which will connect Port Qasim to river Indus upstrem of Sukkur.

Channel dimensions

The relevant factors consider in deciding the channel width are transverse currents, ship maneuvering requirements and the resistance generated by channel section to the ship's movement. Thelevent factors considered in establishing channel depth are squat, underkeel clearance and unevenness of channel bed. Considering all these factors it is worked out that appropriate canal dimensions for navigation through the proposed water routes would be- bed width 128 ft, water depth 9ft and side slope 2:1.

Problems and Their Solutions

The development of inland water routes on the existing irrigation system as proposed above would involved some construction and maintenance problems. These inter-alia relate to (1) development and maintance of canal beds against propeller action and protection of side slopes against wave action (2) remodeling or reconstruction of existing structures like railway, road and canal crossings without interfering with the traffic and water supply (3) construction of navigation locks and remodelling of perennial canals without their closure (4) connection of navigation channel with river Indus upstream of Sukkur barrage (5) silting of pond area in upper reaches of Kotri barrage and maintenance of channel at navigable depth (6) crossing of a hill torrent named Baran Nai at mile 18.06 of Kalri Baghar Feed (7) frequent dredging of Gharo Creek to Maintain navigable depth in the upper reaches (8) discontinuation of navigation during annual closure of canals and (9) grounding of crafts in case of sudden unforeseen closures of canals due to breaches and other accidents.

2.5.3 Channel Development in the Lower Reach of the Red River.

Hydraulic Model Investigation.

The Red River Waterway provides for the development of a 9-ft-deep by 200-ft-wide navigation channel from Lake of Pines near Dangerfield, Texas, to Old River. The lower reach of the Red River with its flat slopes and narrow bends of short radii transverses the flood plain of the Mississippi River. A movable-bed model, reproducing about 4.7 miles of the Red River and a short section of the Black River to a horizontal scale of 1:120 and a vertical scale of 1:80, was used to provide an indication of the amount of maintenance dredging that would be required to: (a) maintain a navigation channel from Lock and Dam 1 to Old River; (b) develop a system of channel training structures to minimize the required dredging; and 9c) determine the ultimate channel section to be expected in Lorran Lake cutoff, a cutoff proposed just upstream of the mouth of the Black River. Results included: 1) The proposed overall channel was too wide to maintain a navigation channel of adequate width and depth above the Black River. 2) Development of an adequate navigation channel in the test reach would require a reduction in the width of the overall channel. This can be obtained with dikes or by moving the banks closer together and revetting them. 3) Structures would be require on the left downstream of the Black River to develop an adequate channel along an alignment suitable for navigation. Keywords: Hydraulic models.

2.5.4 Development and Maintenance of Navigation Channel, Arkansas River, Arkansas and Oklahoma. Hydraulic Model Investigation.

As part of the studies for development of the multiple-purpose plan to provide a navigable waterway from the Mississippi River to the general area of Tulsa, Oklahoma, a scale model of a typical 10-mile reach of the Arkansas River was used to test various plans for channel improvement and stabilization under conditions of normal sediment load. Principal conclusions were: A dredged cut without an appreciable reduction in the river's sediment load or the addition of regulatory works would only temporarily affect channel conditions; Regulating structures to increase the radius of curvature of bends would tend to provide a more uniform channel cross section and improve channel alignment over crossings; Controlling depths could be increased by contraction of the channel, particularly at crossings. Longitudinal groins would be effective in fixing the location and alignment of the channel over crossings, and would produce some deepening of the channel over the crossings. (Author)

2.5.5 Matagorda Ship Channel, Texas: Jetty Stability Study

The entrance of the Matagorda Ship Channel, connecting the Gulf of Mexico to Matagorda Bay, Texas, has experienced a strong currents since its construction in 1963-1964. Strong currents had been predicted in physical model experiments performed during design to determine the optimal location of the new inlet cut through Matagorda Peninsula and entrance configurations. The current has produced a large area of scour on the bay side of the inlet adjacent to the west jetty, and vessels encountering a strong along-channel and cross-channel current at the entrance experience difficulty in navigation. This study was performed to understand the hydrodynamics of the existing condition and evaluate alternatives for stabilizing the jetties to reduce the current velocity, thereby reducing the scour and improving navigation reliability. The interaction between the entrance and Pass Cavallo, the natural inlet to Matagorda Bay located southwest of the Matagorda Ship Channel entrance, was also examined in a regional approach. The study proceeded by review of the engineering and scientific literature, analysis of regional and local trends in the shoreline change at the entrance and at Pass Cavallo, field measurements of the water level and current, bathymetry surveys, and hydrodynamic numerical modeling of tidal circulation, including wind forcing and river discharges to the bay. Alternative configurations of the jetties were investigated with the hydrodynamic model. A frequency-of-occurrence methodology based on the current velocity magnitude was introduced to evaluate the alternatives. Possible changes in salinity were also investigated.

Classification of Channel

In order to organize direct IWT on the mainstream and its tributaries within a river basin and between one river system and other, the main dimensions of the vessels, waterway, locks and structures across- over the navigation channel and the interrelations among them should be well coordinated. Therefore a unified classification of waterway with standard dimensions throughout the country is certainly needed. The advantage of IWT will be fully taken, and the economic development will be surely promoted. In this regard classification of channels for navigation by waterways agencies of china can be referred for further guidance & analysis. Following table shows the Chinese classification.

2.5.6 Classification of Navigable Waterways in China (In Brief)

While going through the table it should be noticed that the minimum channel depth of the river is actually a range with its lower limit equal to the standard water draft of the barge, since the allowances and under keel clearance must be provided for safe navigation, which means that the vessels should navigate with reduced load during the low-water seasons. During the medium-flow and high-flow seasons, the barges can be loaded up to standard and fully loaded respectively, in this way the amount of work for maintaining the channel will be greatly reduced.

Table 2.4 National Waterway system in China

	Barge		Fleel of barge	Channel minimum dimensions during low flow session					
				Natural and canalized river			canal		Radius of curvature (m)
				Depth (m)	Bottom width(m)		Depth (m)	Bottom width (m)	
Class	Carrying capacity	Dimensions(m) (length*width* depth)	Dimensions(m) (length*width* depth)		One way	Two way			
1.	3000	75*16.2*3.5	192*32.1*3.3	3.5-4.0	70	130	5.5	130	580
2.	2000	75*1.1*2.6	180*1.1*2.6	2.6-3	35	70	4.0	61	340
3.	1000	67.5*10.82.0	167*21.6*2.0	2.2-4	45	90	3.2	85	300
4. 5.	300 300	45*10.8*6 35*9.2*1.3	112*21.6*6 80*18.4*1.3	1.6-1.9 1.3-1.6	40 35	80 70	2.5 2.0	80 35	340 270
6.	100	30*6.4*1.0	74*6.4*1.0	1.0-1.2	15	40	1.5	28	
7.	50	200*5.4*0.8	200*5.4*0.8	0.7-1.0	10	10	1.2	20	

Chapter 3
DESCRIPTION OF STUSY AREA AND RELATED PROBLEMS

3.1 General

In this phase of the study, in-depth analysis of the river behaviour have been undertaken with the help of the recent-most multi-band satellite data of latest sensor LISS-IV, in conjunction with hydrological & hydrographic data of the six identified locations on the Ganga.

The study on river morphology with the use of remote sensing data is a relatively new development, and has been in practice for not more than the last 20 to 25 years inIndia. The remote sensing technique is used to collect, analyze and convert remotely sensed data into useful information in order to assist in inventory, mapping and monitoring the earth resources. Useful information has been extracted from the satellite images and analyzed with the help of ERDAS IMAGINE 8.5

Remote Sensing (RS) and Geographical Information System (GIS) play a rapidly increasing role in the field of river engineering. The remote sensing data provides synoptic view of a fairly large area in the narrow and discrete bands of the electromagnetic spectrum. conventional measurements of plan form characteristics of alluvial rivers are a time consuming, laborious and expensive procedure. Their main disadvantage is that they provide information only at a particular point and instant of time. On the other hand, remote sensing techniques are capable of providing information through time and space, which can never be appreciated from the ground. This is one of the greatest advantages of using RS data for hydrological modeling and monitoring is its ability to generate information in spatial and temporal domain, which is very crucial for successful model analysis, prediction and validation. However, the use of RS technology involves huge amount of spatial and temporal database management, and requires an efficient system to handle the database. The GIS technology provides suitable alternatives for efficient management of large and complex databases.

Changes in the river channel alignment and size, development of bars, islands take place during floods, time dependant changes can be monitored in the specific river reach using remote sensing and GIS. Depending on the resolution of the imageries, accurate and dependable predictions can be made covering long reaches. for rivers with multiple channel system or braided rivers, these could be the important aspects resulting in bank erosion. CDM can also be important for planning and management for the development of inland navigation system, locating riverine ports and other structures, design of anti-erosion works, etc. Which are generally affected by the development of islands, bars in the channel, changes in the orientation of channel, avulsion etc. Thus, monitoring of temporal changes in the river channel with the help of satellite imageries would highlight on these aspects.

With the advancement in RS & processing technique, it is possible to deduce other information, about existence of paleo-channels, low lying areas, reaches under active erosion, damages due to over bank flows, breaches in the embankments and damages to the existing hydraulics structures. In spite of the fact that normally the changes in a braided channel are sudden and unpredictable, efforts have been made from time to time with the help of satellite Imageries for identification of any cyclic behavior observed in the past. Efforts are also made to identify physical limits of such changes, if any, observed earlier. About river studies co-relating these observations with other information can give fairly good idea of the river behavior in a specific reach and likely behavior in the immediate future.

In order to study the river plan-form and morphological changes of a river, information at a regular interval of time for whole length of the river may be required. Ground based observations and other methods may not be sufficient to model the plan-form and morphological changes. These may be uneconomical and time consuming if these methods have to be adopted on a repetitive basis. With advancement of remote sensing technique as an analysis tool, it is now possible to acquire information related to river morphology.

Changes in the river channel monitoring include the aspects, like changes in the location, size and alignment. In case of rivers with multiple channel system or braided rivers, these could be the important aspects resulting in bank erosion. This can also be important while considering the development of inland navigation system, locating riverine ports and other structures, design of anti erosion works, etc. These aspects are generally affected by the development of islands, bars in the channel, changes in the orientation of channel etc. Monitoring of temporal changes in the river channel with the help of satellite imageries would highlight on these aspects.

A preliminary proposal to evolve cost-effective river training works for navigation channel development at the following six sites on the Ganga River has been developed at the instance of IWAI. These six sites are namely –

(i) Bateshwarsthan at Ch. 683
(ii) Punarak at Ch. 879
(iii) Digha at Ch. 976
(iv) Arjunpur at Ch.1118
(v) Debchandpur at Ch. 1247
(vi) Nakhwa at Ch. 1299

The problem and the tentative solutions in the corresponding sites are briefly indicated below.

3.2 Bateswarsthan at Ch. 683

In BATESHWARSTHAN site located 139Km upstream of the Farakka Barrage, the required water depth of 2m is maintained with considerable difficulty by resorting to repetitive purely temporary measures of dredging and bandalling. These purely temporary measures are now hard-pressed to perform against the odds of sediment deposition in stream-bed and channel widening due to bank erosion. At this site, there is an imperative need to effect closure of subsidiary channel to bring about flow concentration along the navigation channel with required sand bank fixation. The ongoing severe river bank erosion especially along the upstream left bank of the Farakka Barrage has further added to the process of channel instability. Ideally, a well-concerted holistic approach is warranted through deployment of more durable river training works covering the entire stretch from the Farakka Barrage upwards to achieve all-round stream-bank stability which will benefit not only navigation, but will also restrict flood and erosion. Just upstream of Bateshwarsthan the channel is divided into two sub channels. One follows a gradual curve and the other follows a complex pattern. It is better to develop the gradual curve than the complex pattern. For that submerged vanes have to be implemented for about a distance of 2km and 500m width upstream of the point at which the braiding starts. For this site, the hydrological data of discharge, stage and sediment data as observed at Kahalgaon stream gauging station on the Ganga river have been used for the study.

Fig 3.1 ZOOMED Image of Tentative River Training Structures at BATESHWARSTHAN Ch. 683

Fig 3.2 MACRO Image of Tentative River Training Structures at BATESHWARSTHAN Ch. 683

Fig 3.3 Image (2007 Year) of Tentative River Training Structures at BATESHWARATHAN (Ch. 683)

3.3 Punarak at Ch. 879

In PUNARAK site situated 45Km downstream from Patna, the lean flow velocity of 0.8 to 0.9m/sec is prevalent with a much wider waterway of about 1500m and 2m depth. Such a wider channel configuration is not conducive for sustaining the desired navigation channel alignment of about 45m width. It is desirable to maintain a sinusoidal channelised waterway with a mild curvature for smooth plying of vessels along the navigation channel. Using remote sensing technique we can say that there is a band of length 4.7km upstream of the point from where the braiding starts where the depth is less than that required for navigation purpose. We will try to discourage the flow in the right channel and to develop the left channel by channelising it with the placing of submerged vanes at two locations. This objective can be achieved with channel developing river training works of semi-permanent nature. The observed discharge, stage and sediment data in the Hatidah stream gauging station on the Ganga have been used for design of this site

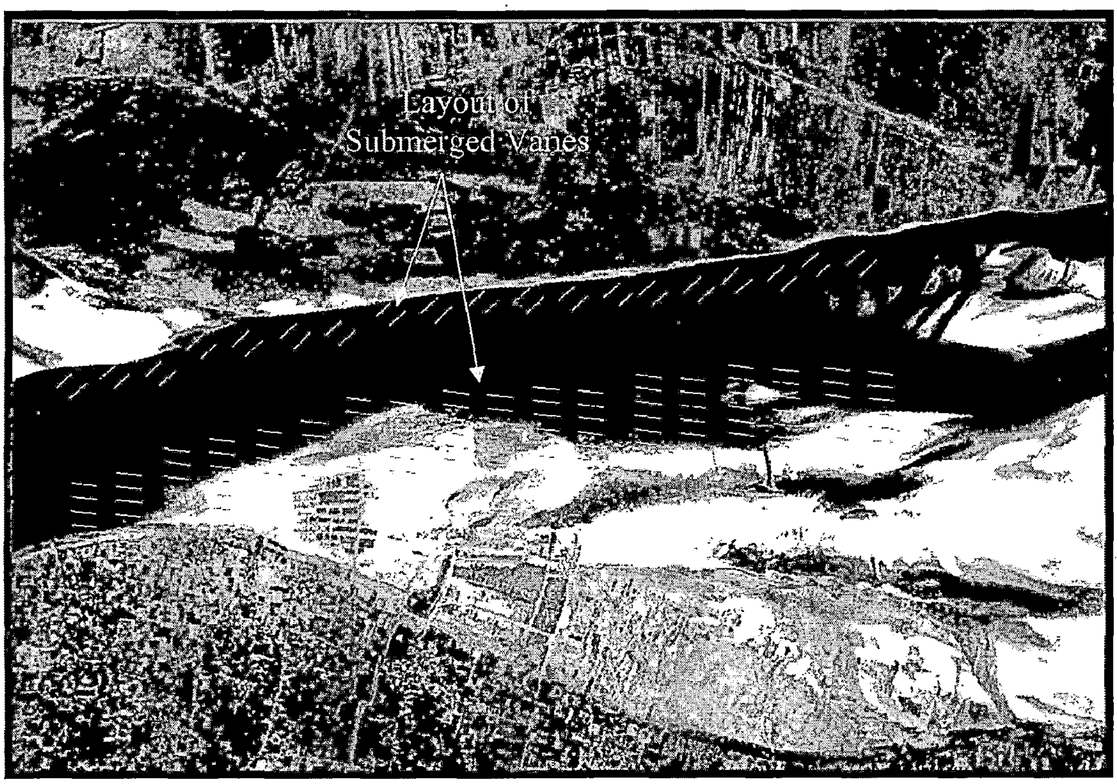


Fig 3.4 ZOOMED Image of Tentative River Training Structures at PUNARAK Ch. 879

Fig 3.5 MACRO Image of Tentative River Training Structures at PUNARAK Ch. 879

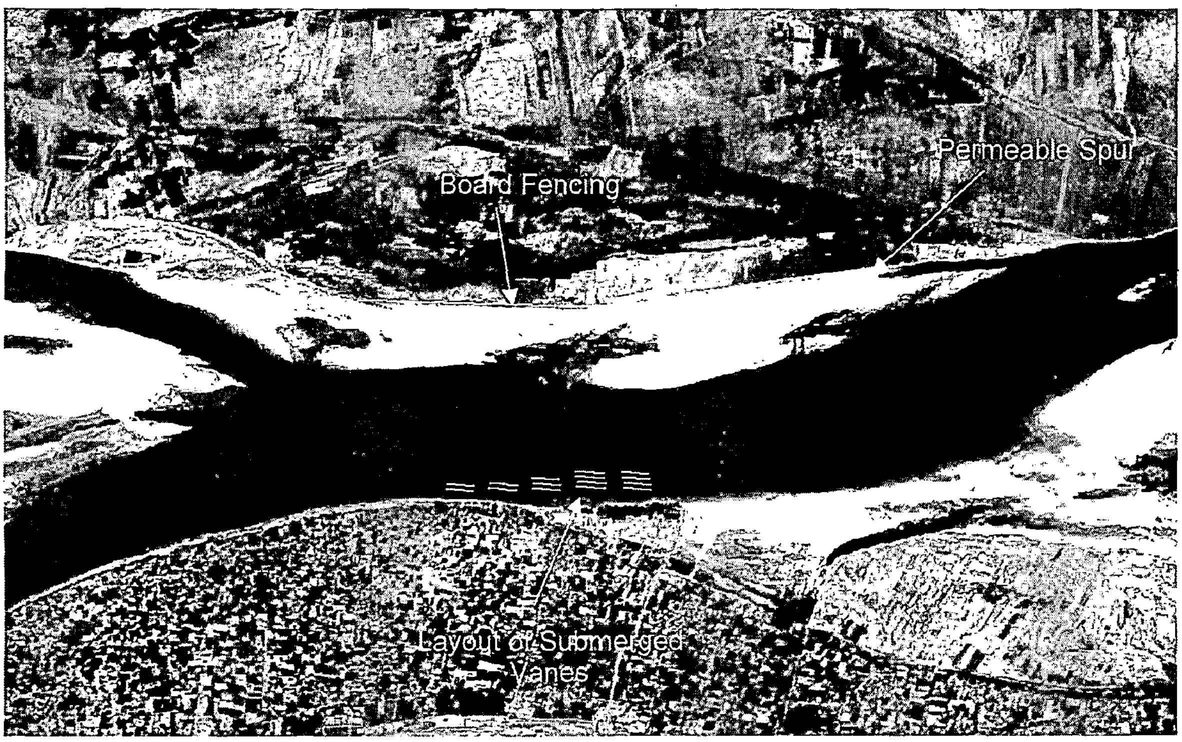


Fig 3.6 Image (2007 Year) of Tentative River Training Structures at PUNARAK (Ch. 879)

3.4 Digha at Ch. 976

In DIGHA site located 12Km upstream of Patna, the developed navigation channel follows a highly sinuous alignment sharply veering from the left bank towards the right. Due to this dominant flow alignment, a sizeable sand shoal got developed along the right bank in proximity of the Mahendru Ghat where the prevalent minimum flow depth is about 0.5m as against corresponding 4m water depth along the left side navigation channel. It is desired by the authorities to secure development of about seven Km long thalweg channel along the left bank from Digha to Mahendru Ghat which will involve shifting of the existing navigation channel to the left bank side. Obviously, this proposition will entail in-depth study involving both physical river modeling along with in-situ field trials for evolving the appropriate river training structures to bring about the desired change in the deep channel alignment. If we will try to develop the right side channel then the slope will increase as the distance will be less with same head. It may inundate the bank. For that we can design a pilot channel. For the purpose of the study at this site, the required river data of discharge, water level and sediment have been used from the observed records at Patna CWC gauging station. At Digha the depth of water is more. So it will be difficult to install submerged vanes there. So there the installation of Geotubes would be a better preposition

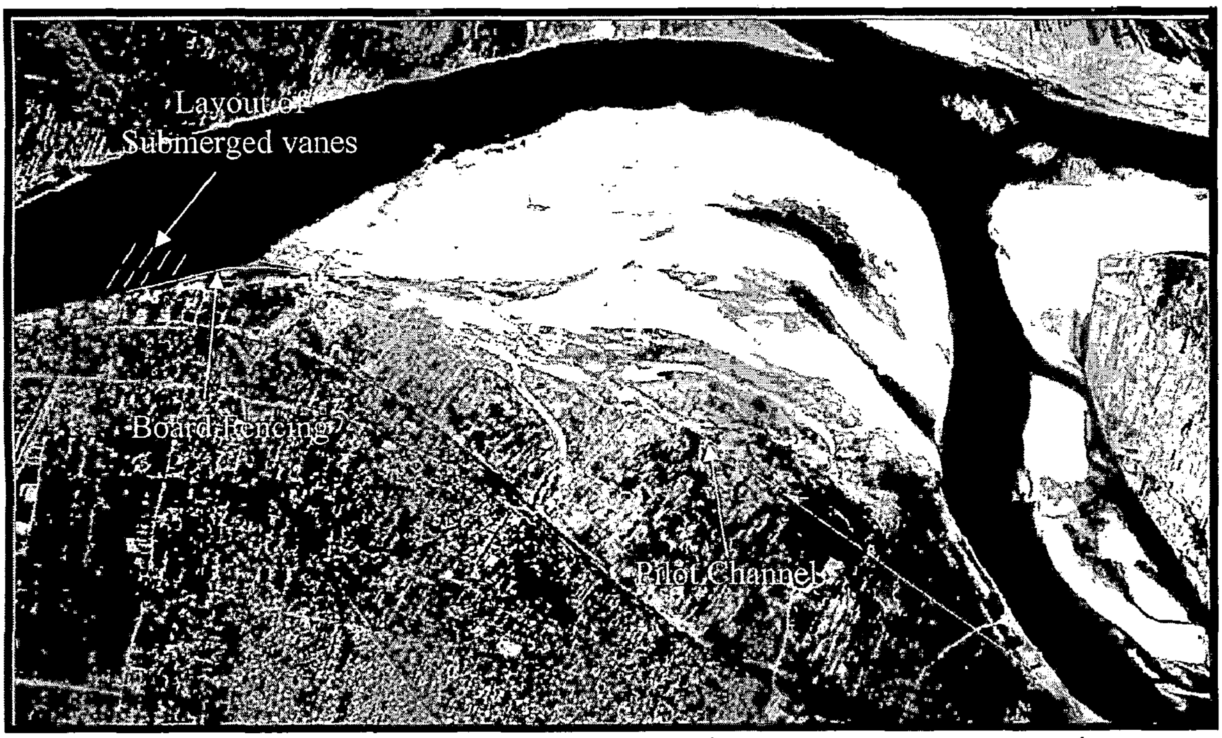


Fig 3.7 ZOOMED Image of Tentative River Training Structures at DIGHA Ch. 976

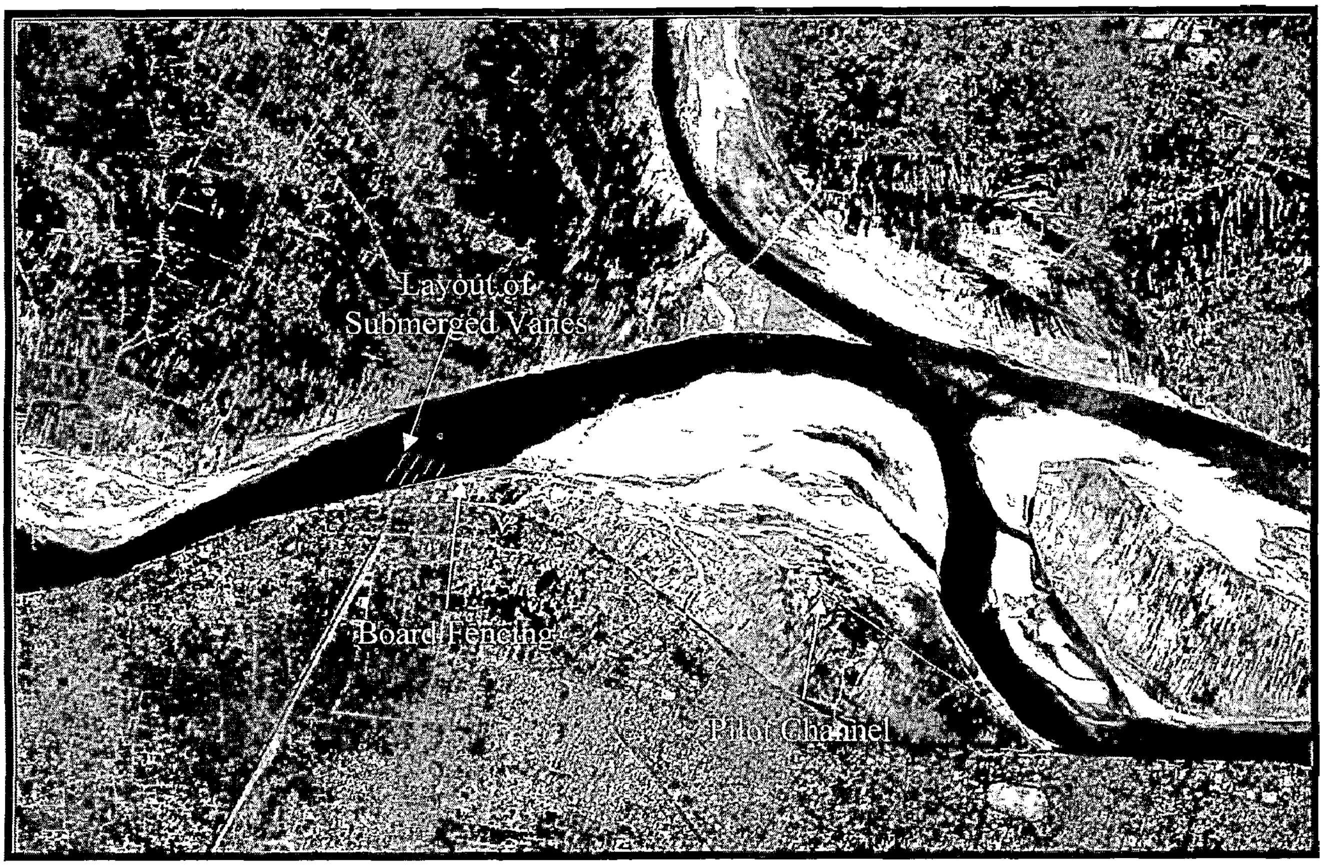


Fig 3.8 MACRO Image of Tentative River Training Structures at DIGHA Ch. 976

Fig 3.9 Image (2007 Year) of Tentative River Training Structures at DIGHA (Ch. 976)

3.5 Arjunpur at Ch.1118

In ARJUNPUR site located 6Km downstream from Buxar & 140Km downstream from Varanasi, the navigation channel depth during lean season period from February to May is about 1m only, thereby posing severe constraint for plying of vessels. The problem is further compounded by occurrence of bank erosion process in inhabited land near Arjunpur along the right bank of the Ganga river. The river plan-form downstream of Buxar fans out into a sort of braided configuration with divided channels and large sand deposits. The presence of a concave curvature of the right bank-line will require bank protection of the order of about 8Km at least, besides channel forming river training structures. For navigation purpose large bends are not favourable. It should be gradual, because the difference of water level between the inner bank and the outer will be less which will be conducive for navigation purpose. So we will try to close the left channel and develop the right channel by placing submerged vanes for a distance of 2km upstream of Arjunpur for a width of 600m making an angle of 20 degree with the direction of flow. The hydrological data of Buxar gauging station have been used for the study at this site.

Fig 3.10 ZOOMED Image of Tentative River Training Structures at ARJUNPUR Ch. 1118

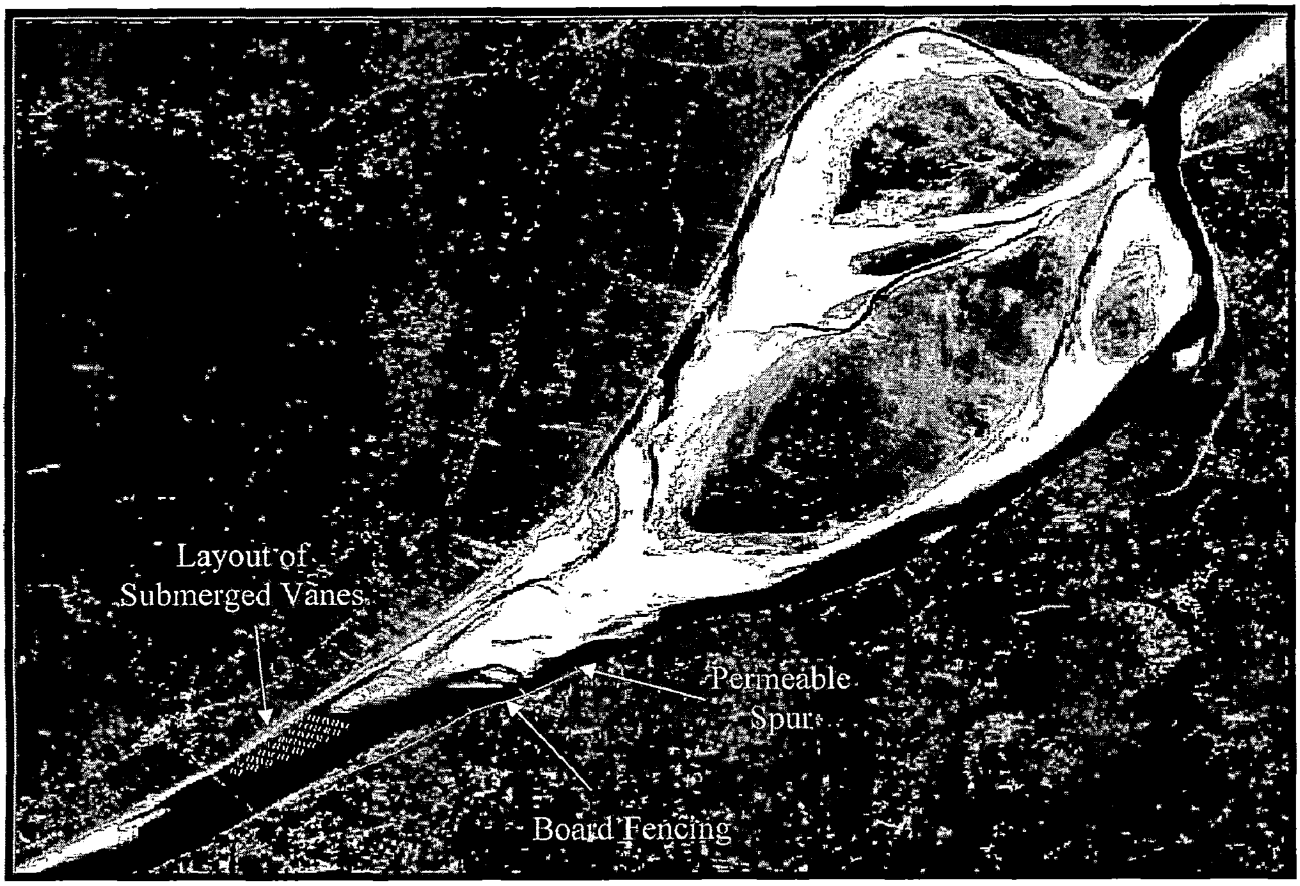


Fig 3.11 MACRO Image of Tentative River Training Structures at ARJUNPUR Ch. 1118

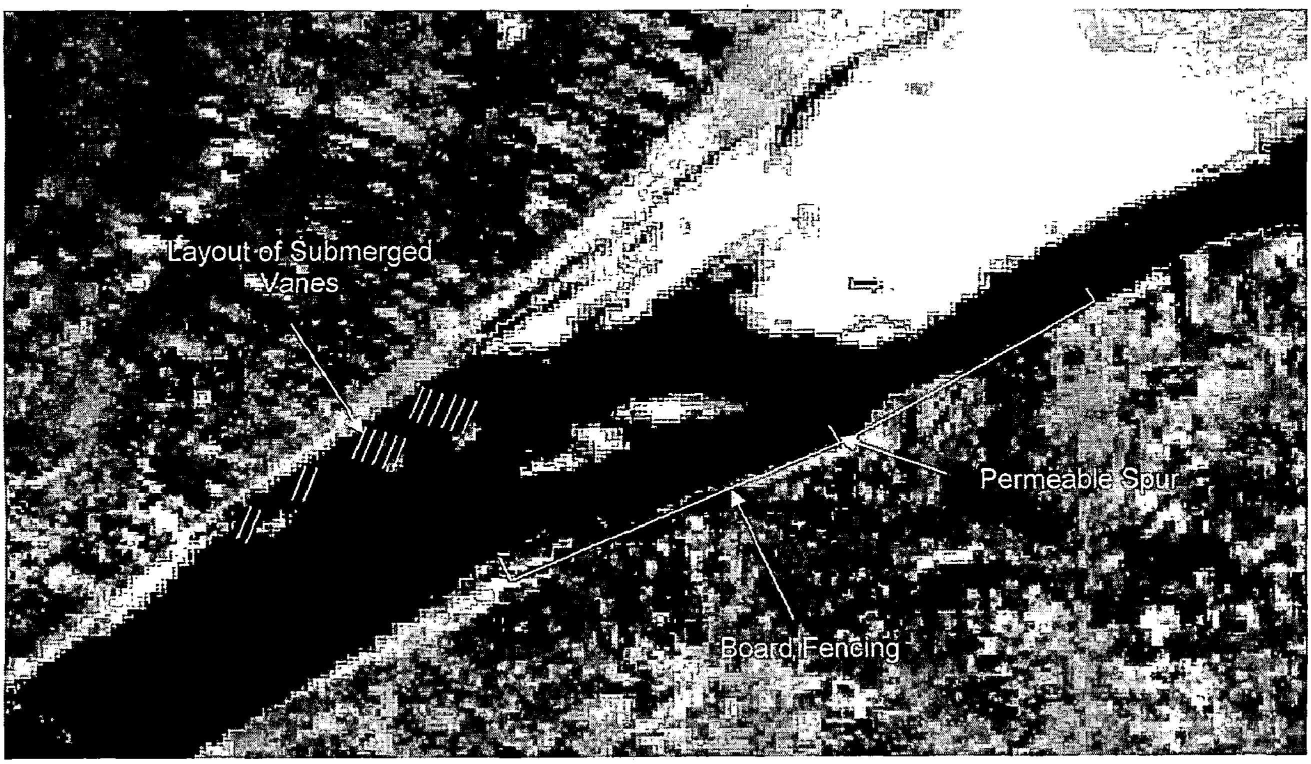


Fig 3.12 Image (2007 Year) of Tentative River Training Structures at ARJUNPUR (Ch. 1118)

3.6 Debchandpur at Ch. 1247

DEOCHANDPUR site is located 18Km downstream of Varanasi, where least available water depth is of the order of 1m, which is usually encountered from February onwards till May. The lean season velocity is about 0.7m/sec. The river bank is firm and the channel is apparently well-defined. During low flow season, as against a required navigation channel width of 45m, the waterway is about 450m which reduces the water depth. A possible approach to attack the problem at this site may be to constrict the waterway to a lesser value say 100m so as to achieve flow concentration during lean period to create required water depth. At least one to two Km length of river may require channelisation at this site. We will try to minimize the flow in the left channel and to develop the right side channel. Bank protection work should be provided for a distance of 2.3km and 300m width in the shoal portion and 3.4km in the curvature portion. For this site, the hydrological data of Varanasi gauging station of CWC have been used.

Fig 3.13 ZOOMED Image of Tentative River Training Structures at DEOCHANDPUR Ch. 1247

Fig 3.14 MACRO Image of Tentative River Training Structures at DEOCHANDPUR Ch. 1247

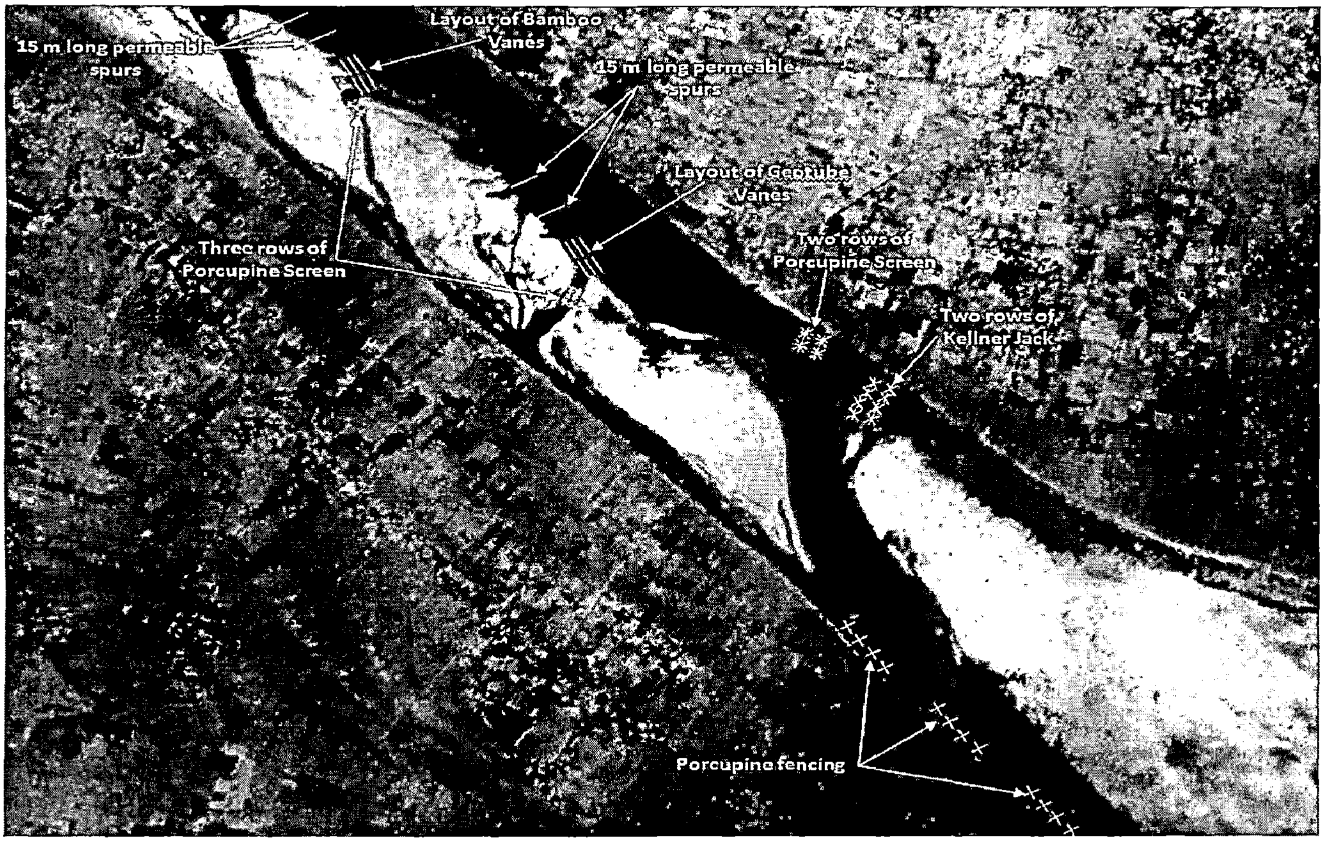


Fig 3.15 Image (2007 Year) of Tentative River Training Structures at DEOCHANDPUR (Ch. 1247)

3.7 Nakhwa at Ch. 1299

NAKHWA site is located 11Km downstream of Varanasi. The stream channel along the right bank is too narrow with only 50m width and generating a velocity of 1.5m/sec. It may be required to explore ways and means to increase the channel width to a value say 100m so that flow conditions conducive for navigation can be induced. Furthermore, the benefit of secondary channel closure needs to be examined for improving the channel hydraulics. We will try to develop the right channel by minimizing the flow in the left channel. this will be done by installing the submerged vanes for a distance of 1.5km and 260m width upstream of Nakhwa . bank protection work should be provided for a distance of 3km in the curvature portion. The river flow data available at CWC gauging station of Varanasi have been used for the study at this site.

Fig 3.16 ZOOMED Upstream Image of Tentative River Training Structures at NAKHWA Ch. 1299

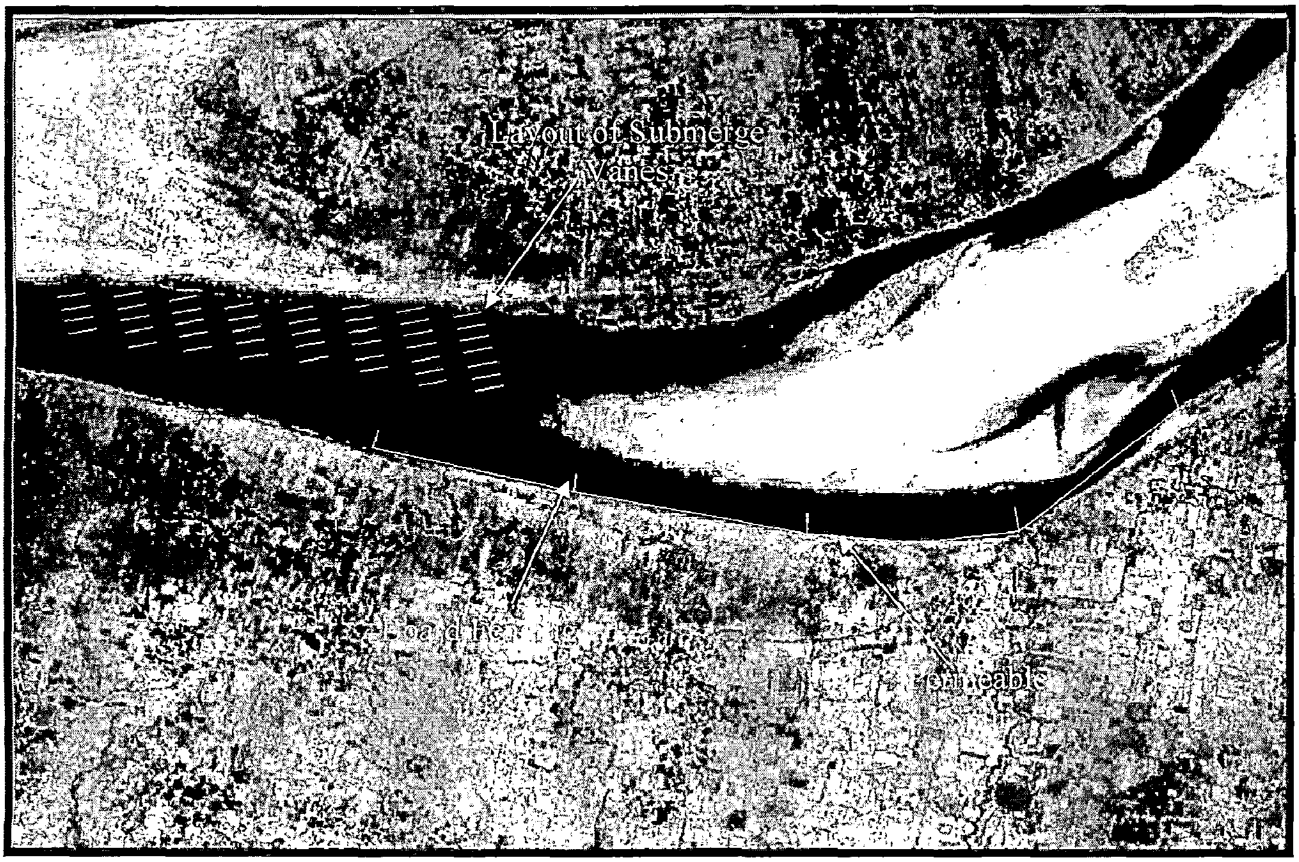


Fig 3.17 ZOOMED Downstream Image of Tentative River Training Structures at NAKHWA Ch. 1299

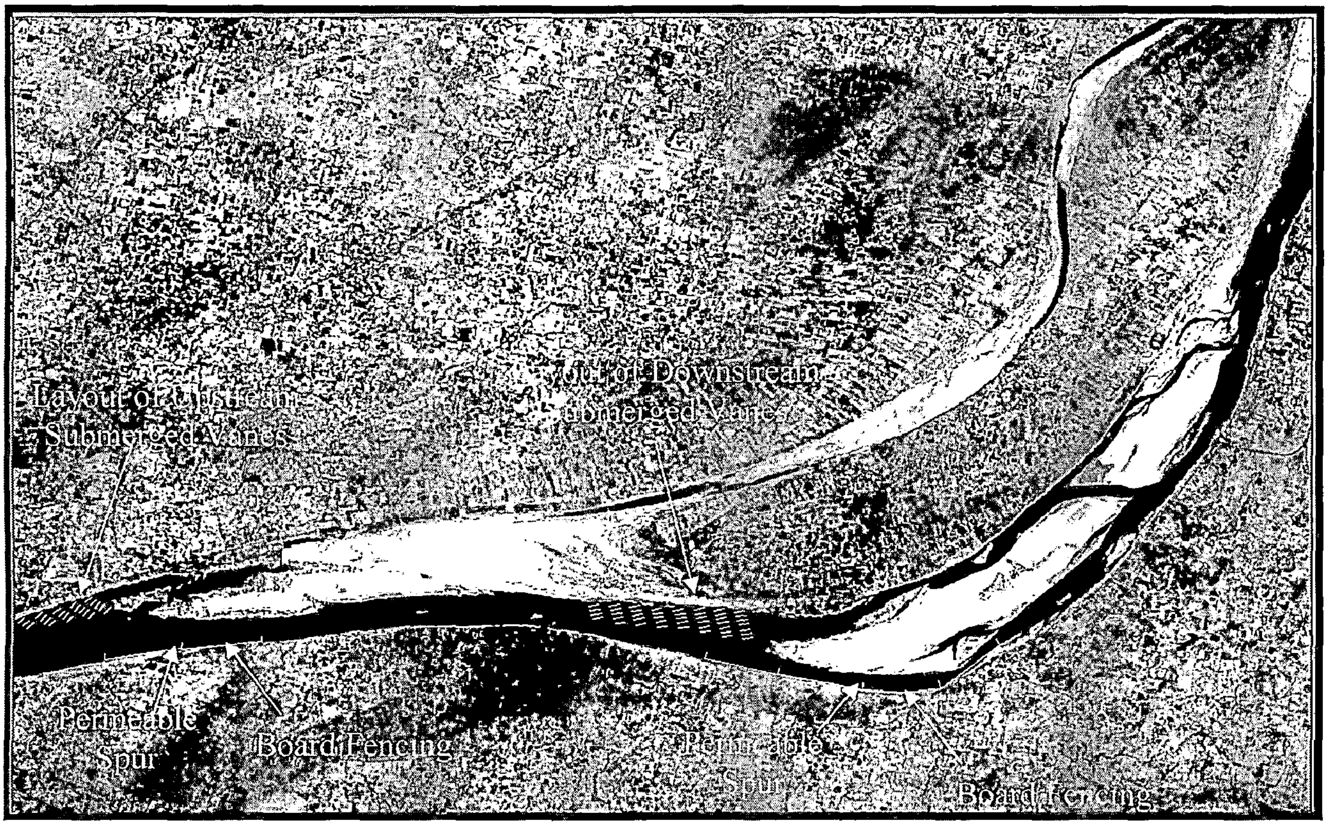


Fig 3.18 MACRO Image of Tentative River Training Structures at NAKHWA Ch. 1299

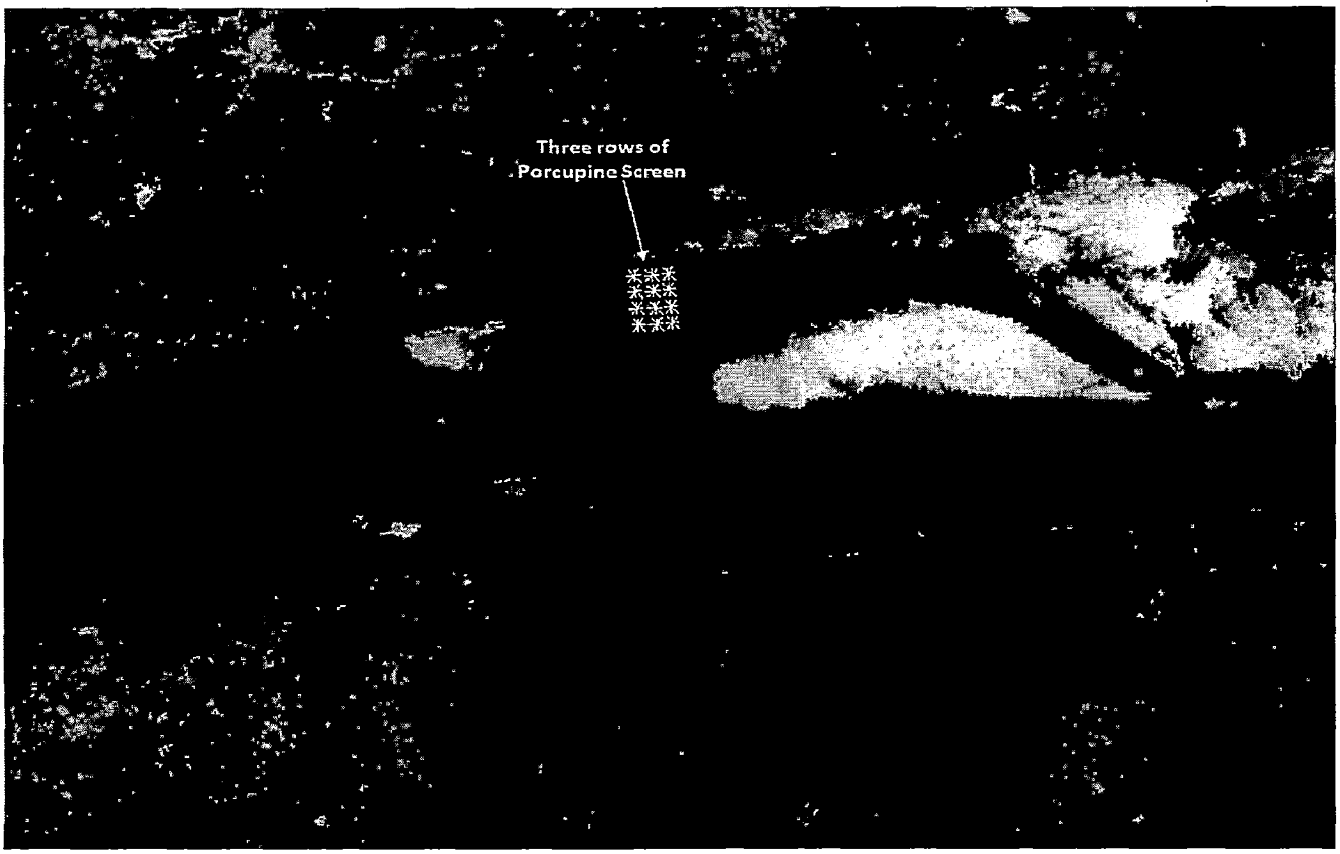


Fig 3.19 Image (2007 Year) of Tentative River Training Structures at NAKHWA Upstream (Ch. 1299)

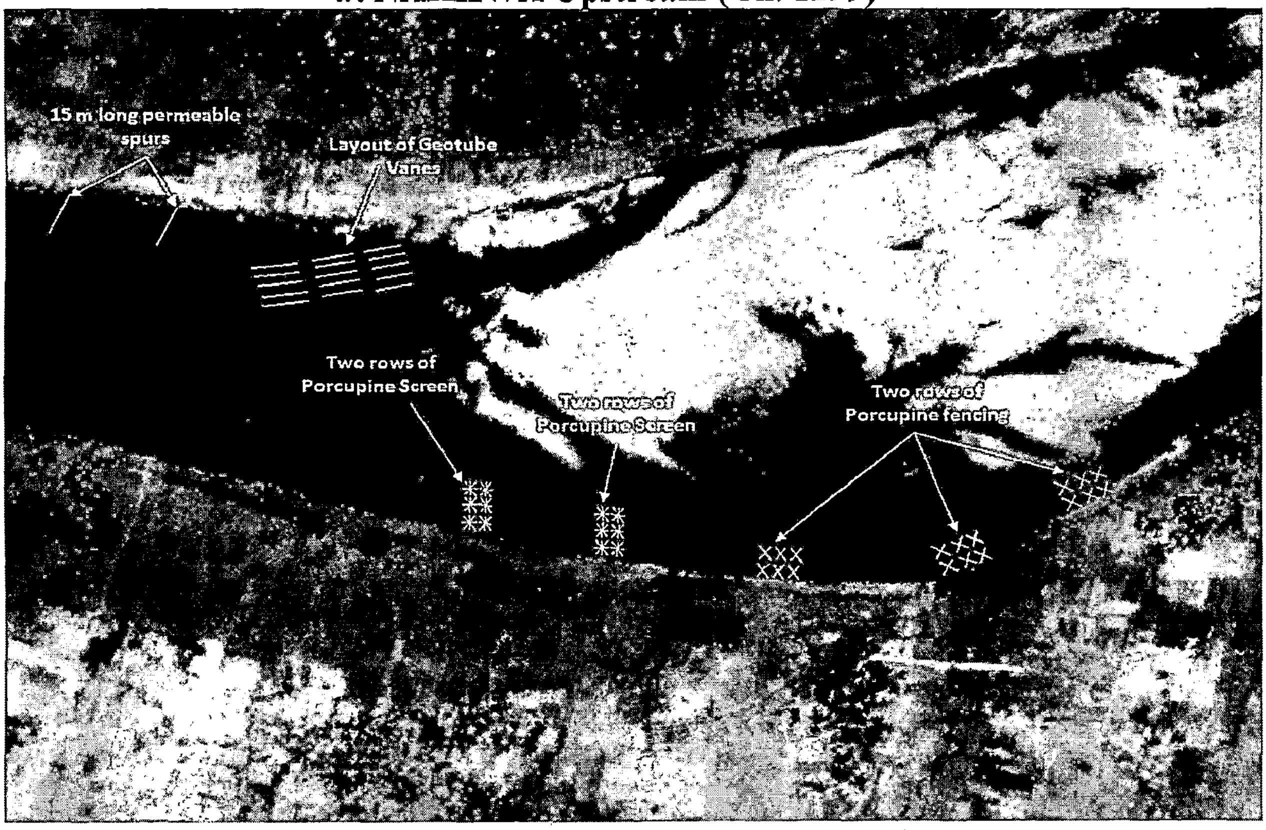


Fig 3.20 Image (2007 Year) of Tentative River Training Structures at NAKHWA Downstream (Ch. 1299)

3.8 Results and Discussion

This initial preliminary study phase has brought out the occurrence of channel flow bifurcation and shoal formation behaviour due to heavy sedimentation at these sites as the root cause of navigation bottlenecks. Stream bank erosion and heavy sediment deposition due to inadequate transport capacity could be identified as major underlying causes leading to channel instability and resultant reduction in draught.

Chapter 4

NUMERICAL FLOW SIMULATION USING HEC-RAS AND HEC-6

4.1 Simulation Methodology Using HEC-RAS and HEC-6

HEC-RAS numerical model developed by Hydrologic Engineering Centre, USA has been used for this study in order to calibrate the model for the values of Rugosity coefficient which have been used in the simulation of the model to predict the changes in channel profiles resulting from scour and/or deposition over rainy season. HEC-RAS is capable to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. This component of the modeling system is intended for calculating water surface profiles for steady/gradually varied flow. The system can handle a single channel reach, or a full network of channels. The steady flow component is capable of modeling sub critical, supercritical, and mixed flow regime water surface profiles.

The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (Le., hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences and channel junctions.

HEC-6 numerical model developed by Hydrologic Engineering Centre, USA has been used for the present study in order to simulate the water surface and bed profile. HEC-6 is a one-dimensional movable boundary open channel flow numerical model designed to simulate and predict changes in channel profiles resulting from scour and/or deposition over moderate time periods. A continuous flow record is partitioned into a series of steady flows of variable discharges and durations. For each flow a water surface profile is calculated thereby providing energy slope, velocity, depth, etc. at each cross section. Potential sediment transport rates are then computed at each section.

HEC-6 processes a discharge hydro graph as a sequence of steady flows of variable durations. Using continuity of sediment, changes are calculated with respect to time and distance along the study reach for the following: total sediment load, volume and gradation of sediment that is scoured or deposited, armouring of the bed surface and the cross section elevations. In addition, sediment outflow at the downstream end of the study reach is calculated. The location and amount of material to be dredged can be obtained if desired.

4.2 **Geometry**

Geometry of the channel system is represented by cross sections which are specified by coordinate points (stations and elevations) and the distances between cross sections. HEC6 raises or lowers cross section elevations to reflect deposition and scour. The horizontal locations of the channel banks are considered fixed; however, they will be moved vertically if they are within the movable bed limits specified by the user.

4.3 **Hydraulics and Hydrology**

The water discharge hydrograph is approximated by a sequence of steady flow discharges, each of which continues for a specified period of time. Water surface profiles are calculated for each flow using the standard-step method to solve the energy and continuity equations. Friction loss is. calculated by Manning's equation and expansion and contraction losses are calculated if the loss coefficients are specified. Hydraulic roughness is described by Manning's n values and can vary from cross section to cross section.

4.4 **Sediment Transport**

Inflowing sediment loads are related to water discharge by sediment-discharge curves for the upstream boundaries of the main channel and local inflow points. For realistic computation of stream behaviour, particularly scour and stable conditions, the gradation of the material forming the stream bed must be measured. HEC-6 allows a different gradation at each cross section. If only deposition is expected, the gradation of material in the bed is less important. Sediment gradations are classified by grain size using the American Geophysical Union scale. HEC-6 will compute transport potential for clay (particles less than 0.004 mm diameter), four classes of silt (0.004-0.0625 mm), five classes of sand (from very fine sand, 0.0625 mm, to very coarse sand, 2.0 mm), five classes of gravel (from very fine gravel, 2.0 mm, to very coarse gravel, 64 mm), two class of cobbles (from small, 64mm, to large cobbles, 256mm) and three classes of boulders (from small, 256mm, to large boulders, 2048mm). Transport potential is calculated at each cross section using hydraulic information from the water surface profile calculation (e.g., width, depth, energy slope, and flow velocity) and the gradation of bed material. Sediment is routed downstream after the backwater computations are made for each successive discharge (time step). Theoretical Basis for Hydraulic Calculations

The basis for water surface profile calculations is essentially the method which is. described in

"Backwater Curves in Channels. Conveyance is calculated from average areas and average hydraulic radii for adjacent cross sections.

4.5 Sediment Transport

When the average shear stress on the bed of an alluvial channel exceeds the critical tractive stress for the bed material, statistically the particles on the bed may begin to move in the direction of the flow as per generally accepted. The particles move in different modes depending on the flow conditions, the ratio of the densities of the fluid and the sediment, and the size of the sediment.

The total load is the sum of the bed load, suspended load and wash load. In flume experiments (except in special cases) wash load is absent and total load would be the bed-material load. On the other hand, in natural streams wash load is invariably present and the total load is the summation of the bed-material load and the wash load. Further, it is difficult to separate out the wash load from the measured total load. Thus the difference in the nature of laboratory and field data, mentioned above, makes it difficult to unify the total load data collected in the laboratory and in the field. But a majority of the total load relationships are based mainly on flume data and river data in which wash load is estimated and excluded from the measured total load; as such these relationships can be expected to yield the bed-material load.

The methods of computation of the total sediment transport rate can be broadly classified into two categories. The methods under the first category, described as microscopic methods, subdivide the total sediment load either into suspended load and bed load, or into measured load and unmeasured load. Since the suspended sediment sampler cannot sample the entire depth of the stream, some suspended load and the bed load remain to be sampled; this is designated as unmeasured load. Addition of the measured and the unmeasured load gives the total load. A method such as Einstein's estimates both suspended load and bed load by analytical means; on the other hand, methods such as Modified Einstein-Procedure make use of some sediment measurements. The advocates of the methods of the second category argue that the process of suspension is an advanced stage of traction along the bed; therefore the total sediment transport rate should be related primarily to the shear parameter and no distinction needs to be made between bed load and suspended load. These methods can be described as macroscopic.

4.5.1Microscopic Methods

The following microscopic methods may be listed after perusal of the literature in this field: Einstein, Colby and Hembree, Van Rijn, Samaga et al. and Swamee and Ojha. It may be mentioned that the method of Colby and Hembree is a modification of Einstein's method and usually goes under the name of Modified Einstein-Procedure.

4.5.2 Macroscopic Methods Based on A Single Size

Several methods of a macroscopic nature using a single representative size of bed material have been proposed for the determination of the total load transport. The following macroscopic methods may be listed after perusal of the literature in this field: Laursen's Method: Engelund and Hansen's Method, Ackers and White's Method, Yang's Equation

Laursen considered the following parameters to be important in the study of total sediment transport: u_{*}/ω_0, d/D the total load concentration C in per cent by weight and the ratio of grain shear stress τ_0' to the critical shear stress τ_{0c}'for the given sediment size. His intuitive analysis gave the following functional relationship:

$$\frac{\overline{C}}{\left[\frac{d}{D}\right]^{7/6}\left[(\tau_0'/\tau_0c)-1\right]} = f\left(\frac{u_*}{\omega_0}\right) \quad \dots\dots\dots \quad 2.8$$

The effective shear stress τ_0' was computed from the Manning-Strickler equation and the critical shear stress τ_{0c}' from Shields' curve given in figure 2.5.

He used primarily flume data to determine the relation between the two parameters in Eq. (2.8); see Fig. 2.4. Application of this relation to rivers by different investigators has shown large errors.

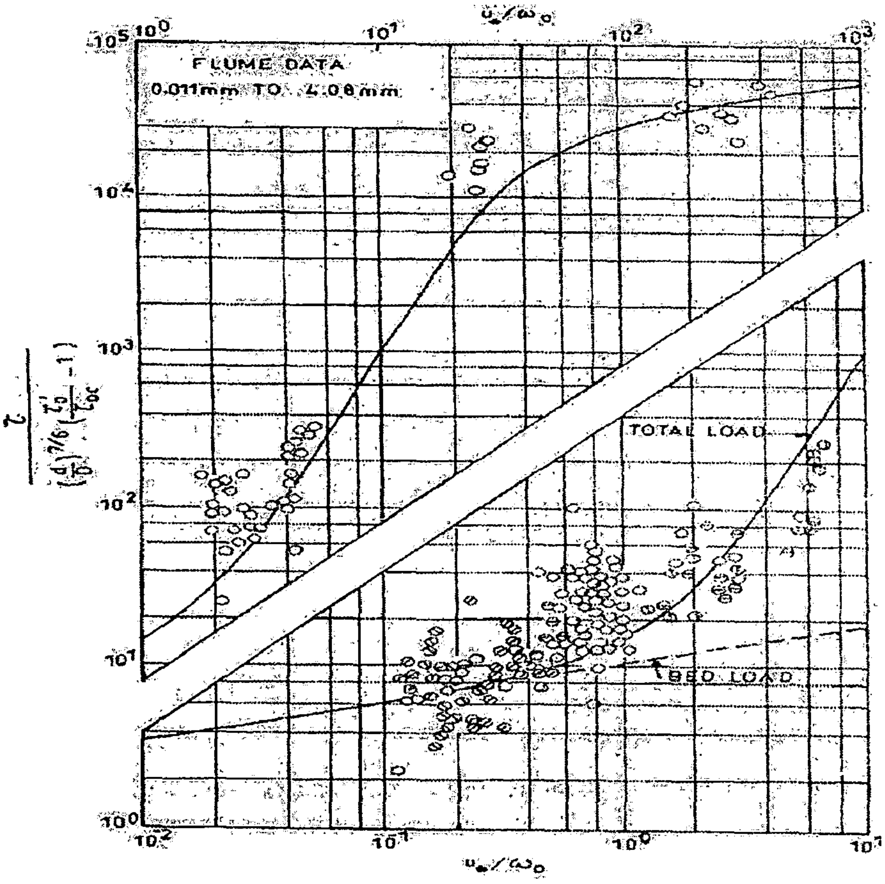


Fig. 4.1 Laursen's Total Load Relation

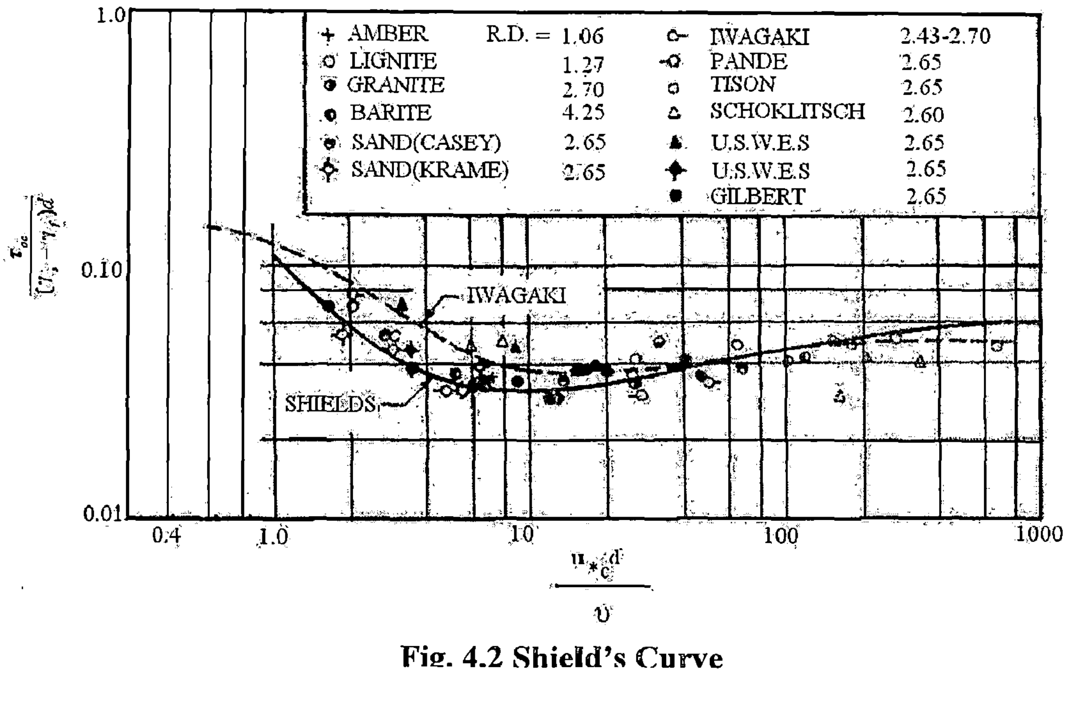


Fig. 4.2 Shield's Curve

Yang's Equation

Yang contended that the rate of sediment transport in an alluvial channel is primarily governed by the rate of expenditure of potential energy per unit weight of water, i.e. the unit stream power. By analysis of a vast amount of data he obtained the following relation for CT, the load concentration in ppm by weigh.

$$\log CT = 5.435 - 0.286\log\frac{\omega_0 d}{\upsilon} - 0.45710gu^{*} +$$

$$\left[1.799 - 0.409\log\frac{\omega_o d}{\upsilon} - 0.314\log\frac{u_*}{\omega_o}\right]\log\left[\frac{US}{\omega_o} - \frac{U_{cr}}{\omega_o}\right]$$

Here U_{cr} is the critical velocity for incipient. Equation (2.11) was found to work satisfactorily both for laboratory and field data.

4.5.3 Comments on the Accuracy of Various Relations

Over a dozen total load relations essentially using a single representative size of the sediment mixture are available. While some of the methods may be considered semi empirical, most of them are based on dimensional analysis and graphical plotting or regression analysis. Hence the basis for the choice of an appropriate sediment transport relation in practice can only be the relative accuracy of these methods. A few checks on the accuracy of practically all the methods available so far have been carried out using both field and laboratory data. Generally speaking, the methods of Ackers-White,Engelund-Hansen and Yang give better results than the other methods.

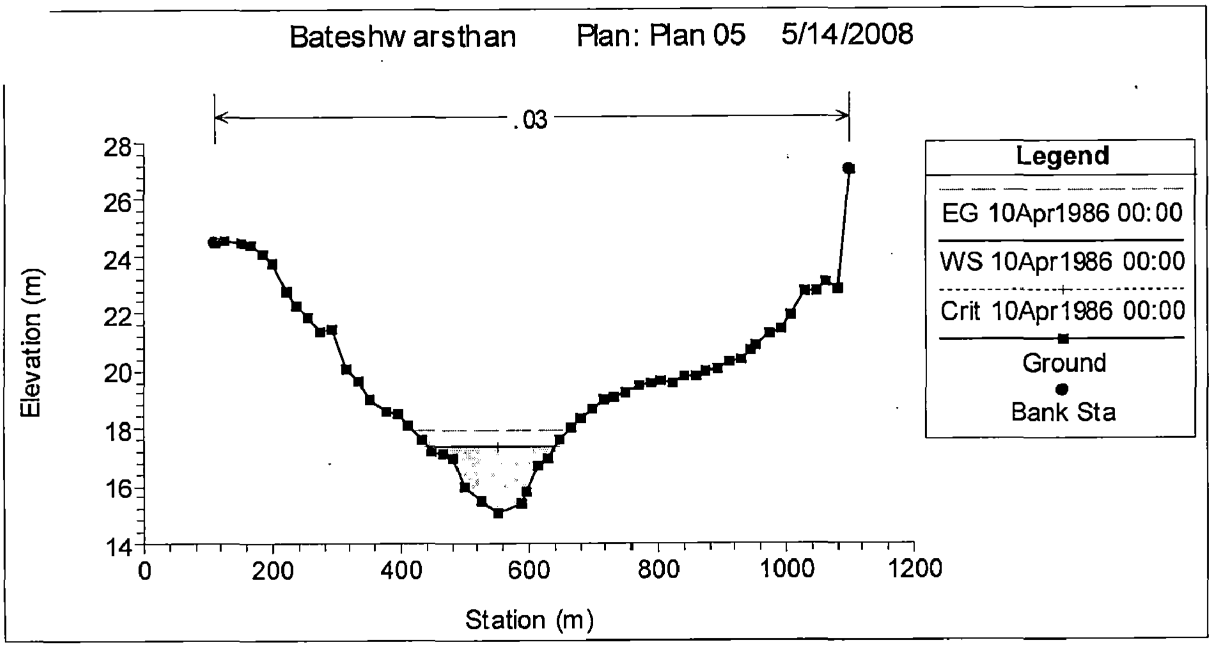


Fig 4.3 Cross Section at 683.787

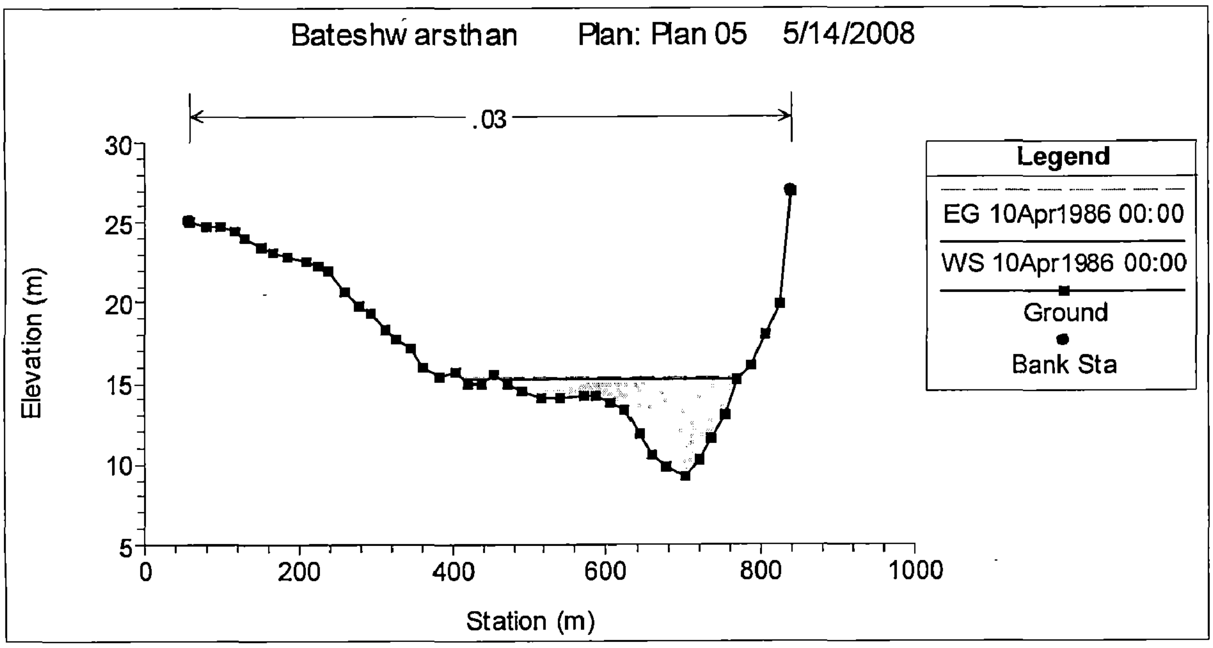


Fig. 4.4 Cross Section at 683.183

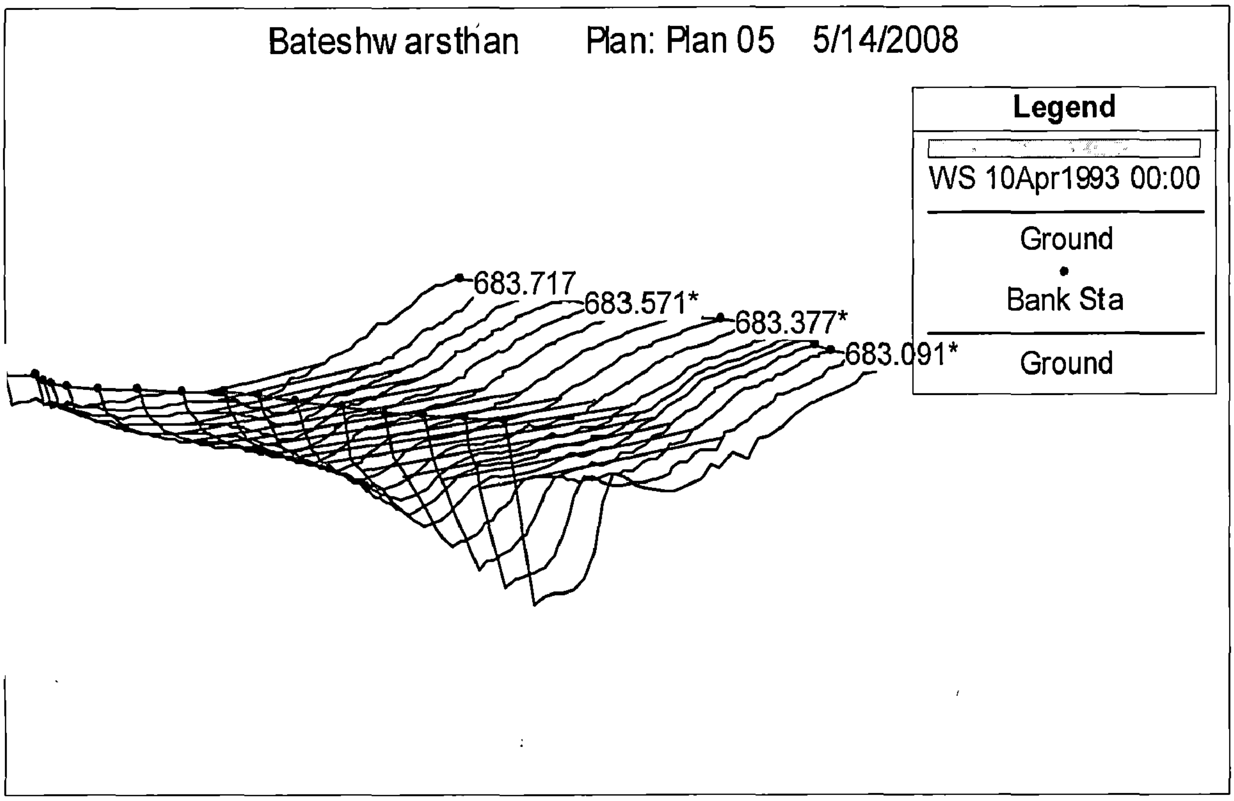


Fig. 4.5 Hec-Ras plot for Bateswarsthan showing water surface profile

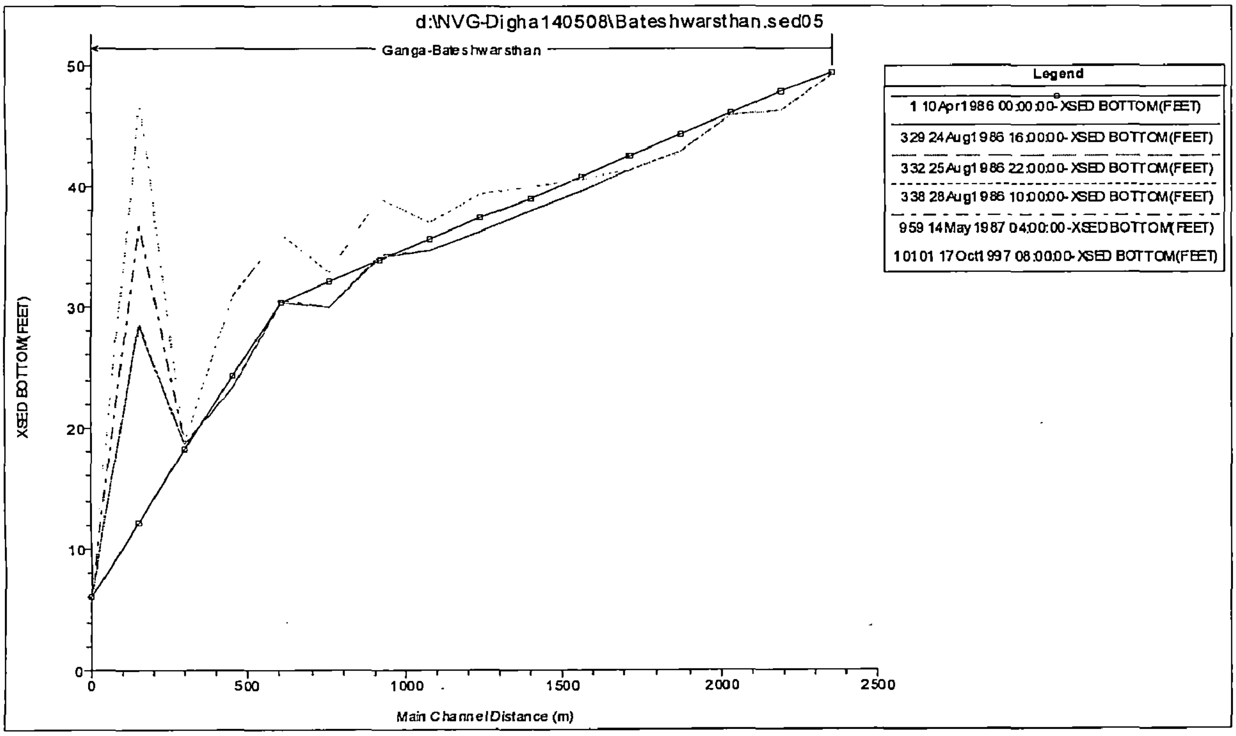


Fig.4.6 Hec-Ras plot for Bateswarsthan showing Bed profile

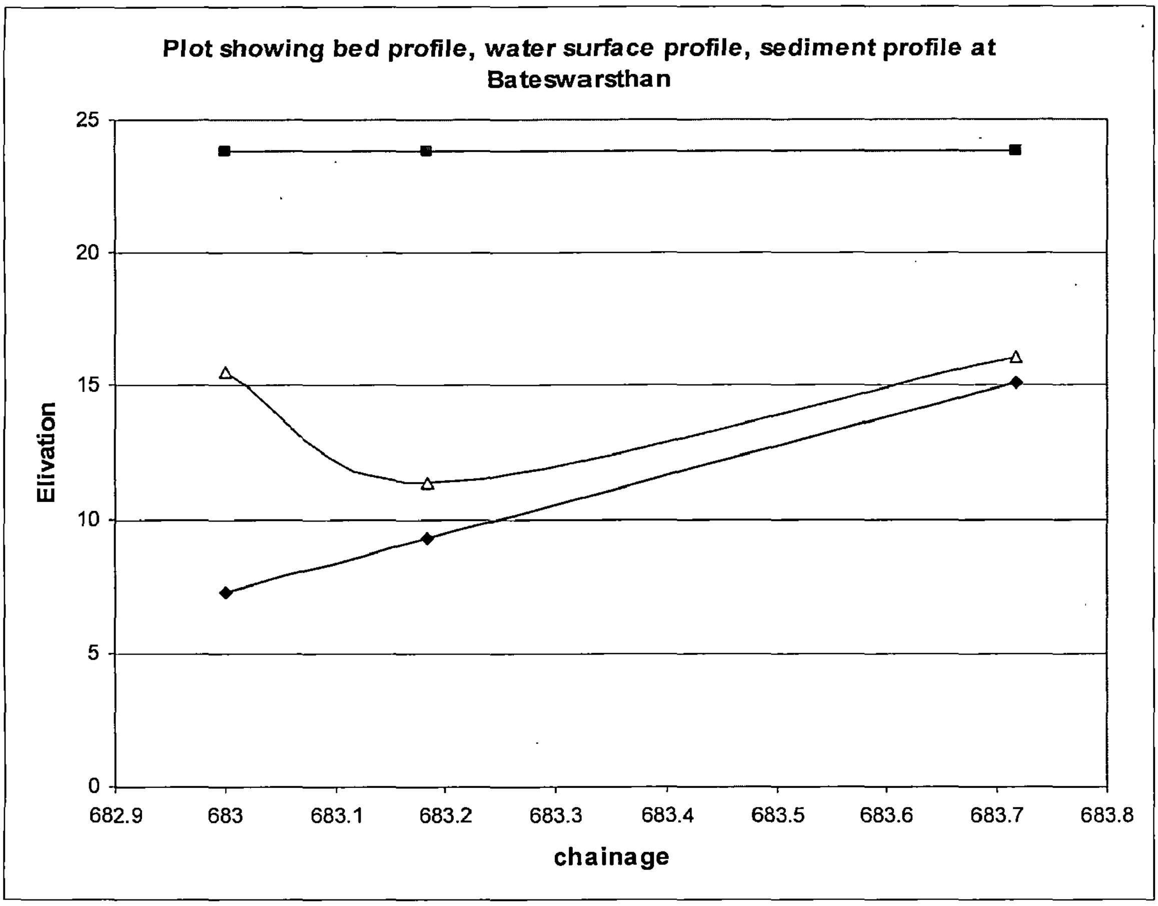


Fig. 4.7 Hec-6 plot for Bateswarsthan showing Bed profile

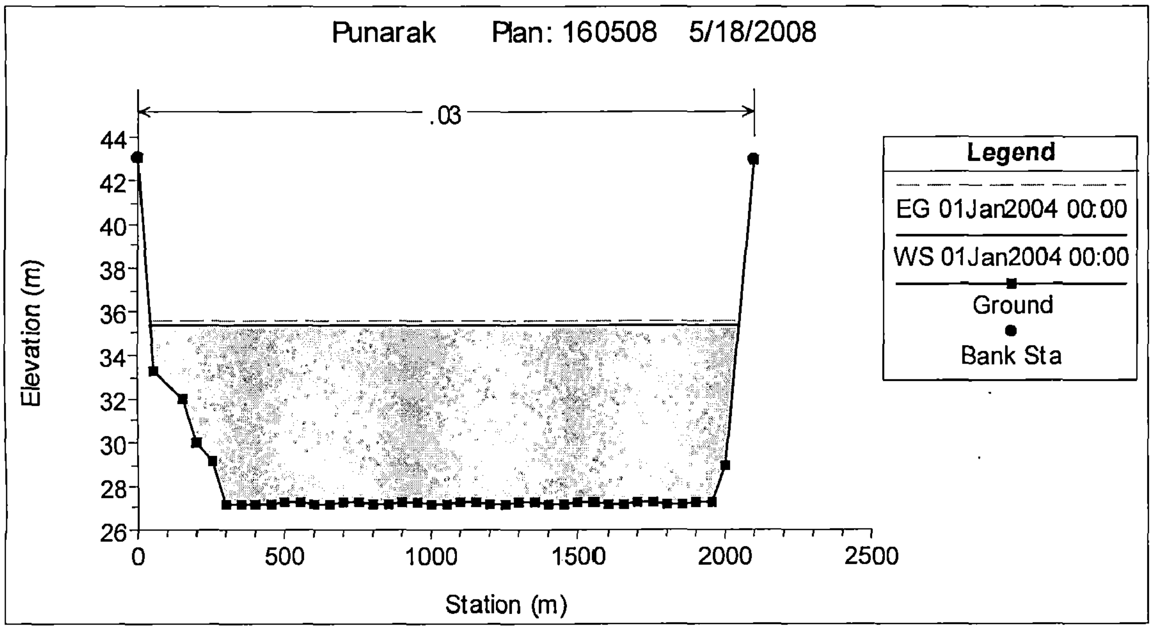


Fig. 4.8 Cross Section at 880km

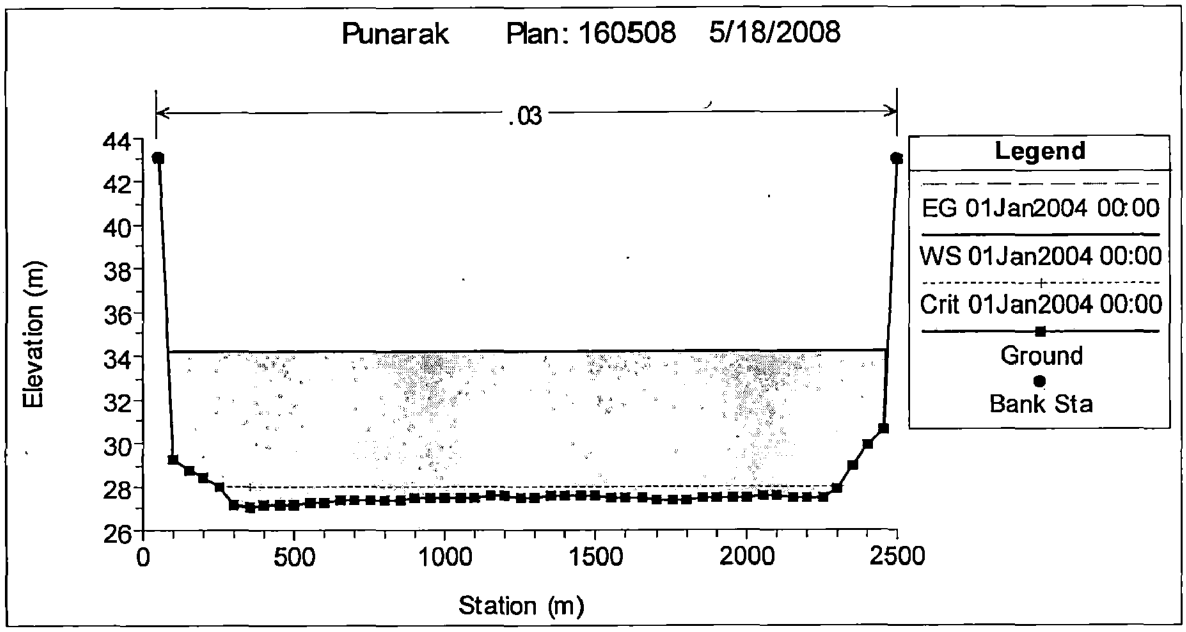


Fig. 4.9 Cross Section at 879km

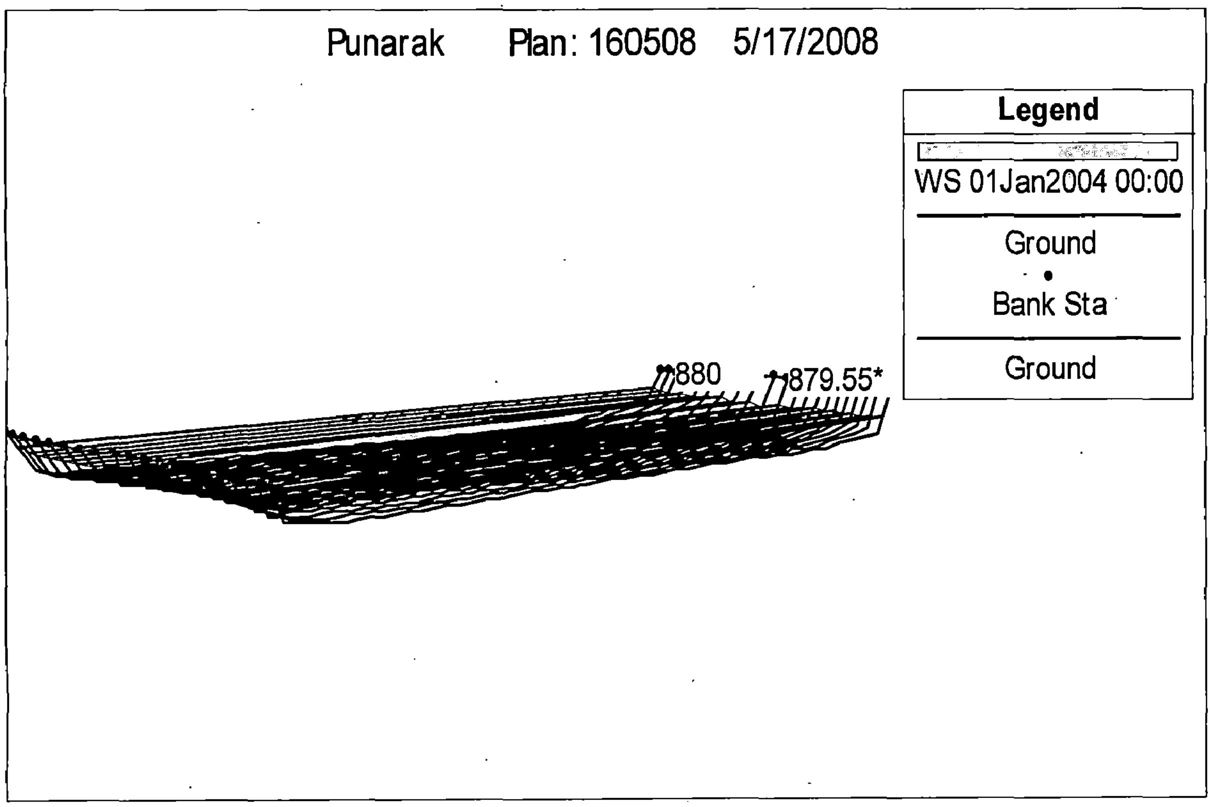


Fig. 4.10 Hec-Ras plot for Punarak showing water surface profile

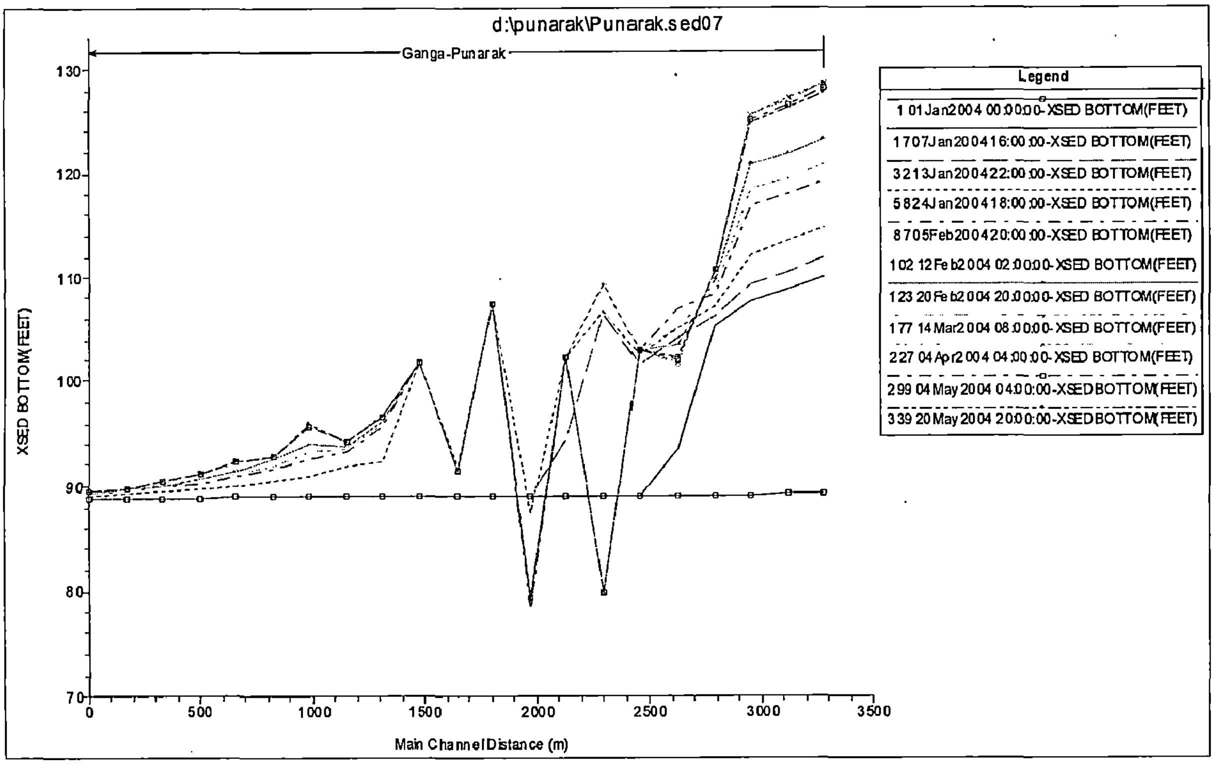


Fig. 4.11 Hec-Ras plot for Punarak showing Bed profile

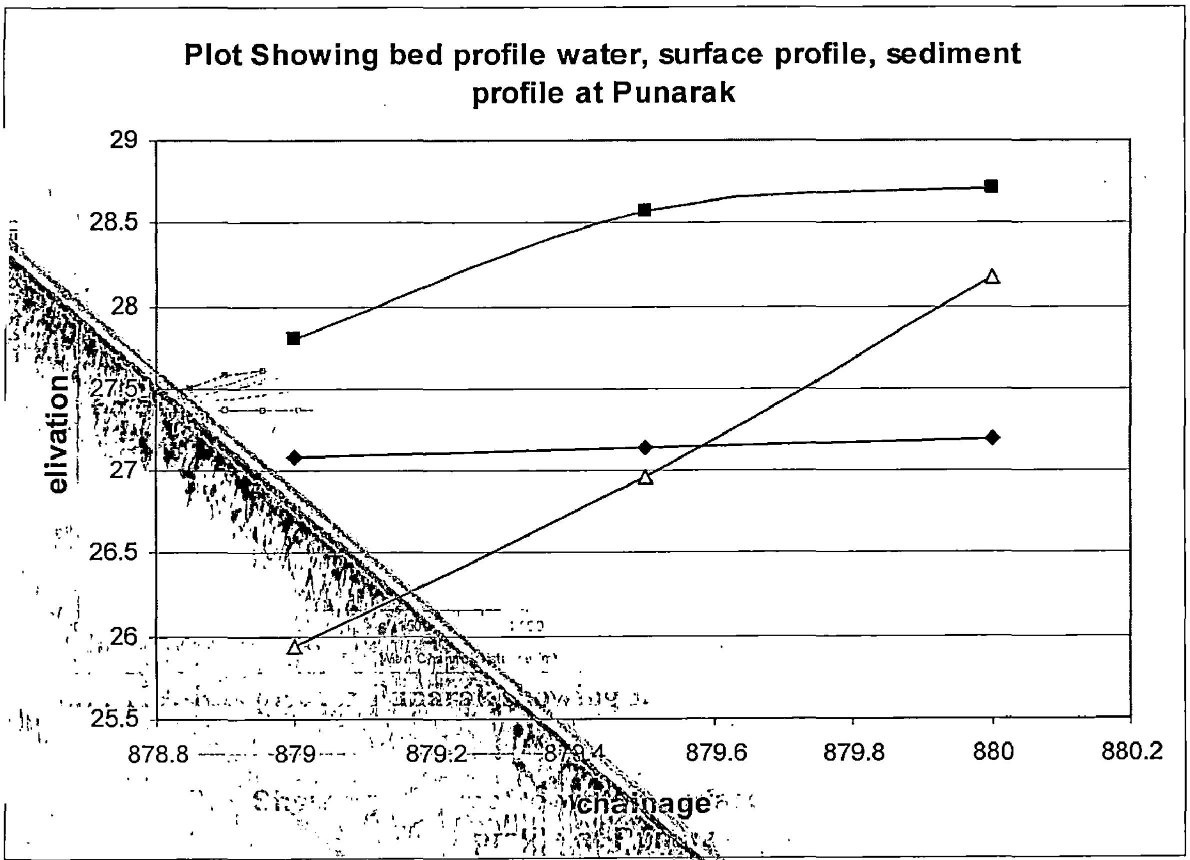


Fig. 4.12 Hec-6 plot for Punarak showing Bed profile

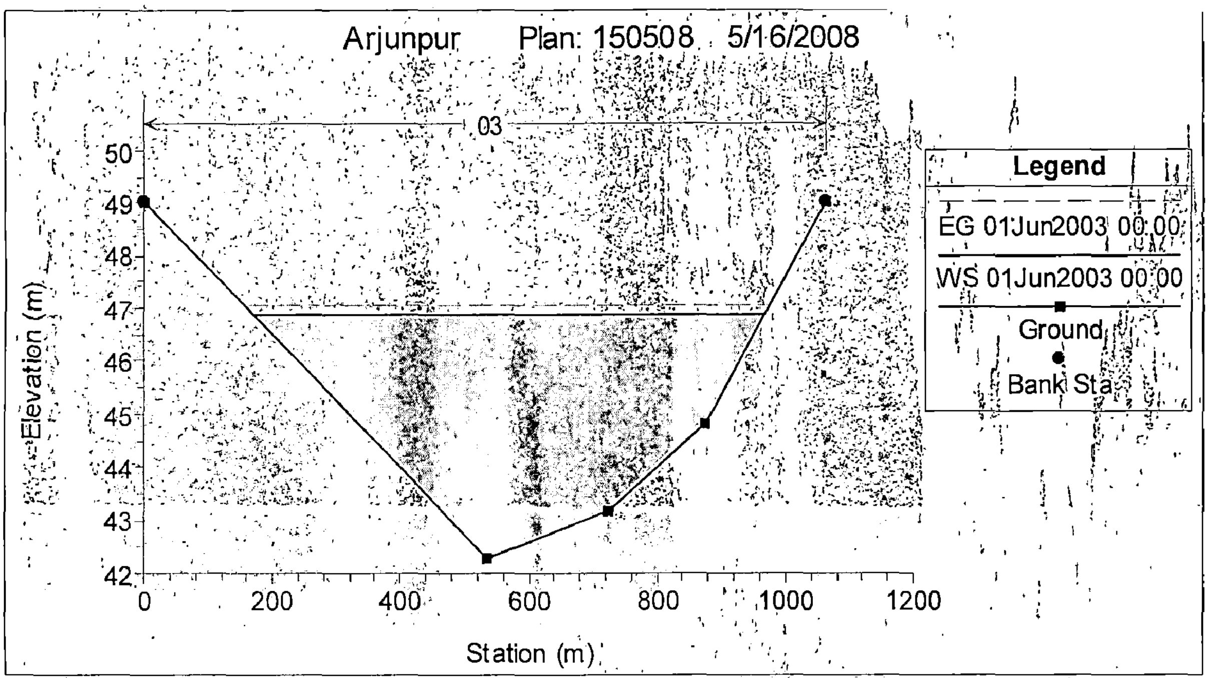


Fig. 4.13 Cross Section at 1121km

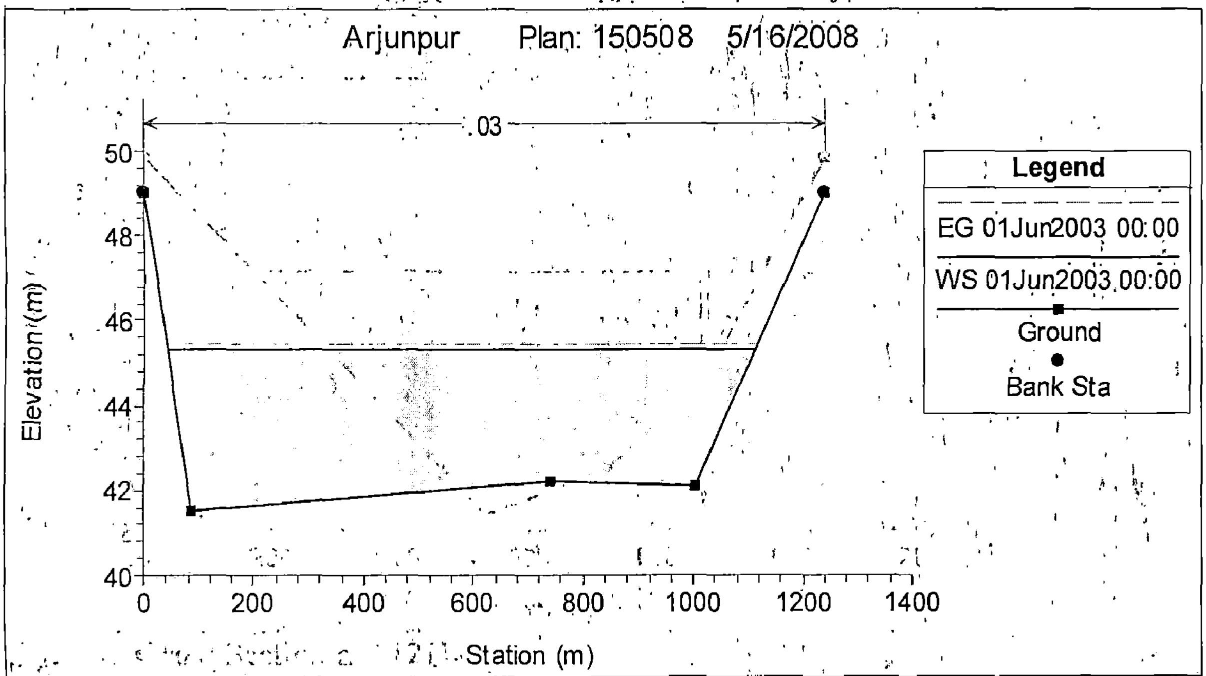


Fig. 4.14 Cross Section at 1119.67km

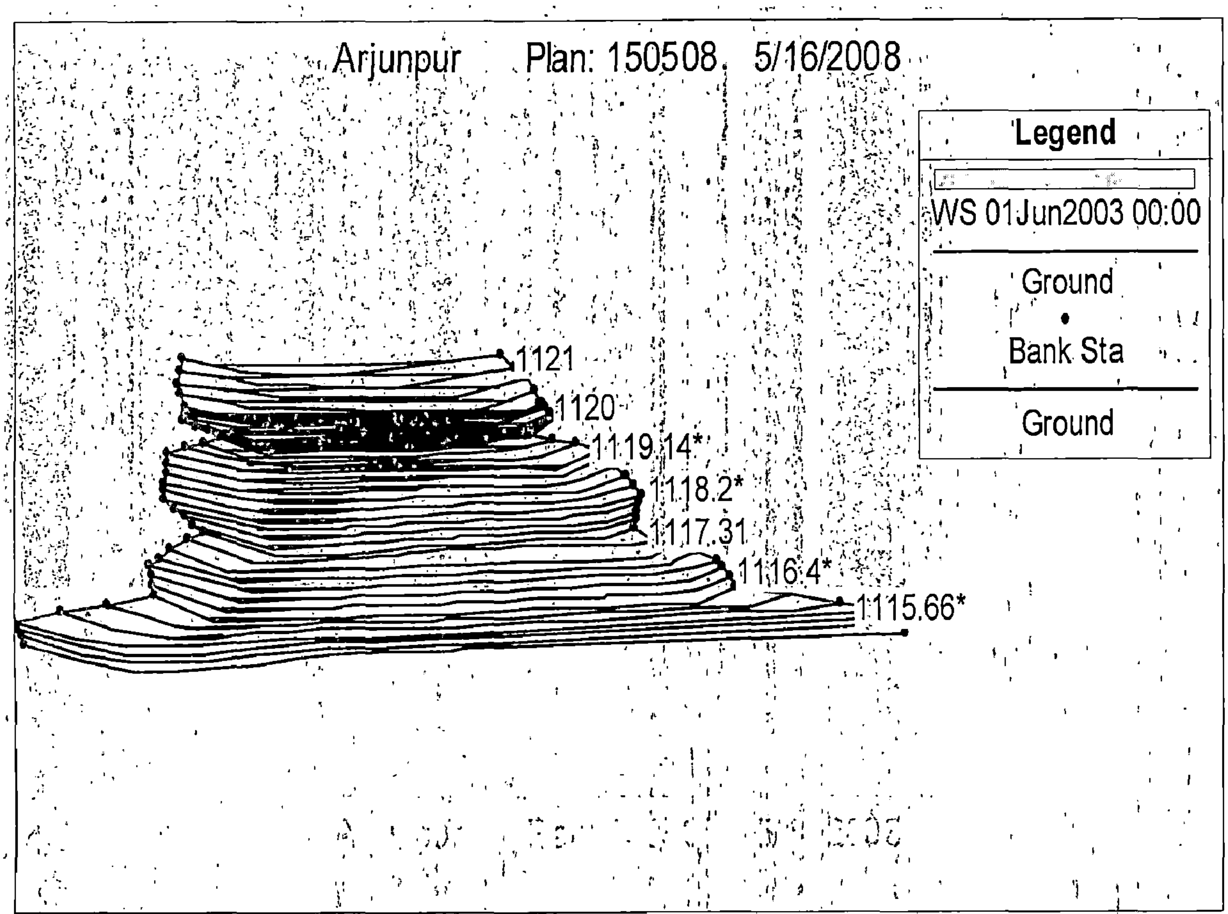


Fig 4.15 Hec-Ras plot for Arjunpur showing water surface profile

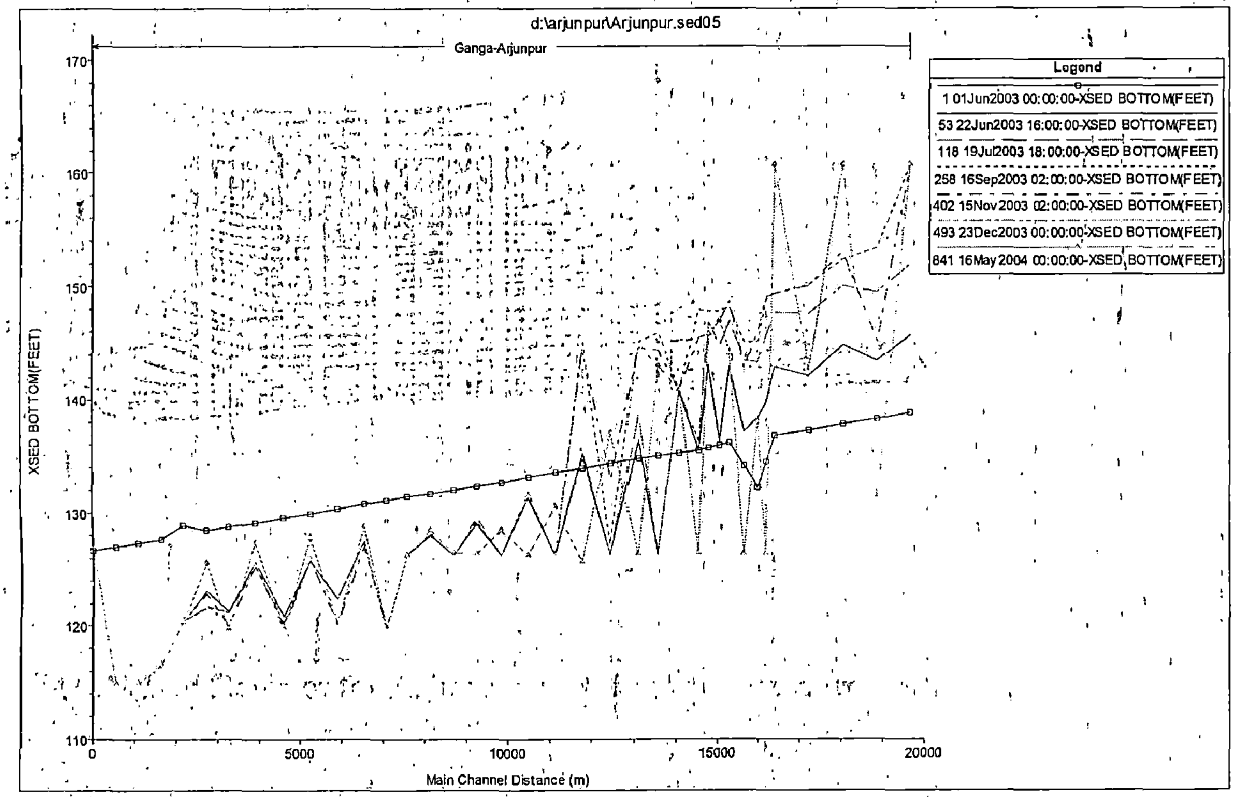


Fig. 4.16 Hec-Ras plot for Arjunpur showing Bed profile

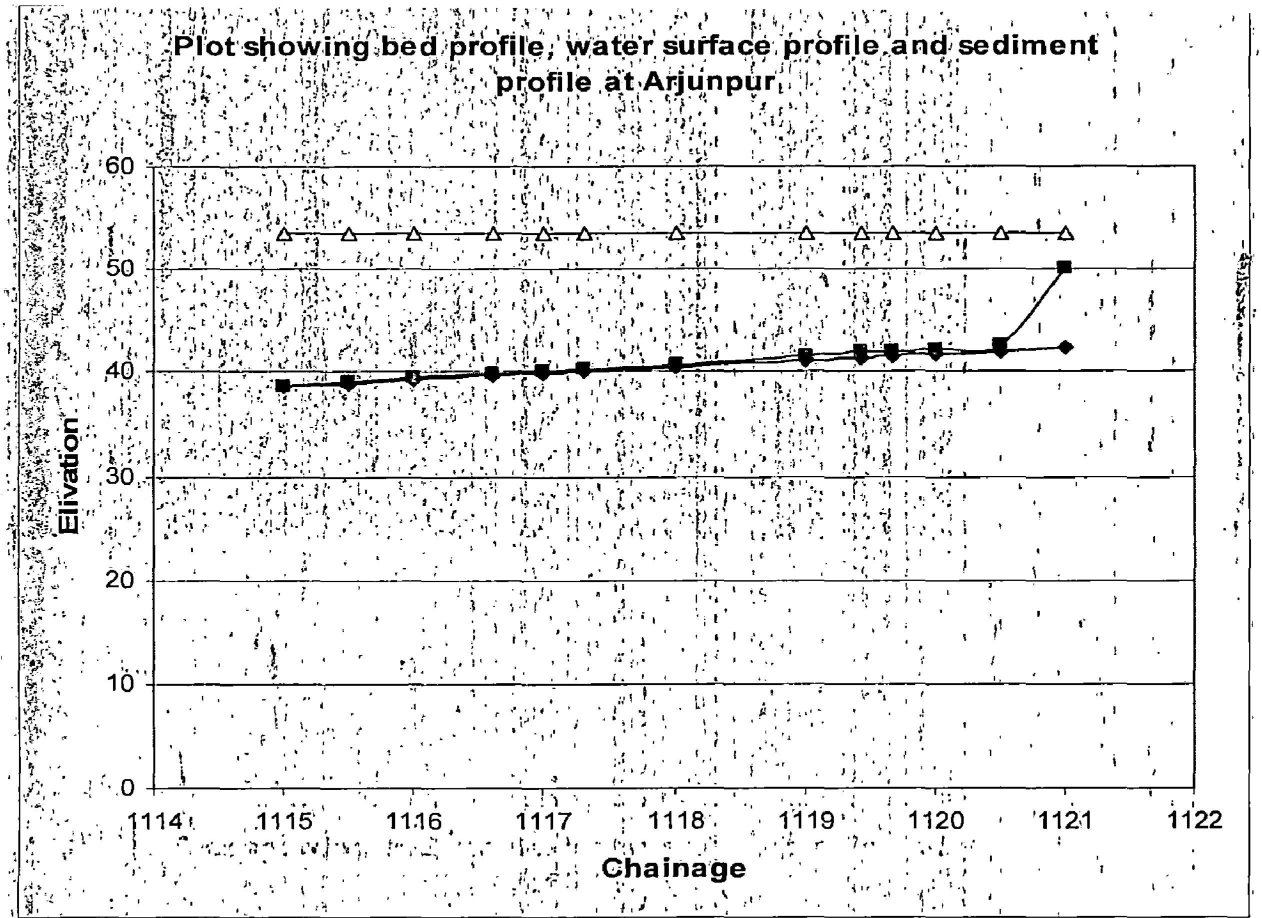


Fig 4.17 Hec-6 plot for Arjunpur showing Bed profile

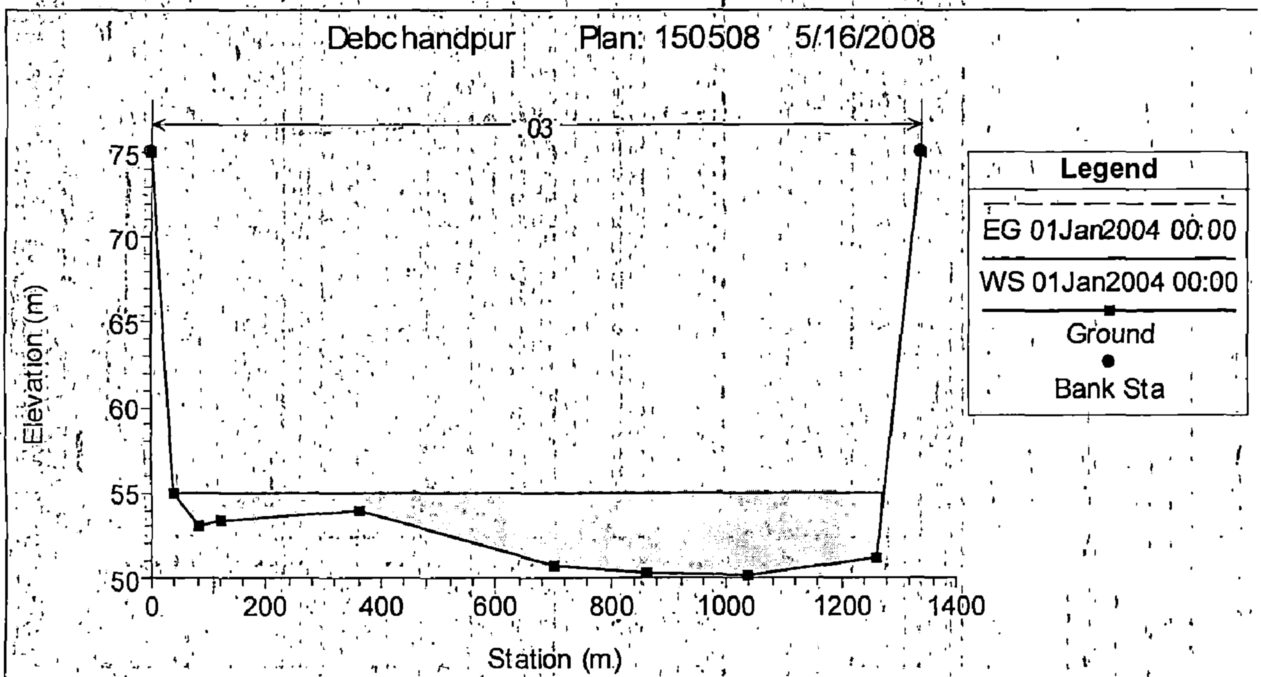


Fig. 4.18 Cross Section at 1253km

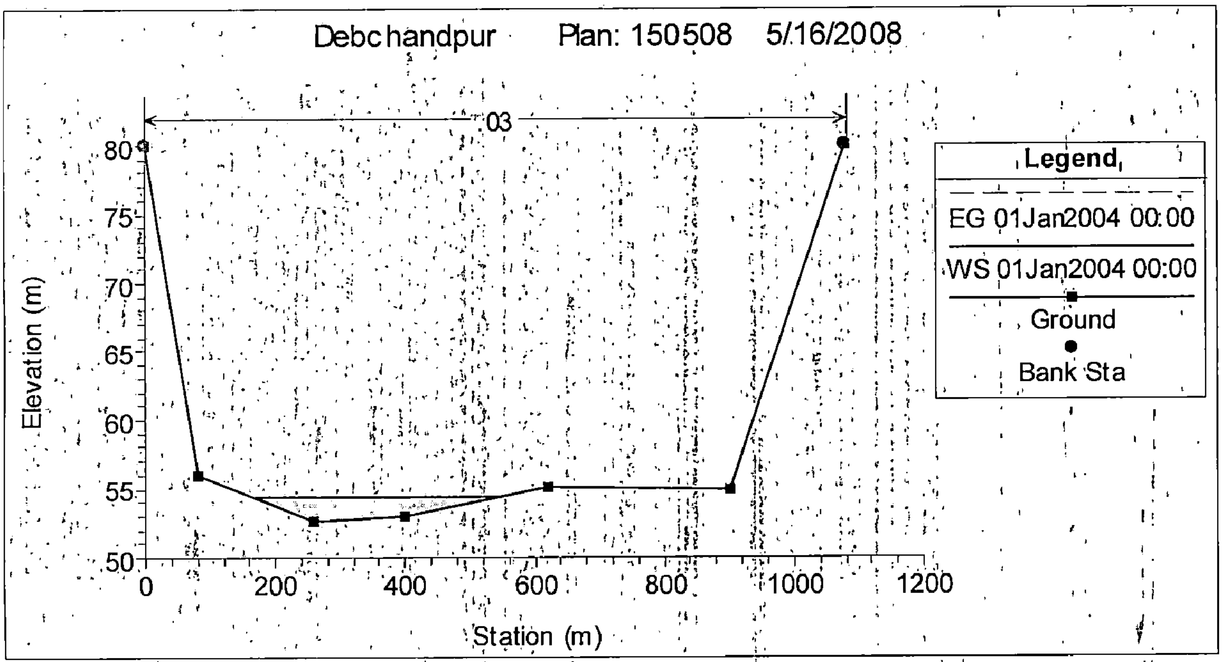


Fig. 4.19 Cross Section at 1248.5km

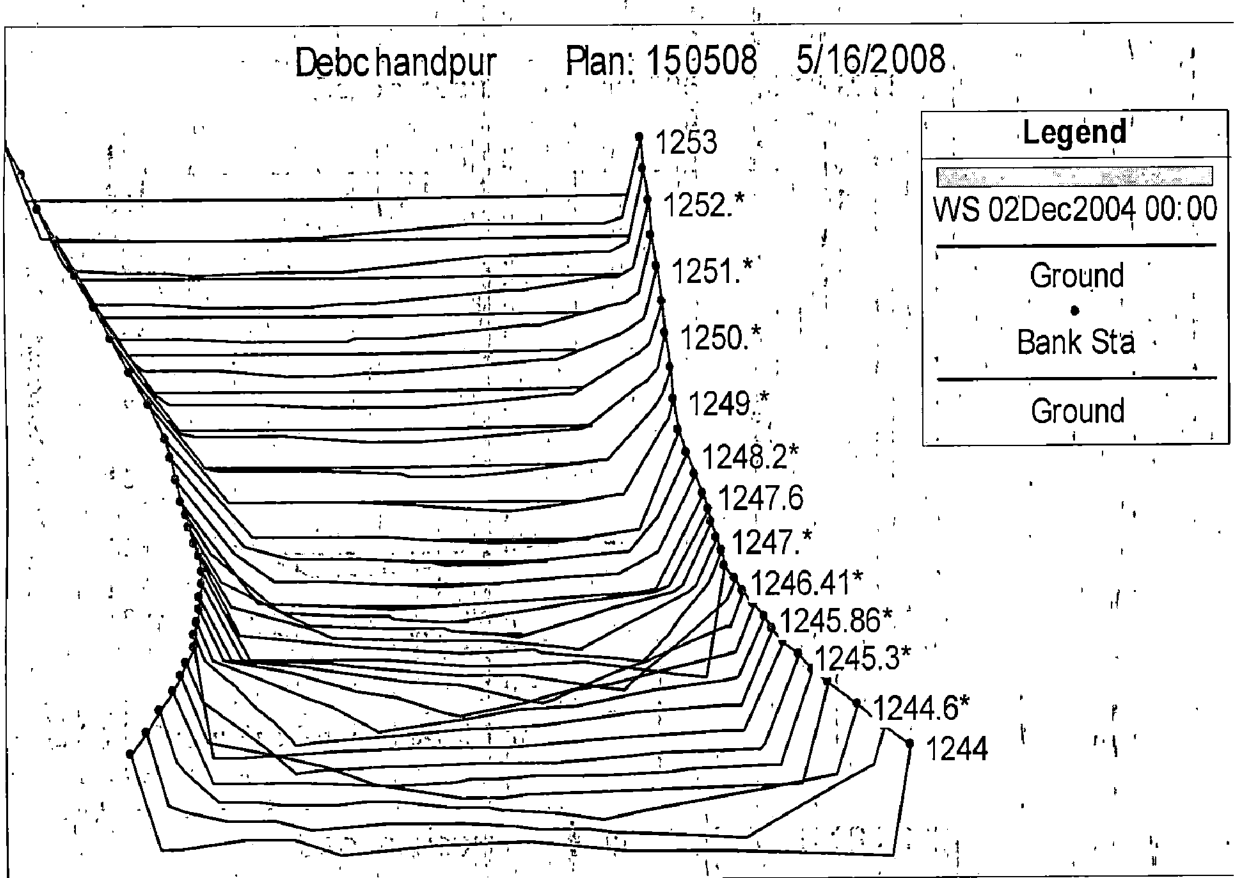


Fig. 4.20 Hec-Ras plot for Debchandpur showing water surface profile

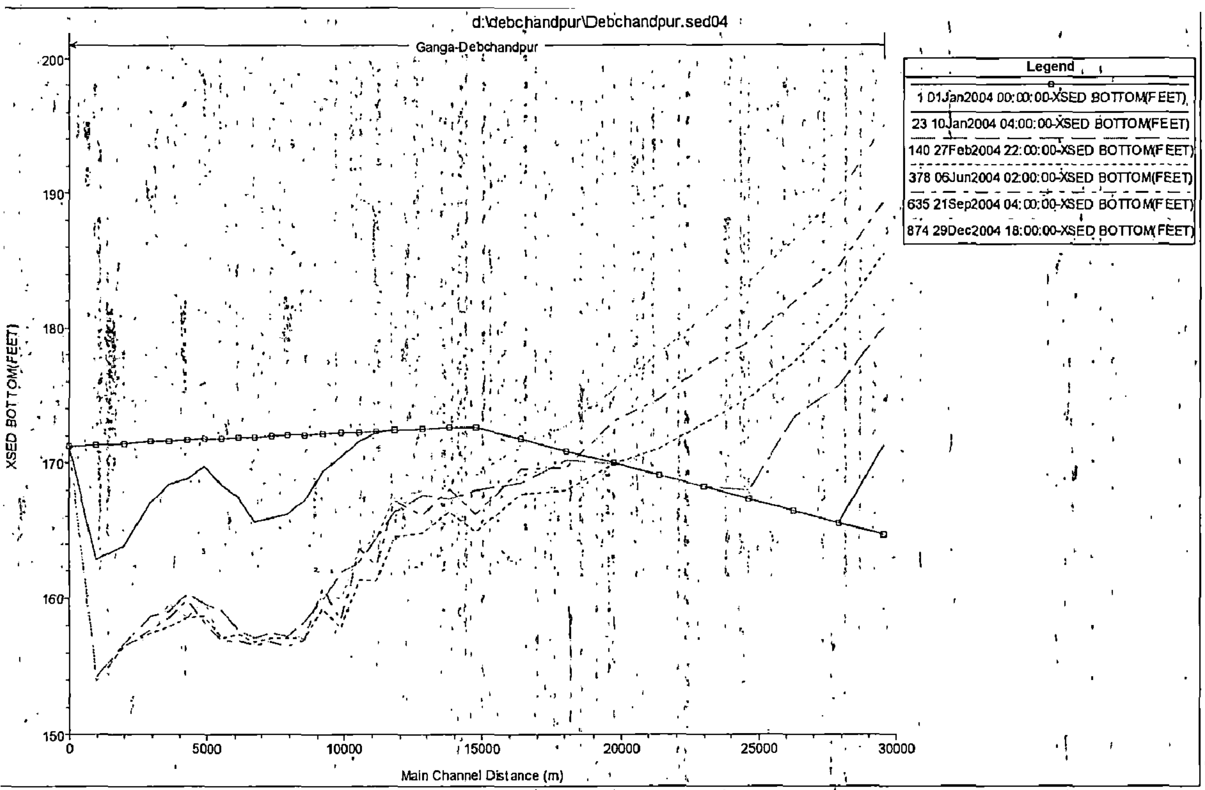


Fig. 4.21 Hec-Ras plot for Debchandpur showing Bed profile

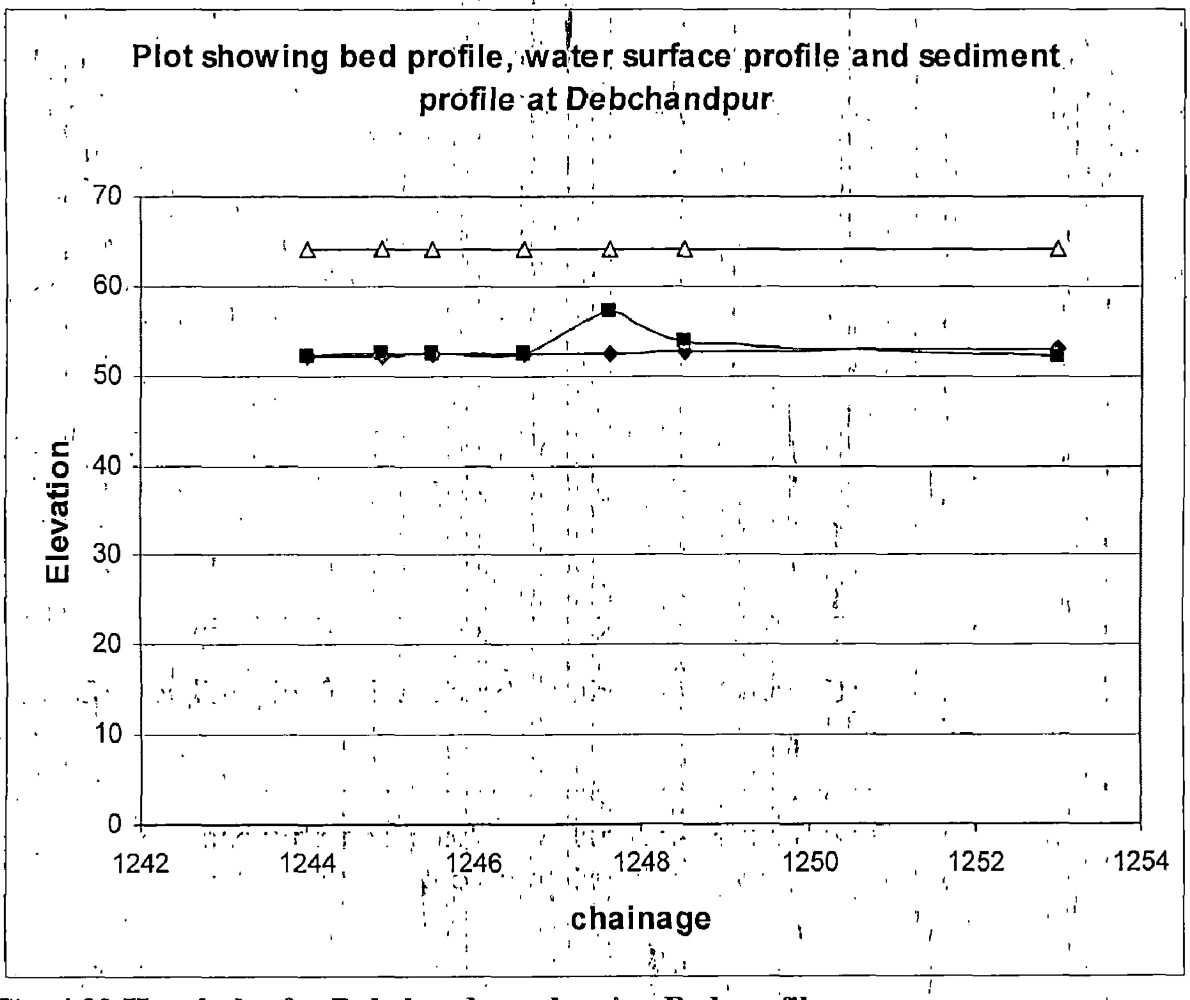


Fig. 4.22 Hec-6 plot for Debchandpur showing Bed profile

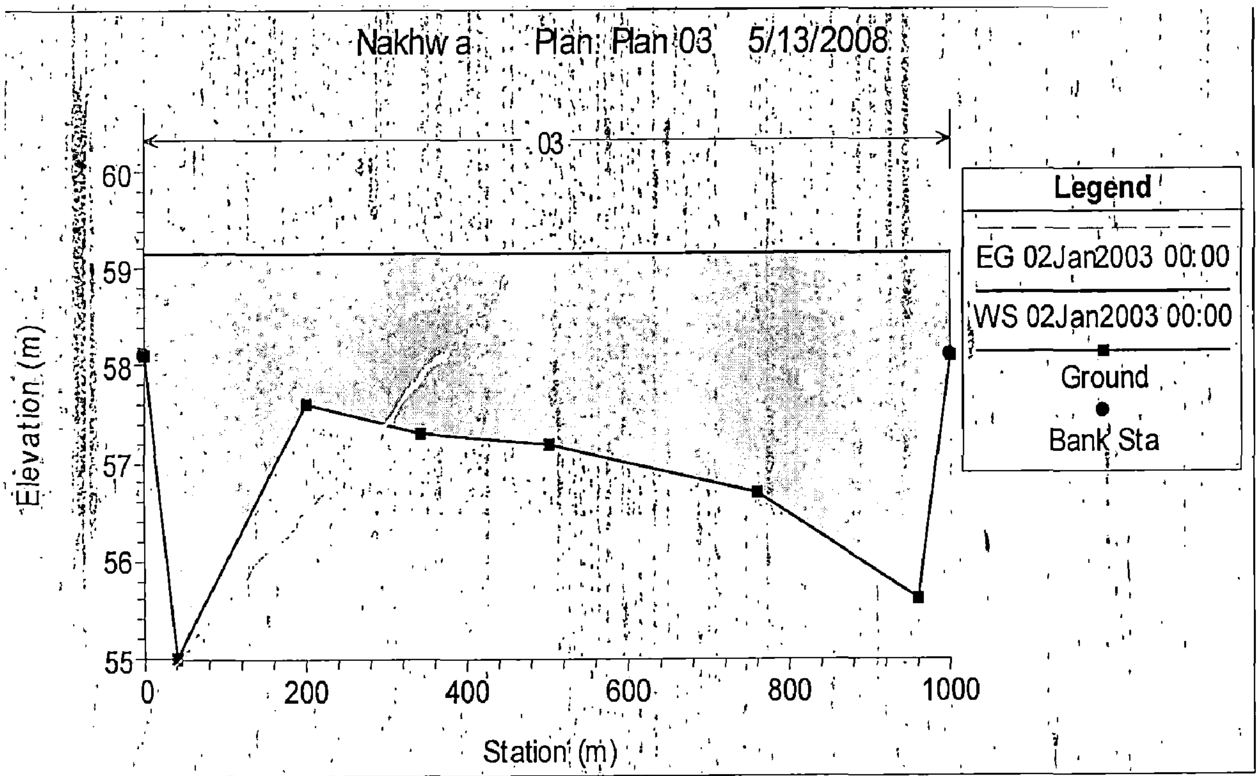


Fig. 4.23 Cross Section at 1298km

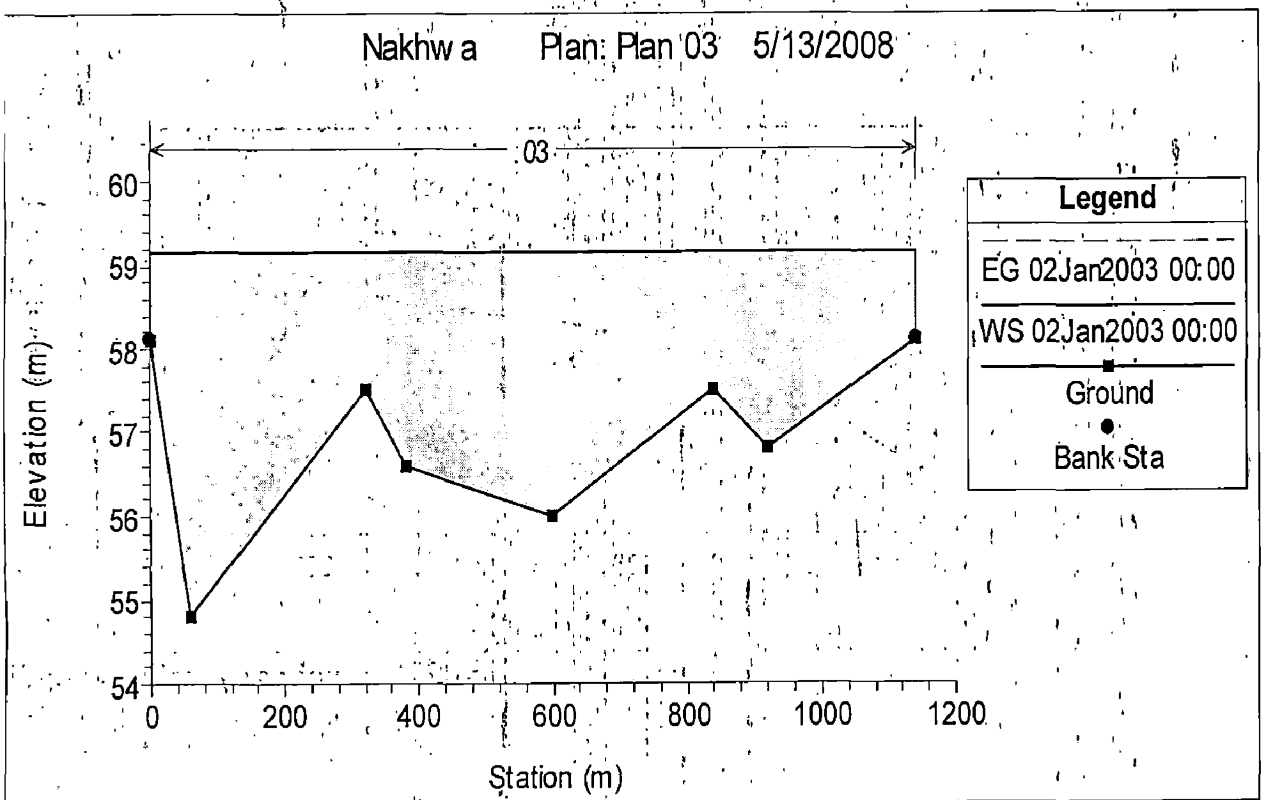


Fig. 4.24 Cross Section at 1296.75km

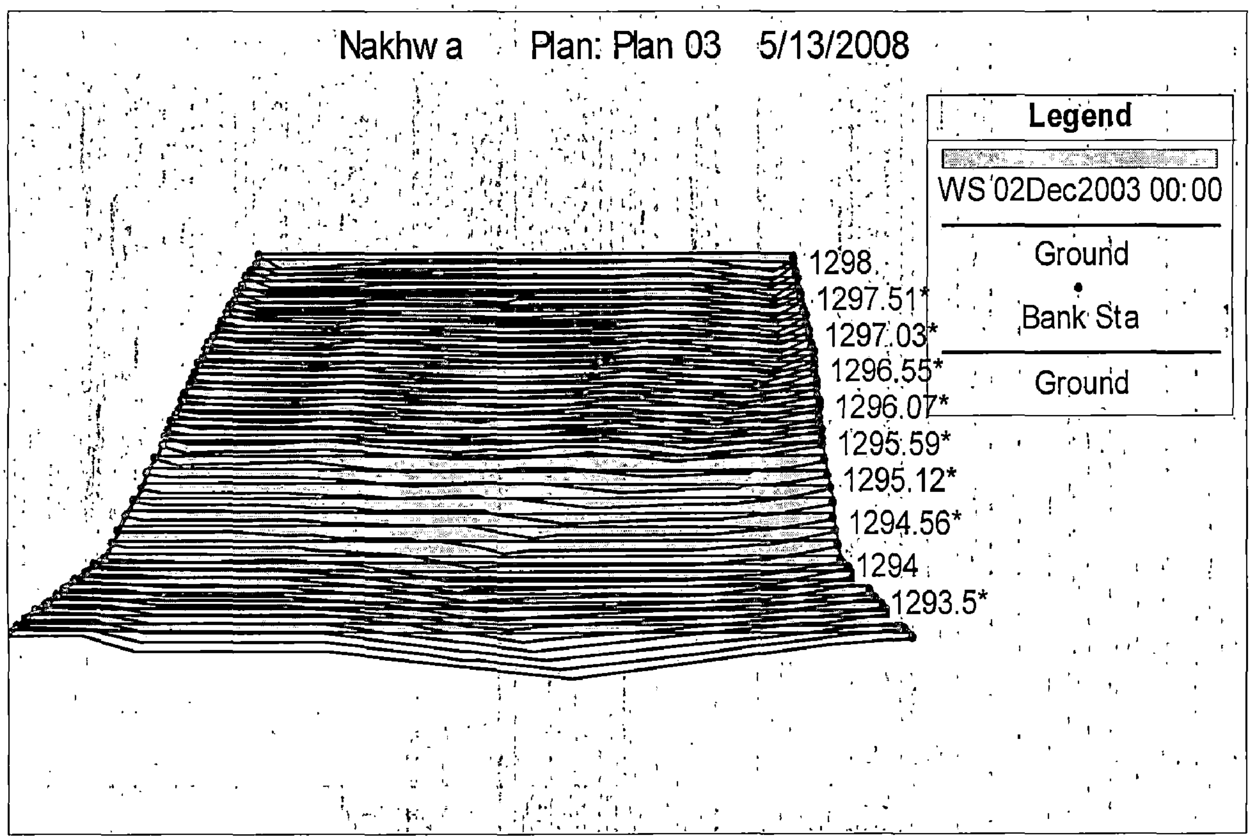


Fig. 4.25 Hec-Ras plot for Nakhwa showing water surface profile

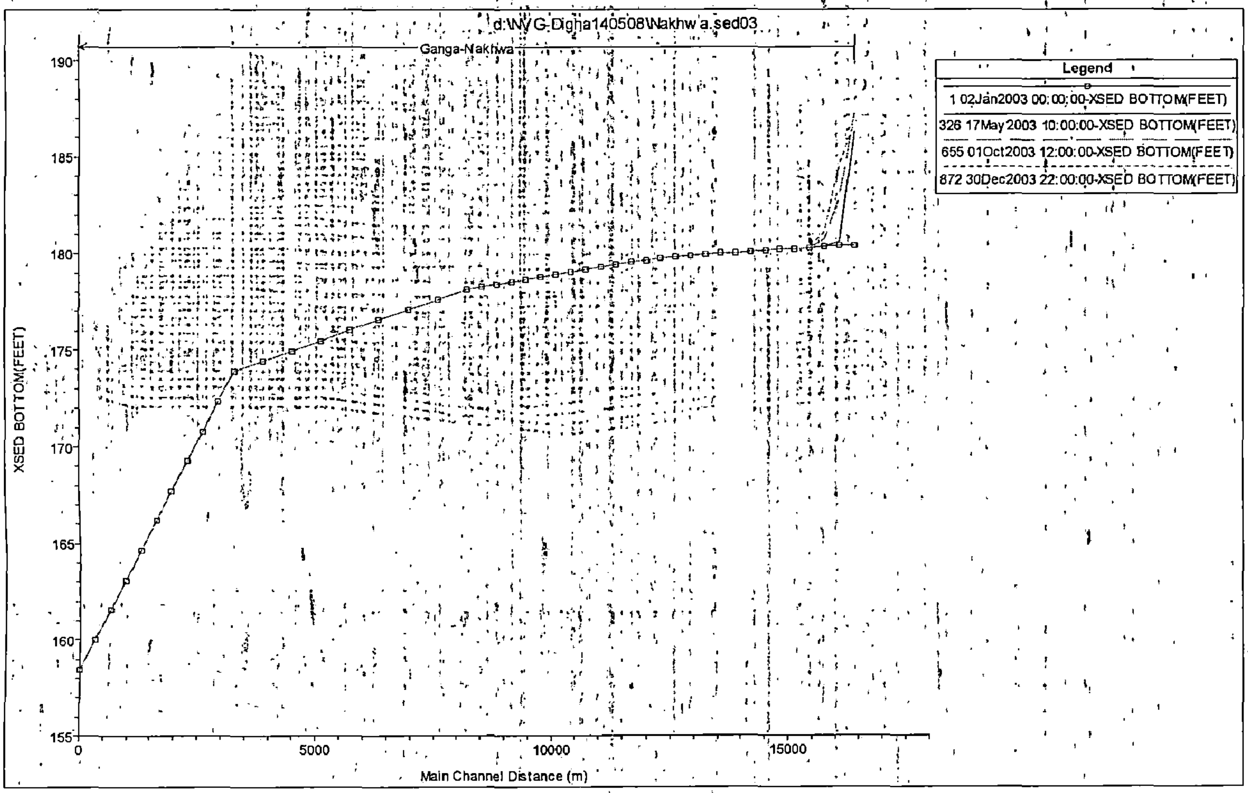


Fig. 4.26 Hec-Ras plot for Nakhwa showing Bed profile

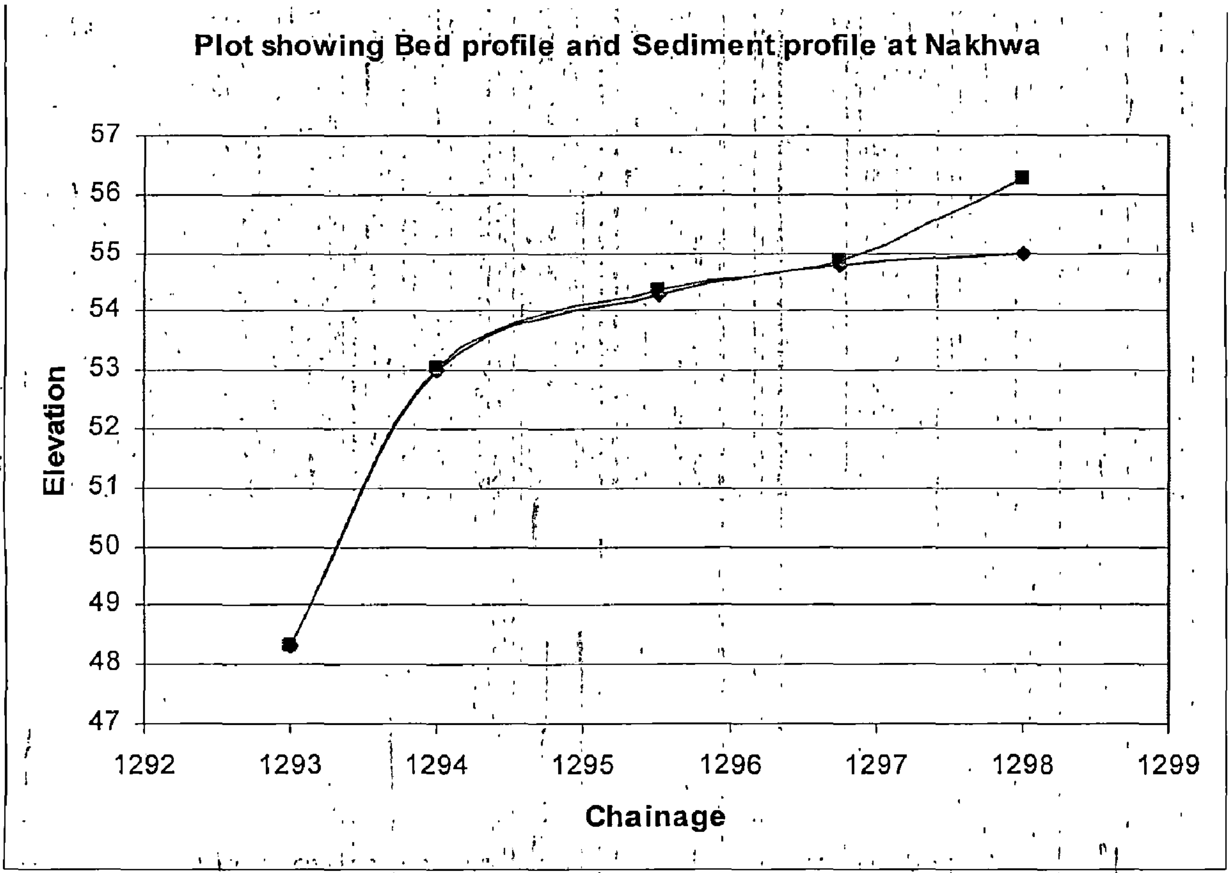


Fig. 4.27 Hec-6plot for Nakhwa showing Bed profile

4.6 Results and Discussion

With the available satellite hydrologic and hydrographic data it has been tried to assess the sediment carrying capacity of the Ganga River at few locations. But the data being collected has some inconsistency and discrepancies for which the results obtained were not up to expectations. Still it has been tried here to analyze with the readily available data with the software HEC-RAS and HEC6 and the results are plotted in the above pages.

5.1 General

In the predefined context, the following guiding principles have been adopted for developing: package of hydraulically efficient river training structures for navigation channel and trying them out in the Ganga river on pilot basis

(i) To contain stream-bank erosion in the study site.

(ii) To attempt sediment redistribution (to the extent possible) for canalization

(iii) To partially choke or reduce flow in the subsidiary channel for achieving desired flow concentration in the adopted navigation channel.

With the above guiding principles, the following hydraulically efficient river training structures have been identified for the implementation in the pilot study in the selected site of the Ganga river on experimental basis for evaluation of their suitable design ultimately.

1. Submerged vanes - for effecting desired sediment redistribution in the channel
2. Board Fencing - to offer resistance against stream-bank erosion
3. Porcupines - to induce siltation, retard scour development & breakup of flow vortex.
4. Kellner Jack Jetty - to assist in siltalion in subsidiary channel

The following combination of river training structures have been proposed for the field study at the six sites selected by IWAI and their performance will be evaluated after the flood seasons.

5.2 Combination of New river Training Package for Field Trials in the Ganga River

i) Geo Tubes made of Geo Bags will be used as submerged vanes for effecting the desired modification of flow to create the required secondary flow vortex to desirably redistribute sediment deposits in the channel.

ii) Board fencing protected bv porcupines along the stream-bank to he protected

iii) Permeable spurs of shorter length at 50 metro interval to create desired flow condition.

iv) Porcupine screens to assist in breakup of erosive vortex flow and to accentuate the sedimentation effect along the desired channel zones where submerged vanes are to be installed for the above purpose

v) Kellner Jack Jetty to assist in siltation along the desired channel alignment

5.3 Geo Tubes

The specification of all these methods are mentioned below with required dimension.

1. GeoTubes filled mechanically with sand can achieve maximum 50 to 60% height of its diameter. So, to achieve vanes of height 2m, we require GeoTubes of 3m dia & 7m length, whose budgetary cost will be about Rs. 20,000/- per no., and to achieve vanes of height 3m, we require a set of GeoTubes of 2m dia & 10m length over GeoTubes of 3m dia & 10m length, whose total budgetary cost will be about Rs. 50,000/- per set. Costs of its placement & mechanical sand filling will be extra. If its underwater construction, cost of hiring barge, rope net, heavy crane, etc. will be extra and very high.
2. If pump pressure is not properly controlled, there will be possibility of bursting of GeoTubes during sand filling.

   As such, it has been proposed a simple but a technically efficient, more durable, more reliable and more economical proposal for vane construction using flexible copper & polymer gabions (made of 8mm dia-3strands PP rope dia, 150x150mm mesh aperture & 1x1x1m size) filled with sand filled jumbo geobags (made of TFI 5200 woven fabric in 1x1x1m size). Cost of these gabions and jumbo geobags will be Rs. 1150/- per set (per cum), only. Cost of its placement and manual sand filling will be extra but much lower, compared to GeoTubes one. Even if it's underwater construction, these filled gabions can easily be lifted using lighter crane and placed at much lower cost.

For evaluating drag force and water current pressure to be experienced by the submerged vanes made of Geo Tubes, the following design details has been adopted.

i) Highest observed discharge has been adopted for assessing the stability of Geo Tubes (Submerged Vanes).

ii) Maximum scour depth has been computed by Lacey's formula for highest observed discharge.

iii) The waterway of the river has been adopted as measured from latest Satellite Images.

iv) Maximum velocity for highest observed flood has been calculated using Garde Rangaraju approach.

v) Water current pressure and drag force to be experienced by Geo Tubes has been calculated as per most prudent and latest methodologies available in the literature.

The horizontal resisting force of Geo Tubes to counter drag force / water current pressure has been calculated by considering the submerged weight of RBM filling material of Geo Tubes and adopting coefficient of friction as 0.6 and considering the incident velocity at the top of the Geo Tubes (taken from logarithmic velocity distribution).

Design of Submerged vanes

Bateshwarthan at ch 683km.

Period	Max WL	Min WL	LAD	Total Depth
Sept '03				
31.54				
31.46				
31.355				
31.27				
31.53				
31.86				
31.11				
32.29				
32.36				
32.4				
32.48				
32.68				
32.72				
32.76	32.87	31.11	5	6.76
32.83				
32.85				
32.86				
32.87				
32.83				
32.74				
32.62				
32.5				
32.39				
32.3				
32.24				
32.2				
32.19				
32.17				
32.12				
32.04				

Period	Max WL	Min WL	LAD	Total Depth
Oct '03				
31.99				
31.9				
31.83				
31.79				
31.72				
31.52				
31.28	31.99	26.27	4	9.72
31.96				
26.64				
26.69				
26.7				
26.69				
26.51				
26.27				
26.98				
26.74				

Period	Max WL	Min WL	LAD	Total Depth
June '04				
24.63				
24.63				
24.63				
24.63				
24.66				
24.83				
25.07				
25.78	27.53	24.63	2	4.9
26.67				
27.07				
27.17				
27.37				

Period	Max WL	Min WL	LAD	Total Depth
July '04				
27.09				
27				
26.91				
26.85				
26.85				
26.97				
27.06				
27.27				
27.59				
27.91				
28.21				
28.33				
28.88				
29.82				
29.06				
30.19	30.35	26.85	3.3	6.8
30.19				
30.12				
30.11				
30.11				
30.2				
30.32				
30.35				
30.35				
30.25				
30.08				

27.53	29.91
27.47	29.78
27.33	29.77
27.17	29.51

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
August'04					Sept '04				
29.39					31.28				
29.34					31.15				
29.41					30.91				
29.52					30.55				
29.62					30.31				
29.65					30.07				
29.65					29.89				
29.55					29.74				
29.38					29.5				
29.11					29.26				
29.12					29.1				
29.13					29.98				
29.25	31.46	29.06	3.9	6.3	28.97				
29.46					28.97				
29.81					28.86				
29.06					28.68	31.28	28.08	2.65	5.85
30.13					28.59				
30.18					28.7				
30.29					28.76				
30.41					28.68				
30.53					28.56				
30.74					28.44				
30.06					28.32				
31.35					28.2				
31.46					28.1				
31.39					28.18				
31.15					28.6				
30.97					28.08				
30.89					29.3				
30.94					29.19				
31.26									

Period	Max WL	Min WL	LAD	Total Depth
Oct '04				
29.95				
28.71				
28.58				
28.54				
28.43				
28.37				
28.36	29.95	27.91	2.95	4.99
28.69				
28.86				

28.69
28.52
28.38
28.17
28.03
27.91

Average depth =	6.47	m
Height of vane =	1.94	m
Let us provide	2	m
Length of vane =	6	m
Vane to vane distance =	7	m
Bank to vane distance =	8	m
No.of vanes in an array =	65	nos
Distance between two arrays =	60	m
No.of arrays =	27	nos

Punarak at ch 879km.

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Mar '01					**Apr ' 01**				
33.025					32.865				
32.995					32.855				
32.985					32.825				
32.975					32.805				
32.955					32.795				
32.925					32.785				
32.915					32.805				
32.915					32.815				
32.915					32.835				
32.915					32.855				
32.915					32.855				
32.895					32.855				
32.89	33.025	32.715	2.05	2.36	32.835	32.865	32.785	2	2.08
32.915					32.85				
32.92					32.855				
32.905					32.855				
32.88					32.825				
32.865					32.81				
32.845					32.785				
32.805					32.85				
32.775					32.855				
32.77					32.84				
32.77					32.83				
32.76					32.815				
32.755					32.795				
32.745					32.795				
32.725					32.815				
32.72					32.825				
32.715					32.835				
32.745					32.845				
32.865									

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
May' 01					**Mar ' 02**				
32.875					33.445				
32.895					33.385				
32.975					33.365				
33.005					33.345				
33.055					33.315				
33.115					33.255				
33.185					33.185				
33.195					33.095				
33.265					33.005				
33.335					32.945				
33.335					32.885				
33.305					32.825				
33.305					32.795				
33.305	33.565	32.875	2.25	2.94	32.79	33.445	32.625	2.1	2.92
33.365					32.825				
33.295					32.825				
33.245					32.835				
33.215					32.815				
33.235					32.835				
33.255					32.835				
33.325					32.855				
33.335					32.815				
33.365					32.785				
33.37					32.755				
33.37					32.735				
33.395					32.705				
33.335					32.675				
33.315					32.665				
33.305					32.64				
33.385					32.625				
33.565									

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Apr ' 02					**May ' 02**				
32.615					33.065				
32.645					33.095				
32.655					33.135				
32.675					33.13				
32.755					33.115				
32.755					33.115				
32.785					33.105				
32.825					33.11				
32.865					33.115				
32.885					33.105				
32.885					33.075				
32.875					33.07				
32.865					33.065				
32.88					33.075				

32.945	33.215	32.615	2.1	2.7	33.085	34.375	33.065	2.15	3.46
33.025					33.275				
33.135					33.365				
33.205					33.465				
33.215					33.545				
33.185					33.605				
33.185					33.565				
33.185					34.295				
33.185					34.375				
33.145					34.215				
33.105					34.155				
33.065					34.135				
33.015					34.105				
33.01					34.065				
33.01					33.995				
33.045					33.935				
					33.855				

Period	**Max WL**	**Min WL**	**LAD**	**Total Depth**	**Period**	**Max WL**	**Min WL**	**LAD**	**Total Depth**
Mar ' 03					**Apr ' 03**				
33.425					32.945				
33.465					32.965				
33.5					32.935				
33.505					32.935				
33.435					32.935				
33.365					32.915				
33.315					32.885				
33.205					32.905				
33.145					32.925				
33.095					32.925				
33.045					32.925				
33.015					32.915				
32.975					32.905				
32.945					32.905				
32.915	33.505	32.685	2.05	2.87	32.895	33.365	32.845	2.05	2.57
32.895					32.885				
32.875					32.915				
32.845					32.875				
32.805					32.845				
32.755					32.895				
32.715					32.94				
32.715					33.035				
32.685					33.135				
32.745					33.175				
32.835					33.195				
32.865					33.225				
32.865					33.265				
32.745					33.26				
32.815					33.295				
32.785					33.365				
32.815									

Period	Max WL	Min WL	LAD	Total Depth
May ' 03				
33.415				
33.495				
33.505				
33.505				
33.495				
33.465				
33.435				
33.375				
33.415				
33.405				
33.445				
33.515				
33.495				
33.465				
33.445				
33.425	33.515	33.295	2.1	2.32
33.405				
33.385				
33.375				
33.355				
33.335				
33.315				
33.305				
33.295				
33.335				
33.395				
33.435				
33.415				
33.395				
33.365				
33.395				

Period	Max WL	Min WL	LAD	Total Depth
Mar ' 04				
33.26				
33.245				
33.225				
33.215				
33.2				
33.195				
33.205				
33.215				
33.195				
33.175				
33.185				
33.195				
33.165				
33.145				
33.125				
33.115	33.26	33.1	2	2.16
33.115				
33.125				
33.115				
33.115				
33.115				
33.126				
33.135				
33.12				
33.11				
33.11				
33.11				
33.105				
33.1				
33.105				
33.105				

Period	Max WL	Min WL	LAD	Total Depth
Apr ' 04				
33.145				
33.155				
33.135				
33.135				
33.108				
33.075				
33.115				
33.165				
33.175				
33.19				
33.215				
33.3				
33.335	33.345	33.075	2	2.27
33.345				
33.315				

Period	Max WL	Min WL	LAD	Total Depth
May ' 04				
33.285				
33.275				
33.295				
33.315				
33.365				
33.385				
33.345				
33.295				
33.255				
33.235				
33.255				
33.235				
33.215	33.715	33.045	2.05	2.72
33.21				
33.235				

33.335	33.225
33.315	33.205
33.305	33.165
33.295	33.115
33.305	33.085
33.325	33.065
33.315	33.045
33.295	33.075
33.315	33.4
33.315	33.715
33.315	33.675
33.315	33.565
33.305	33.505
33.315	33.555
33.305	33.585
	33.63

Average depth =	2.62	m
Height of vane =	0.78	m
Let us provide	2	m
Length of vane =	6	m
Vane to vane distance =	7	m
Bank to vane distance =	8	m
No.of vanes in an array =	55	nos
Distance between two arrays =	60	m
No.of arrays =	72	nos

Digha at ch976km.

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Mar ' 01					**Apr ' 01**				
40.769					40.719				
40.749					40.619				
40.759					40.589				
40.759					40.589				
40.779					40.659				
40.759					40.639				
40.739					40.679				
40.749					40.669				
40.739					40.669				
40.749					40.699				
40.749					40.699				
40.759	40.779	40.569	1.75	1.96	40.699	40.889	40.589	1.95	2.25
40.759					40.739				
40.759					40.739				
40.769					40.749				
40.729					40.769				
40.729					40.769				
40.729					40.769				

40.689					40.849				
40.629					40.799				
40.619					40.769				
40.619					40.789				
40.609					40.789				
40.569					40.799				
40.569					40.829				
40.569					40.844				
40.569					40.849				
40.569					40.849				
40.679					40.889				
40.759					40.889				
40.769									

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
May ' 01					**Mar '02**				
40.949					41.499				
41.049					41.479				
41.069					41.469				
41.149					41.449				
41.189					41.399				
41.189					41.309				
41.189					41.219				
41.289					41.169				
41.309					41.119				
41.269					41.079				
41.219					41.049				
41.249					41.019				
41.269					41.039				
41.249	41.669	40.949	1.97	2.69	41.069	41.499	40.879	2.1	2.72
41.329					41.089				
41.229					41.069				
41.209					41.059				
41.269					41.069				
41.349					41.099				
41.319					41.099				
41.359					41.059				
41.379					41.029				
41.359					40.999				
41.359					40.979				
41.359					40.969				
41.329					40.969				
41.369					40.939				
41.329					40.939				
41.449					40.919				
41.649					40.879				
41.669					40.889				

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
April '02					**May '02**				
40.919					41.379				

40.929					41.419				
40.949					41.379				
41.049					41.369				
41.019					41.339				
41.029					41.379				
41.109					41.399				
41.149					41.459				
41.169					41.469				
41.149					41.399				
41.139					41.359				
41.129					41.409				
41.159					41.429				
41.229					41.439				
41.324	41.449	40.919	2.10	2.63	41.629	43.169	41.339	2.25	4.08
41.419					41.709				
41.449					41.769				
41.449					41.889				
41.409					41.819				
41.399					41.939				
41.419					42.619				
41.429					42.549				
41.369					42.319				
41.329					42.289				
41.289					42.289				
41.229					42.279				
41.279					42.229				
41.269					43.169				
41.289					43.119				
41.339					43.069				
					41.979				

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Mar ' 03					**Apr ' 03**				
41.689					41.259				
41.729					41.289				
41.749					41.259				
41.739					41.239				
41.569					41.239				
41.499					41.289				
41.409					41.289				
41.359					41.369				
41.319					41.279				
41.299					41.189				
41.249					41.189				
41.219					41.219				
41.199					41.249				
41.169					41.269				
41.149	41.749	40.989	2.15	2.91	41.299	41.769	41.189	1.70	2.28
41.109					41.269				
41.069					41.259				
41.059					41.249				
41.029					41.269				
40.99					41.349				
40.999					41.459				
40.989					41.529				
41.119					41.569				

41.119					41.579				
41.129					41.609				
41.129					41.629				
41.139					41.639				
41.139					41.669				
41.129					41.699				
41.159					41.769				
41.259									

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
May ' 03					**Mar ' 04**				
41.839					41.429				
41.849					41.429				
41.819					41.419				
41.819					41.399				
41.819					41.399				
41.819					41.399				
41.789					41.399				
41.829					41.389				
41.849					41.379				
41.859					41.379				
41.849					41.379				
41.789					41.369				
41.779					41.329				
41.779					41.329				
41.779	41.859	41.609	2.10	2.35	41.319	41.429	41.299	2.05	2.18
41.729					41.319				
41.689					41.319				
41.669					41.319				
41.639					41.319				
41.629					41.309				
41.609					41.329				
41.609					41.349				
41.609					41.329				
41.619					41.329				
41.659					41.329				
41.759					41.299				
41.739					41.299				
41.739					41.319				
41.739					41.319				
41.739					41.319				
41.739					41.359				

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Apr ' 04					**May ' 04**				
41.369					41.449				
41.369					41.469				
41.349					41.489				
41.339					41.489				
41.329					41.439				
41.369					41.419				
41.439					41.439				

41.429					41.429				
41.429					41.419				
41.459					41.429				
41.459					41.389				
41.509					41.389				
41.489					41.409				
41.469					41.429				
41.489	41.509	41.329	2.05	2.23	41.379	41.819	41.239	2.00	2.58
41.459					41.369				
41.449					41.369				
41.449					41.309				
41.479					41.269				
41.489					41.269				
41.459					41.239				
41.429					41.369				
41.379					41.669				
41.449					41.819				
41.439					41.719				
41.449					41.619				
41.429					41.609				
41.409					41.709				
41.439					41.759				
41.429					41.799				
					41.809				

Average depth =	2.57	m
Height of vane =	0.77	m
Let us provide	2	m
Length of vane =	6	m
Vane to vane distance =	7	m
Bank to vane distance =	8	m
No.of vanes in an array =	59	nos
Distance between two arrays =	60	m
No.of arrays =	21	nos

Arjunpur at ch. 1118km.

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Mar ' 01					**Apr ' 01**				
50.376					50.166				
50.371					50.156				
50.366					50.156				
50.366					50.156				
50.366					50.156				
50.341					50.146				
50.316					50.136				
50.296					50.126				
50.266					50.116				
50.246					50.116				
50.236					50.111				
50.226					50.106				
50.226					50.096				

50.216					50.086				
50.216	50.376	50.106	1.45	1.72	50.076	50.166	50.046	1.40	1.52
50.196					50.066				
50.166					50.056				
50.146					50.046				
50.126					50.046				
50.116					50.046				
50.116					50.046				
50.106					50.051				
50.126					50.056				
50.136					50.066				
50.136					50.066				
50.146					50.071				
50.146					50.071				
50.151					50.066				
50.176					50.071				
50.176					50.076				
50.176									

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
May ' 01					**Mar ' 02**				
50.076					50.681				
50.076					50.656				
50.071					50.606				
50.066					50.556				
50.066					50.506				
50.056					50.466				
50.046					50.426				
50.026					50.376				
50.016					50.326				
49.996					50.306				
49.986					50.296				
49.976					50.286				
49.966					50.266				
49.956	50.076	49.891	1.30	1.49	50.236	50.681	50.026	1.45	2.10
49.956					50.206				
49.956					50.176				
49.956					50.166				
49.956					50.156				
49.956					50.136				
49.951					50.166				
49.946					50.096				
49.931					50.086				
49.926					50.096				
49.926					50.086				
49.926					50.066				
49.936					50.056				
49.976					50.036				
49.916					50.026				
49.906					50.026				
49.896					50.026				
49.891					50.041				

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Apr ' 02					**May ' 02**				
50.056					49.766				
50.056					49.776				
50.046					49.776				
50.046					49.771				
50.026					49.766				
50.006					49.756				
49.991					49.746				
49.976					49.736				
49.956					49.726				
49.946					49.706				
49.946					49.706				
49.946					49.706				
49.936					49.706				
49.906					49.706				
49.876	50.056	49.666	1.50	1.89	49.696	49.776	49.626	1.50	1.65
49.846					49.696				
49.826					49.686				
49.806					49.676				
49.776					49.666				
49.756					49.659				
49.746					49.716				
49.756					49.716				
49.746					49.636				
49.716					49.636				
49.706					49.636				
49.696					49.626				
49.684					49.626				
49.676					49.634				
49.666					49.656				
49.716					49.676				
					49.686				

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Mar ' 03					**Apr ' 03**				
50.256					49.576				
50.236					49.546				
50.216					49.526				
50.196					49.486				
50.166					49.446				
50.146					49.446				
50.176					49.436				
50.226					49.426				
50.216					49.426				
50.196					49.426				
50.146					49.446				
50.096					49.456				
50.056					49.476				
50.016					49.466				
49.966	50.256	49.596	1.55	2.21	49.457	49.576	49.296	1.30	1.58

49.896					49.446				
49.836					49.426				
49.816					49.416				
49.796					49.406				
49.776					49.386				
49.751					49.366				
49.731					49.346				
49.731					49.326				
49.751					49.316				
49.726					49.316				
49.686					49.316				
49.656					49.296				
49.636					49.296				
49.616					49.296				
49.606					49.296				
49.596									

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
May ' 03					**Mar ' 04**				
49.301					50.676				
49.301					50.666				
49.316					50.656				
49.306					50.646				
49.286					50.626				
49.271					50.616				
49.291					50.596				
49.346					50.586				
49.416					50.576				
49.421					50.556				
49.421					50.536				
49.421					50.546				
49.416					50.556				
49.416					50.536				
49.416					50.516				
49.416	49.421	49.056	1.25	1.62	50.506	50.676	50.286	1.55	1.94
49.396					50.496				
49.366					50.497				
49.336					50.486				
49.216					50.466				
49.276					50.446				
49.246					50.436				
49.216					50.421				
49.186					50.416				
49.156					50.406				
49.126					50.386				
49.106					50.366				
49.076					50.356				
49.056					50.346				
49.056					50.316				
49.056					50.286				

Period	Max	Min	LAD	Total	Period	Max	Min	LAD	Total

	WL	WL		Depth		WL	WL		Depth
Apr ' 04					**May ' 04**				
50.246					49.926				
50.226					49.916				
50.216					49.906				
50.206					49.906				
50.196					49.906				
50.186					49.926				
50.176					49.926				
50.166					49.926				
50.156					49.921				
50.156					49.916				
50.146					49.911				
50.136					49.901				
50.121					49.881				
50.106					49.856				
50.096					49.846				
50.066	50.246	49.946	1.60	1.90	49.841	49.926	49.731	1.60	1.80
50.046					49.836				
50.026					49.836				
50.006					49.826				
50.001					49.806				
49.996					49.786				
49.991					49.786				
49.991					49.776				
49.986					49.776				
49.986					49.771				
50.016					49.771				
50.016					49.766				
49.986					49.761				
49.966					49.746				
49.946					49.736				
					49.731				

Average depth =	1.78	m
Height of vane =	0.54	m
Let us provide	2	m
Length of vane =	6	m
Vane to vane distance =	7	m
Bank to vane distance =	8	m
No.of vanes in an array =	59	nos
Distance between two arrays =	60	m
No.of arrays =	21	nos

Debchandpur at ch1247km.

Period	Max	Min	LAD	Total	Period	Max	Min	LAD	Total
	WL	WL		Depth		WL	WL		Depth
Mar ' 04					**Apr '04**				
52.68					52.175				
52.65					52.17				
52.615					52.17				
52.59					52.165				
52.55					52.165				

52.52					52.16				
52.5					52.16				
52.47					52.14				
52.45					52.13				
52.42					52.12				
52.39					52.12				
52.38					52.12				
52.37					52.11				
52.36					52.11				
52.35	52.68	52.175	1.60	2.11	52.11	52.175	52.09	1.25	1.33
52.34					52.105				
52.33					52.105				
52.31					52.1				
52.285					52.1				
52.28					52.1				
52.27					52.1				
52.26					52.095				
52.25					52.09				
52.25					52.09				
52.24					52.09				
52.235					52.1				
52.215					52.11				
52.2					52.105				
52.19					52.1				
52.175					52.1				
52.175									

Period	**Max WL**	**Min WL**	**LAD**	**Total Depth**	**Period**	**Max WL**	**Min WL**	**LAD**	**Total Depth**
May ' 04					**Mar ' 05**				
52.1					52.41				
52.095					52.41				
52.095					52.39				
52.09					52.35				
52.09					52.31				
52.08					52.31				
52.08					52.3				
52.07					52.3				
52.065					52.29				
52.05					52.28				
52.045					52.28				
52.04					52.31				
52.04					52.33				
52.03	52.1	52	1.15	1.25	52.35	52.48	52.28	1.30	1.50
52.03					52.36				
52.03					52.36				
52.025					52.39				
52.02					52.4				
52.02					52.39				
52.02					52.39				
52.02					52.4				
52.02					52.42				
52.015					52.44				
52.015					52.46				

52.01					52.48				
52.01					52.48				
52.01					52.475				
52.005					52.46				
52.005					52.46				
52					52.46				
52					52.44				

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Apr '05					**May ' 05**				
52.42					52.11				
52.42					52.115				
52.41					52.12				
52.41					52.12				
52.39					52.12				
52.38					52.115				
52.36					52.1				
52.35					52.09				
52.34					52.08				
52.32					52.08				
52.31					52.07				
52.3					52.07				
52.3					52.06				
52.29					52.04				
52.29	52.42	52.11	1.20	1.51	52.02	52.12	51.98	1.10	1.24
52.26					52.01				
52.23					52				
52.2					51.995				
52.16					51.99				
52.14					51.99				
52.14					51.98				
52.13					51.98				
52.13					52.02				
52.12					52.06				
52.12					52.09				
52.115					52.09				
52.115					52.085				
52.11					52.08				
52.11					52.075				
52.11					52.07				
					52.07				

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Mar '06					**Apr ' 06**				
52.32					52.21				
52.33					52.21				
52.34					52.16				
52.28					52.14				
52.22					52.14				
52.19					52.14				
52.18					52.15				

52.15					52.15				
52.14					52.12				
52.14					52.12				
52.14					52.11				
52.17					52.1				
52.215					52.1				
52.24					52.09				
52.25					52.06				
52.28					52.05				
52.3	52.34	52.14	1.20	1.40	52.04	52.21	52.04	1.20	1.37
52.31					52.06				
52.32					52.08				
52.325					52.1				
52.32					52.105				
52.31					52.1				
52.285					52.08				
52.285					52.08				
52.26					52.08				
52.23					52.08				
52.21					52.09				
52.21					52.09				
52.21					52.09				
52.24					52.12				
52.24									

Average depth =	1.46	m
Height of vane =	0.44	m
Let us provide	2	m
Length of vane =	6	m
Vane to vane distance =	7	m
Bank to vane distance =	8	m
No.of vanes in an array =	50	nos
Distance between two arrays =	60	m
No.of arrays =	22	nos

Nakhwa at ch1299km.

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
Mar '04					**April '04**				
59.57					59.31				
59.55					59.3				
59.55					59.3				
59.52					59.29				
59.51					59.29				
59.49					59.29				
59.49					59.29				
59.48					59.29				
59.48					59.29				
59.46					59.29				
59.44					59.29				
59.44					59.27				
59.44					59.25				
59.42					59.24				
59.42	59.57	59.32	1.60	1.85	59.22	59.31	59.09	1.25	1.47
59.42					59.2				

59.41					59.18				
59.4					59.17				
59.39					59.17				
59.38					59.17				
59.38					59.17				
59.37					59.17				
59.36					59.17				
59.35					59.16				
59.34					59.15				
59.33					59.15				
59.32					59.14				
59.32					59.14				
59.35					59.12				
59.37					59.09				
59.32									

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
May '04					**Mar '05**				
59.1					59.16				
59.1					59.15				
59.09					59.14				
59.08					59.14				
59.07					59.15				
59.06					59.15				
59.05					59.15				
59.04					59.13				
59.03					59.12				
59.03					59.11				
59.03					59.16				
59.03					59.17				
59.03					59.16				
59.03	59.1	58.89	1.15	1.36	59.16	59.28	59.11	1.30	1.47
59.03					59.16				
59.03					59.17				
59.03					59.18				
59.03					59.205				
59.03					59.23				
59.01					59.26				
59					59.27				
58.98					59.28				
58.97					59.28				
58.96					59.28				
58.95					59.25				
58.94					59.22				
58.91					59.2				
58.9					59.18				
58.9					59.16				
58.9					59.15				
58.89					59.14				

Period	Max WL	Min WL	LAD	Total Depth	Period	Max WL	Min WL	LAD	Total Depth
April '05					**May '05**				

59.12					58.84				
59.12					58.84				
59.12					58.82				
59.1					58.82				
59.1					58.8				
59.1					58.78				
59.09					58.77				
59.08					58.77				
59.07					58.76				
59.05					58.76				
59.04					58.74				
59.01					58.73				
58.99					58.72				
58.98					58.7				
58.98					58.69				
58.98	59.12	58.84	1.20	1.48	58.66	58.84	58.58	1.10	1.36
58.98					58.65				
58.97					58.64				
58.96					58.64				
58.95					58.63				
58.92					58.63				
58.92					58.63				
58.92					58.63				
58.9					58.63				
58.89					58.62				
58.87					58.61				
58.86					58.6				
58.85					58.59				
58.85					58.58				
58.84					58.58				
					58.58				

Period	**Max WL**	**Min WL**	**LAD**	**Total Depth**	**Period**	**Max WL**	**Min WL**	**LAD**	**Total Depth**
Mar '06					**April'06**				
58.8					58.65				
58.9					58.64				
58.9					58.63				
58.9					58.63				
58.77					58.63				
58.71					58.63				
58.71					58.61				
58.69					58.6				
58.68					58.6				
58.67					58.59				
58.73					58.58				
58.76					58.57				
58.78					58.56				
58.79	58.9	58.66	1.20	1.44	58.56	58.65	58.49	1.25	1.41
58.82					58.55				
58.85					58.54				
58.85					58.53				
58.82					58.53				

58.8	58.57
58.8	58.57
58.78	58.54
58.77	58.52
58.74	58.5
58.73	58.5
58.71	58.5
58.7	58.5
58.69	58.5
58.68	58.49
58.68	58.49
58.68	58.53
58.66	

Average depth =	1.48	m
Height of vane =	0.44	m
Let us provide	2	m
Length of vane =	6	m
Vane to vane distance =	7	m
Bank to vane distance =	8	m
No.of vanes in an array =	86	nos
Distance between two arrays =	60	m
No.of arrays =	25	nos

Stability of Geotubes at Bateswarsthan

Max discharge is in 2003(as per data) =	70000	cumecs
Normal scour depth as per Lacey =	$0.473(Q/f)^{1/3}$	
silt factor,f =	1	
Normal scour depth,d =	19.49	m
local scour depth =	1.5*19.49	
	29	m
Depth of scour below river bed =	9.51	m
Waterway width(from 2007, satellite image) =	1500	m
Water depth(from scour depth consideration) =	19	m
Area of waterway =	1500*19.49	
=	29235	sq.m
Perimeter, P =	1500+ 2*19.49	
=	1539.00	m
Hydraulic mean redius, R =	29235/153!	m
=	19	m
Channel bed slope,s =	1 / 7000	
	0.00014	
Length of Geotube =	7	m
Width of Geotube =	1.6	m
Height of Geotube =	1.6	m
sediment on bed,d_{50} =	0.15	mm

Garde Rangaraju approach

Now R/d =	19 / 0.15	
=	126667	
S / ($\Delta\rho s$ / ρ) =	0.00014	

The flow regime falls under Antidune regime

Water Current Pressure Aprroach

From fig(5.4, Garde Rangaraju), corresponding to d_{50} =	0.15	mm
k_1 =	1.3	
k_2 =	1.1	
$k_2\,(R/d)^{1/3} * (S/ (\Delta\gamma s/\gamma f))$ =	0.00773	
c_1 =	0.3	
velocity =	c1/k1*$\sqrt{(\Delta\gamma s/\rho f)}$	
	3.180	m/sec
Water Current Pressure, P =	KV^2	
K =	78.8	
P =	78.8 * 3.18^2	
	796.86	kg/m²
Water current force =	7*1.6*796.86	
	8925	kg

Drag Force Aprroach

The average shear stress on the wetted surface, $\tau_0 = C_d \rho V^2/2$

$= f\rho V^2/8$

Colebrook - White formula for estimating the friction factor

$$1/\sqrt{f} = -2.0\log_{10}((K_s/3.71D_H)+(2.51/R_e\sqrt{f}))$$

$$Re=\rho VD_H/\mu$$

Density of water, ρ =	1000	kg/m^3
Dynamic viscousity, μ =	0.001	kg/m/s
Velocity of water, V=	3.180	m/sec
D_H =	76	m
Reynolds number, Re =	477	
Sand roughness, Ks=	0.00015	m
Skin friction factor, f =	0.08	
$1/\sqrt{f}$ =	3.5	
$-2.0\log10((Ks/3.71DH)+(2.51/Re\sqrt{f}))$	3.5	
Average shear stress, τ_0 =	101	kg/m^2
Drag force =	**1133**	kg
Volume of Geotube =	7 * 1.6 * 1.6	
=	17.92	cubic m.
Submerged Unit weight of river bed material =	1000	kg/cubic m.
Self weight of Geotube =	17.92*1000	
=	17920	kg
Frictional coefficient, μ =	0.6	
Horizontal resisting force =	17920*0.6	
=	**10752**	kg

Horizontal resisting force greater than drag force and Water Current Force

Hence safe

<u>Stability of Polymer Geotubes at Punarak</u>

Max discharge is in 1996(as per data) =	61000	cumecs
Normal scour depth as per Lacey =	$0.473(Q/f)^{1/3}$	
silt factor,f =	1	
Normal scour depth,d =	18.62	m
local scour depth =	28	m
Depth of scour below river bed =	9.38	m
Waterway width(from 2007, satellite image) =	1800	m
Water depth(from scour depth consideration) =	18.62	m
Area of waterway =	1800*18.62	
=	33516	sq.m
Perimeter, P =	1800+ 2*18.62	
=	1837.00	m
Hydraulic mean redius, R =	33516/183:	m
=	18	m
Channel bed slope,s =	1 / 7000	
	0.00014	

Length of Geotube =	7	m
Width of Geotube =	1.6	m
Height of Geotube =	1.6	m
sediment on bed,d_{50} =	0.2	mm

Garde Rangaraju approach

Now R/d = 18 / (0.2*1000)

= 90000

$S / (\Delta\rho s / \rho)$ = 0.00014

The flow regime falls under Antidune regime

From fig(5.4, Garde Rangaraju), corresponding to d_{50} =	0.2	mm
k_1 =	1.3	
k_2 =	1.1	
$k_2 (R/d)^{1/3} * (S/ (\Delta\gamma s/\gamma f))$ =	0.00690	
c_1 =	0.3	
velocity =	$c1/k1*\sqrt{(\Delta\gamma s/\rho f)}$	
	3.100	m/sec

Water Current Pressure, P = KV^2

K =	78.8	
P =	78.8 * 3.1^2	
	757.27	kg/m²
Water current force =	7*1.6*757.27	
	8481	kg

Drag Force Aprroach

The average shear stress on the wetted surface, $\tau_0 = C_d\rho V^2/2$

$= f\rho V^2/8$

Colebrook - White formula for estimating the friction factor

$$1/\sqrt{f} = -2.0\log_{10}((K_s/3.71D_H)+(2.51/R_e\sqrt{f}))$$

$$Re=\rho VD_H/\mu$$

Density of water,ρ =	1000	kg/m³
Dynamic viscousity, μ =	0.001	kg/m/s
Velocity of water, V=	3.100	m/sec
D_H =	72	m
Reynolds number, Re =	620	
Sand roughness, Ks=	0.0002	m
Skin friction factor, f =	0.075	
$1/\sqrt{f}$ =	3.7	
$-2.0\log10((Ks/3.71DH)+(2.51/Re\sqrt{f}))$	3.7	
Average shear stress, τ_0 =	90	kg/m²
Drag force =	**1009**	kg

Volume of Geotube =	7 * 1.6 * 1.6	
=	17.92	cubic m.
Submerged Unit weight of river bed material =	1000	kg/cubic m.

Self weight of Geotube = 17.92*1000
= 17920 kg
Frictional coefficient, μ = 0.6
Horizontal resisting force = 17920*0.6
= **10752** kg

Horizontal resisting force greater than drag force and Water Current Force

Hence safe

<u>Stability of Geotubes at Digha</u>

Max discharge is in 1994 & 2003(as per data) = 77000
Max discharge in the northern channel(as per data) = 38500 cumecs
Normal scour depth as per Lacey = $0.473(Q/f)^{1/3}$
silt factor,f = 1
Normal scour depth,d = 15.97 m
local scour depth = 24 m
Depth of scour below river bed = 8.03 m

Waterway width(from 2007, satellite image) = 1200 m
Water depth(from scour depth consideration) = 15.97 m
Area of waterway = 1200*15.972
= 19166 sq.m
Lacey's Perimeter, P = 1200+ 2*15.972
= 1232.00 m
Hydraulic mean redius, R = 19166/123: m
= 16 m
Channel bed slope,s = 1 / 7000
0.00014
Length of Geotube = 7 m
Width of Geotube = 1.6 m
Height of Geotube = 1.6 m
sediment on bed,d_{50} = 0.25 mm

<u>Garde Rangaraju approach</u>

Now R/d = 16 / (0.25*1000)
= 64000
S / ($\Delta\rho s$ / ρ) = 0.00014

The flow regime falls under Antidune regime

From fig(5.4, Garde Rangaraju), corresponding to d_{50} = 0.25 mm
k_1 = 1.3
k_2 = 1.1
$k_2 (R/d)^{1/3}$ * (S/ ($\Delta\gamma s/\gamma f$)) = 0.00616
c_1 = 0.3
velocity = $c1/k1*\sqrt{(\Delta\gamma s/\rho f)}$
2.920 m/sec

Water Current Pressure, P = KV^2
K = 78.8
P = 78.8 * 2.92^2
671.88 kg/m²

Water current force = 7*1.6*671.88

7525 kg

Drag Force Aprroach

The average shear stress on the wetted surface, $\tau_0 = C_d \rho V^2/2$

$= f\rho V^2/8$

Colebrook - White formula for estimating the friction factor

$1/\sqrt{f} = -2.0\log_{10}((K_s/3.71D_H)+(2.51/R_e\sqrt{f}))$

$Re=\rho V D_H/\mu$

Density of water, ρ =	1000	kg/m^3
Dynamic viscousity, μ =	0.001	kg/m/s
Velocity of water, V=	2.920	m/sec
D_H =	64	m
Reynolds number, Re =	730	
Sand roughness, Ks=	0.00025	m
Skin friction factor, f =	0.077	
$1/\sqrt{f}$ =	3.6	
$-2.0\log10((Ks/3.71DH)+(2.51/Re\sqrt{f}))$	3.8	
Average shear stress, τ_0 =	82	kg/m^2
Drag force =	**919**	kg

Volume of Geotube = 7 * 1.6 * 1.6

= 17.92 cubic m.

Submerged Unit weight of river bed material = 1000 kg/cubic m.

Self weight of Geotube = 17.92*1000

= 17920 kg

Frictional coefficient, μ = 0.6

Horizontal resisting force = 17920*0.6

= **10752** kg

Horizontal resisting force greater than drag force and Water Current Force

Hence safe

<u>Stability of Geotubes at Arjunpur</u>

Max discharge is in 1994(as per data) =	47100	cumecs
Normal scour depth as per Lacey =	$0.473(Q/f)^{1/3}$	
silt factor,f =	1	
Normal scour depth,d =	17.08	m
local scour depth =	26	m
Depth of scour below river bed =	8.92	m

Waterway width(from 2007, satellite image) =	2600	m
Water depth(from scour depth consideration) =	17.08	m
Area of waterway =	2600*17.082	
=	44413	sq.m

Lacey's Perimeter, P = 2600+ 2*17.082
= 2634.00 m
Hydraulic mean redius, R = 44413/263‹ m
= 17 m
Channel bed slope,s = 1 / 7000
0.00014
Length of Geotube = 7 m
Width of Geotube = 1.6 m
Height of Geotube = 1.6 m
sediment on bed,d_{50} = 0.3 mm

Garde Rangaraju approach

Now R/d = 17 / (0.3*1000)
= 56667
$S / (\Delta\rho s / \rho)$ = 0.00014

The flow regime falls under Antidune regime

From fig(5.4, Garde Rangaraju), corresponding to d_{50} = 0.3 mm
k_1 = 1.3
k_2 = 1.1
$k_2 (R/d)^{1/3} * (S/ (\Delta\gamma s/\gamma f))$ = 0.00592
c_1 = 0.3
velocity = $c1/k1*\sqrt{(\Delta\gamma s/\rho f)}$
3.010 m/sec

Water Current Pressure, P = KV^2
K = 78.8
P = 78.8 * 3.01^2
713.94 kg/m²
Water current force = 7*1.6*713.94
7996 kg

Drag Force Aprroach

The average shear stress on the wetted surface, $\tau_0 = C_d\rho V^2/2$

$= f\rho V^2/8$

Colebrook - White formula for estimating the *friction factor*

$1/\sqrt{f} = -2.0\log_{10}((K_s/3.71D_H)+(2.51/R_e\sqrt{f}))$

$Re=\rho VD_H/\mu$

Density of water,ρ =	1000	kg/m^3
Dynamic viscousity, μ =	0.001	kg/m/s
Velocity of water, V=	3.010	m/sec
D_H =	68	m
Reynolds number, Re =	903	
Sand roughness, Ks=	0.0003	m
Skin friction factor, f =	0.065	

$1/\sqrt{f}$ =	3.9	
$-2.0\log10((Ks/3.71DH)+(2.51/Re\sqrt{f}))$	3.9	
Average shear stress, τ_0 =	74	kg/m^2
Drag force =	**824**	kg
Volume of Geotube =	7 * 1.6 * 1.6	
=	17.92	cubic m.
Submerged Unit weight of river bed material =	1000	kg/cubic m.
Self weight of Geotube =	17.92*1000	
=	17920	kg
Frictional coefficient, μ =	0.6	
Horizontal resisting force =	17920*0.6	
=	**10752**	kg

Horizontal resisting force greater than drag force and Water Current Force

Hence safe

<u>Stability of Geotubes at Debchandpur</u>

Max discharge is in 2003 & 2004(as per data) =	33972	cumecs
Normal scour depth as per Lacey =	$0.473(Q/f)^{1/3}$	
silt factor,f =	1	
Normal scour depth,d =	15.32	m
local scour depth =	23	m
Depth of scour below river bed =	7.68	m
Waterway width(from 2007, satellite image) =	1800	m
Water depth(from scour depth consideration) =	15.32	m
Area of waterway =	1800*15.319	
=	27574	sq.m
Lacey's Perimeter, P =	1800+ 2*15.319	
=	1831.00	m
Hydraulic mean redius, R =	27574/183	m
=	15	m
Channel bed slope,s =	1 / 7000	
	0.00014	
Length of Geotube =	7	m
Width of Geotube =	1.6	m
Height of Geotube =	1.6	m
sediment on bed,d_{50} =	0.35	mm

<u>Garde Rangaraju approach</u>

Now R/d =	15 / (0.35*1000)	
=	42857	
$S / (\Delta\rho s / \rho)$ =	0.00014	

The flow regime falls under Antidune regime

From fig(5.4, Garde Rangaraju), corresponding to d_{50} =	0.35	mm
k_1 =	1.3	
k_2 =	1.1	
$k_2 (R/d)^{1/3} * (S/ (\Delta\gamma s/\gamma f))$ =	0.00539	

$c_1 =$ 0.3

velocity = $c1/k1*\sqrt{(\Delta\gamma s/\rho f)}$

2.830 m/sec

Water Current Pressure, P = KV^2

K = 78.8

P = 78.8 * 2.83^2

631.10 kg/m^2

Water current force = 7*1.6*631.1

7068 kg

Drag Force Aprroach

The average shear stress on the wetted surface, $\tau_0 = C_d\rho V^2/2$

$= f\rho V^2/8$

Colebrook - White formula for estimating the friction factor

$$1/\sqrt{f} = -2.0\log_{10}((K_s/3.71D_H)+(2.51/R_e\sqrt{f}))$$

$$Re=\rho VD_H/\mu$$

Density of water, ρ =	1000	kg/m^3
Dynamic viscousity, μ =	0.001	kg/m/s
Velocity of water, V=	2.830	m/sec
D_H =	60	m
Reynolds number, Re =	990.5	
Sand roughness, Ks=	0.00035	m
Skin friction factor, f =	0.063	
$1/\sqrt{f}$ =	4.0	
$-2.0\log10((Ks/3.71DH)+(2.51/Re\sqrt{f}))$	4.0	
Average shear stress, τ_0 =	63	kg/m^2
Drag force =	**706**	kg

Volume of Geotube = 7 * 1.6 * 1.6

= 17.92 cubic m.

Submerged Unit weight of river bed material = 1000 kg/cubic m.

Self weight of Geotube = 17.92*1000

= 17920 kg

Frictional coefficient, μ = 0.6

Horizontal resisting force = 17920*0.6

= **10752** kg

Horizontal resisting force greater than drag force and Water Current Force

Hence safe

<u>Stability of Geotubes at Nakhwa</u>

Max discharge is in 2003 & 2004(as per data) = 33972 cumecs

Normal scour depth as per Lacey = $0.473(Q/f)^{1/3}$

silt factor,f = 1

Normal scour depth,d = 15.32 m

local scour depth = 23 m

Depth of scour below river bed = 7.68 m

Waterway width(from 2007, satellite image) = 1500 m

Water depth(from scour depth consideration) = 15.32 m

Area of waterway = 1500*15.319

= 22979 sq.m

Lacey's Perimeter, P = 1500+ 2*15.319

= 1531.00 m

Hydraulic mean redius, R = 22978/153 m

= 15 m

Channel bed slope,s = 1 / 7000

0.00014

Length of Geotube = 7 m

Width of Geotube = 1.6 m

Height of Geotube = 1.6 m

sediment on bed,d_{50} = 0.4 mm

Garde Rangaraju approach

Now R/d = 15 / (0.4*1000)

= 37500

$S / (\Delta\rho s / \rho)$ = 0.00014

The flow regime falls under Antidune regime

From fig(5.4, Garde Rangaraju), corresponding to d_{50} = 0.4 mm

k_1 = 1.3

k_2 = 1.1

$k_2 (R/d)^{1/3} * (S/ (\Delta\gamma s/\gamma f))$ = 0.00515

c_1 = 0.3

velocity = $c1/k1*\sqrt{(\Delta\gamma s/\rho f)}$

2.830 m/sec

Water Current Pressure, P = KV^2

K = 78.8

P = 78.8 * 2.83^2

631.10 kg/m^2

Water current force = 7*1.6*631.1

7068 kg

Drag Force Aprroach

The average shear stress on the wetted surface, $\tau_0 = C_d\rho V^2/2$

$= f\rho V^2/8$

Colebrook - White formula for estimating the friction factor

$$1/\sqrt{f} = -2.0\log_{10}(K_s/3.71D_H)+(2.51/R_e\sqrt{f}))$$

$$Re=\rho VD_H/\mu$$

Density of water,ρ = 1000 kg/m^3

Dynamic viscousity, μ =	0.001	kg/m/s
Velocity of water, V=	2.830	m/sec
D_H =	60	m
Reynolds number, Re =	1132	
Sand roughness, Ks=	0.0004	m
Skin friction factor, f =	0.06	
$1/\sqrt{f}$ =	4.1	
$-2.0\log10((Ks/3.71DH)+(2.51/Re\sqrt{f}))$	4.1	
Average shear stress, τ_0 =	60	kg/m^2
Drag force =	**673**	kg

Volume of Geotube =	7 * 1.6 * 1.6	
=	17.92	cubic m.
Submerged Unit weight of river bed material =	1000	kg/cubic m.
Self weight of Geotube =	17.92*1000	
=	17920	kg
Frictional coefficient, μ =	0.6	
Horizontal resisting force =	17920*0.6	
=	**10752**	kg

Horizontal resisting force greater than drag force and Water Current Force

Hence safe

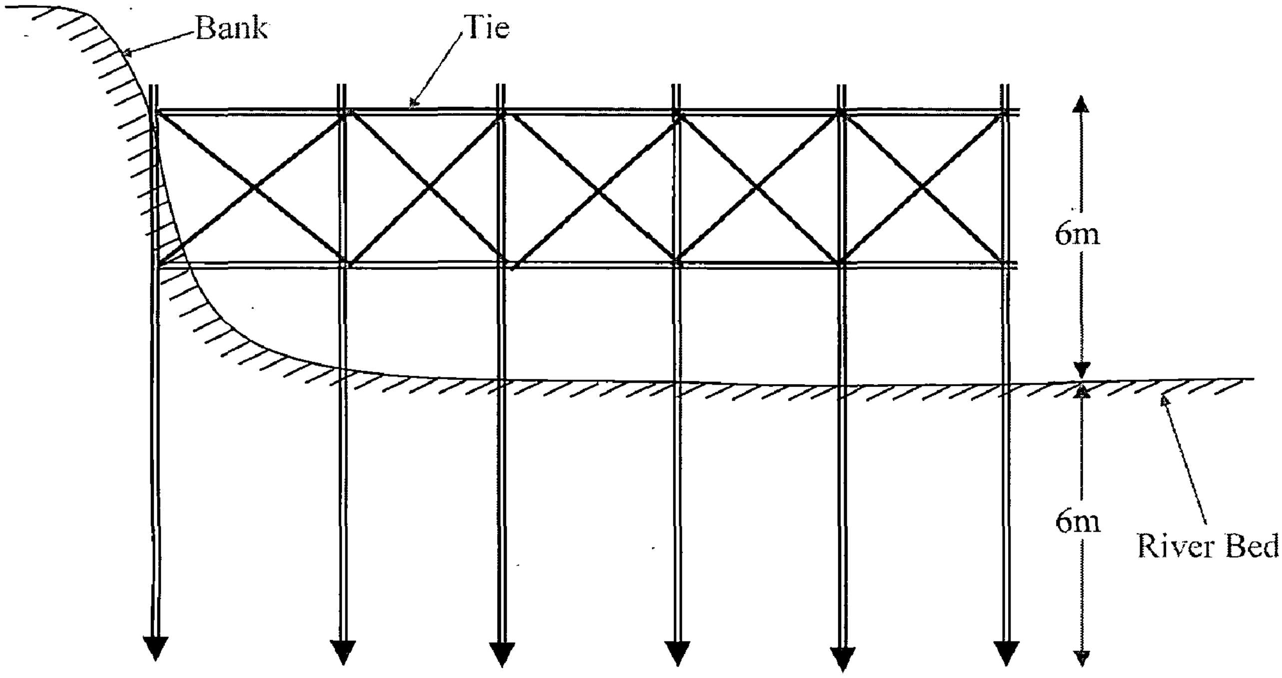
5.4 Permeable Spur

These allow some restricted flow of water to pass through them. These types of spur partially obstruct the flow and slacken it to cause deposition of sediment carried by the river. Therefore permeable spurs are classified as sedimenting or silting spurs. They are best suited for the type of rivers which carry considerable amount of sediment in suspension, e.g., alluvial river in plains. Permeable spurs are most effective on alluvial streams with considerable bed load and high sediment concentration, which favours rapid deposition around the spurs (Beckstead, 1975; Alam and Faruque, 1986).

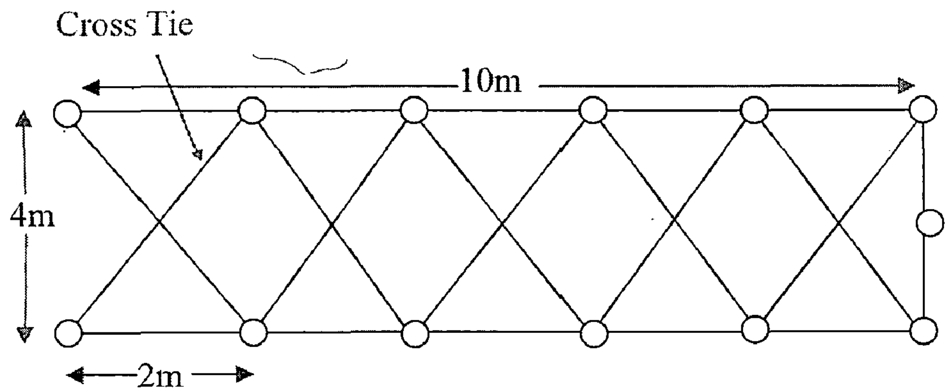
As the sediment accumulates between spurs in series, the foreshore becomes more or less permanent soil such the there is no need to use any other material for its protection. However, in the rivers which carry relatively clear water, the action of such permeable spurs is to slow down the flow, thereby dampen the erosion strength of the current and thus also to prevent the local bank erosion upto some distance upstream and downstream. Due to above mentioned factors, United Nations Economic Commission For Asia And The Far East (1953) recommends permeable type of spurs to be spaced further apart than the solid types.

The advantages of permeable spurs may be summarized as below

1. They are relatively cheap because they need only temporary or semi permanent construction.
2. As small quantity of stones are required for construction of permeable spurs, they are specially suited for places where stones are scarce.
3. Permeable spurs are more effective in the regulation of river courses or in the protection of banks especially in a silt laden river than the impermeable spurs.
4. There are not serious eddies or scour holes in case of permeable spurs as in the case of solid spurs because the flow discharge does not change abruptly like in solid spurs.
5. In case of deep and narrow rivers, where depth are considerable, solid spurs become expensive and may cause undesirable flow conditions. But when permeable spurs are employed, they do provide necessary bank protection without much cost and serious eddy and scours formation.
6. The permeable spurs may be spaced farther apart than solid spurs.



Sectional Elevation



Plan

Fig 5.1 Details of Permeable Spur

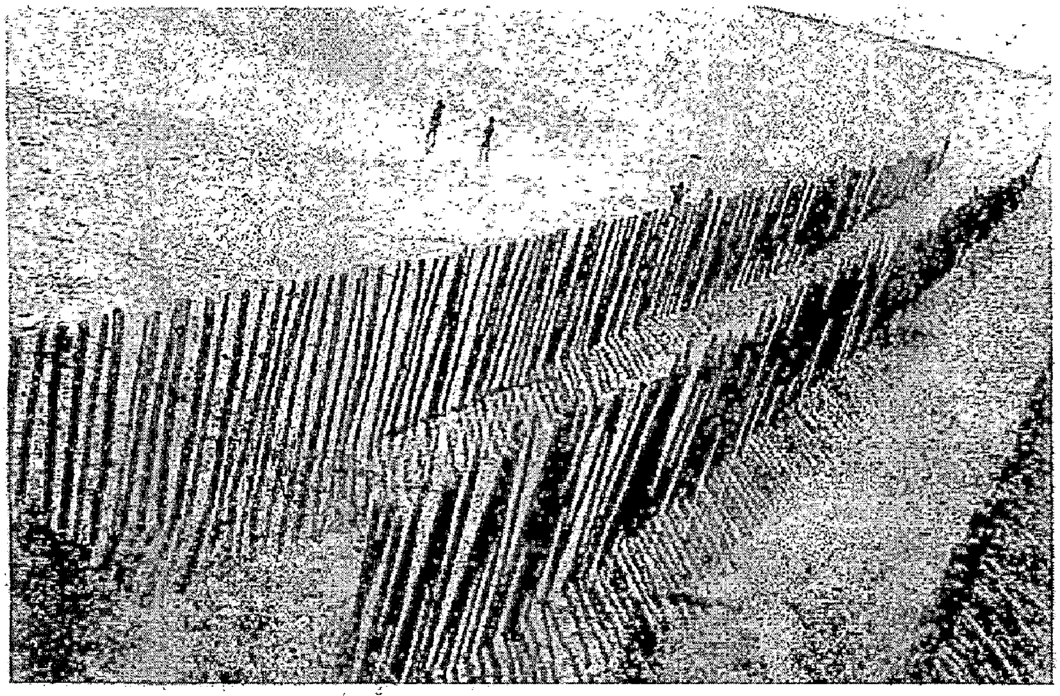


Fig 5.2 : **PERMEABLE SPURS AT GUMI ASSAM IN RIVER BRAHAMAPUTRA**

Fig 5.3 : **PERMEABLE SPURS AT GURMUKHTESHVER MURADABAD IN RIVER GANGA**

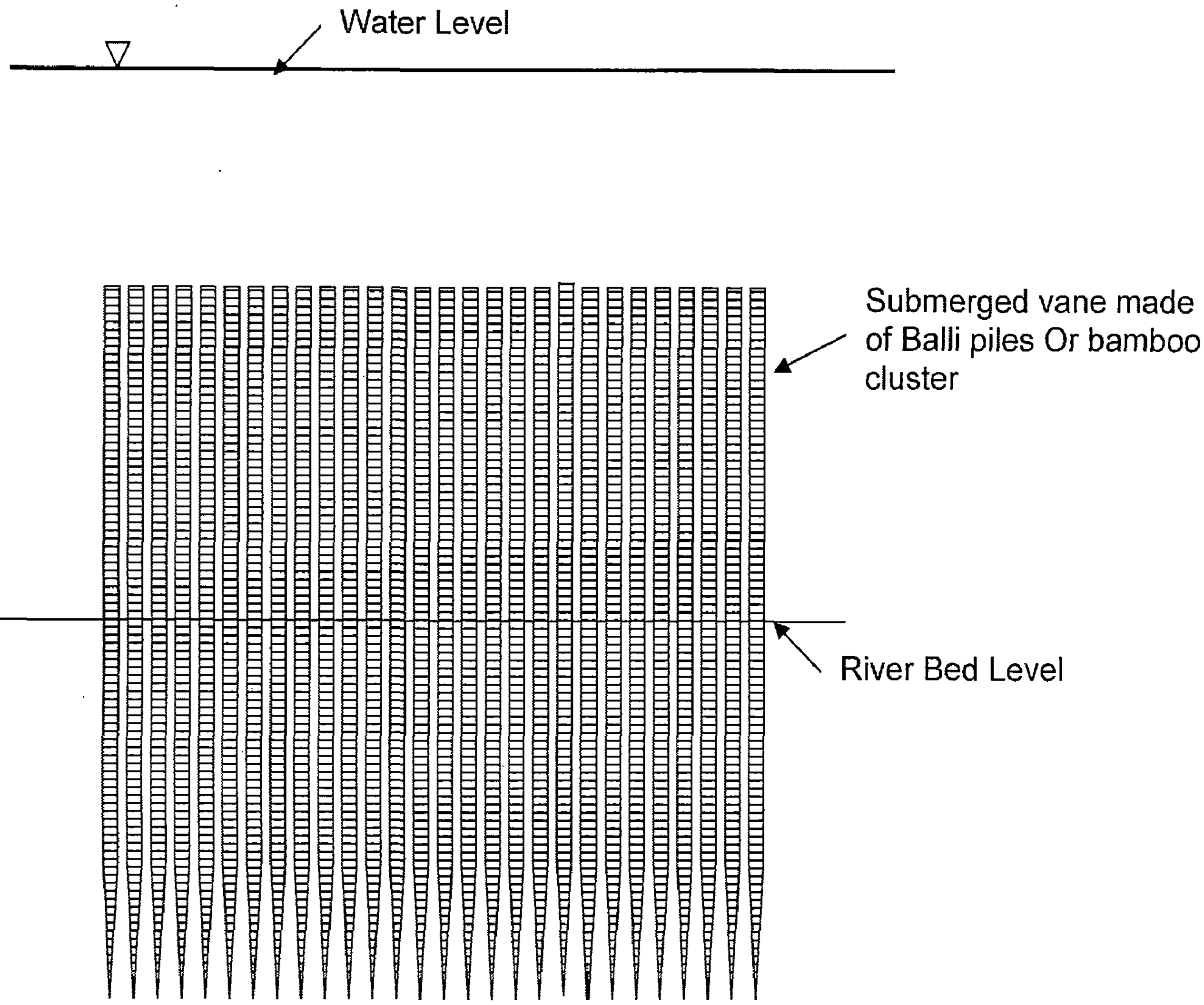


Fig 5.4 Balli Pile Type Submerged vane

5.5 Board Fencing

Numerous other methods have been used to protect eroding banks primarily by private land owners but also at test sites by the USACE,1981. some of these measures are less expensive than the revetments and dikes and most have a shorter useful life expectancy but are considered by environmentalist to be less unsightly that stone vconrete and piling of the other measures. Only fencing of various types and revetment have proven to be generally effective.

Permeable fencing is one of the oldest and most economical types of bank protection used in the upstream of streams of low gradients. It has been used longitudinally roughly parallel to the bank as revetments and at an angle to the flow as dikes. It reduces local velocities, traps debris behind the fence and facilitates sediment deposition and the establishment of native vegetation. Many types of new and used materials have been used for fencing. Posts have been of treated or untreated wood, rails, beams, pipes and concrete and the fencing has generally been steel wire of 4 in mesh or wood. At some locations two rows of fencing have been used parallel to the bank with brush old tires, stone placed between the rows to trap sediment where the bank is high and steep. Only one row of fencing is normally used with filler material placed between the fences and the bank.

Wire fencing was used in some reaches of the Los Angeles flood control project constructed in southern California in the 1930s. a single line of fencing was used in straight reaches and two lines of fencing filled with brush were used in curved reaches. Gildea (1963) reported that fencing performed well along straight or nearly straight reaches in the major flood of 1938 with exceeding design channel capacity but that section failed on the outside of bands and where the main current was directed against the fencing.

Steinberg described the successful use of fencing on the Russian river, California 4 years after installation. Fencing has been in place for more than 10 years at several location on big sand crack, Mississippi and double row fencing filled with stones constructed on the big blue river Kanas in the period 1963-1967 and on the gerin drain Nebraska in the period 1963-1969 was reported as being good condition in the late 1970s.

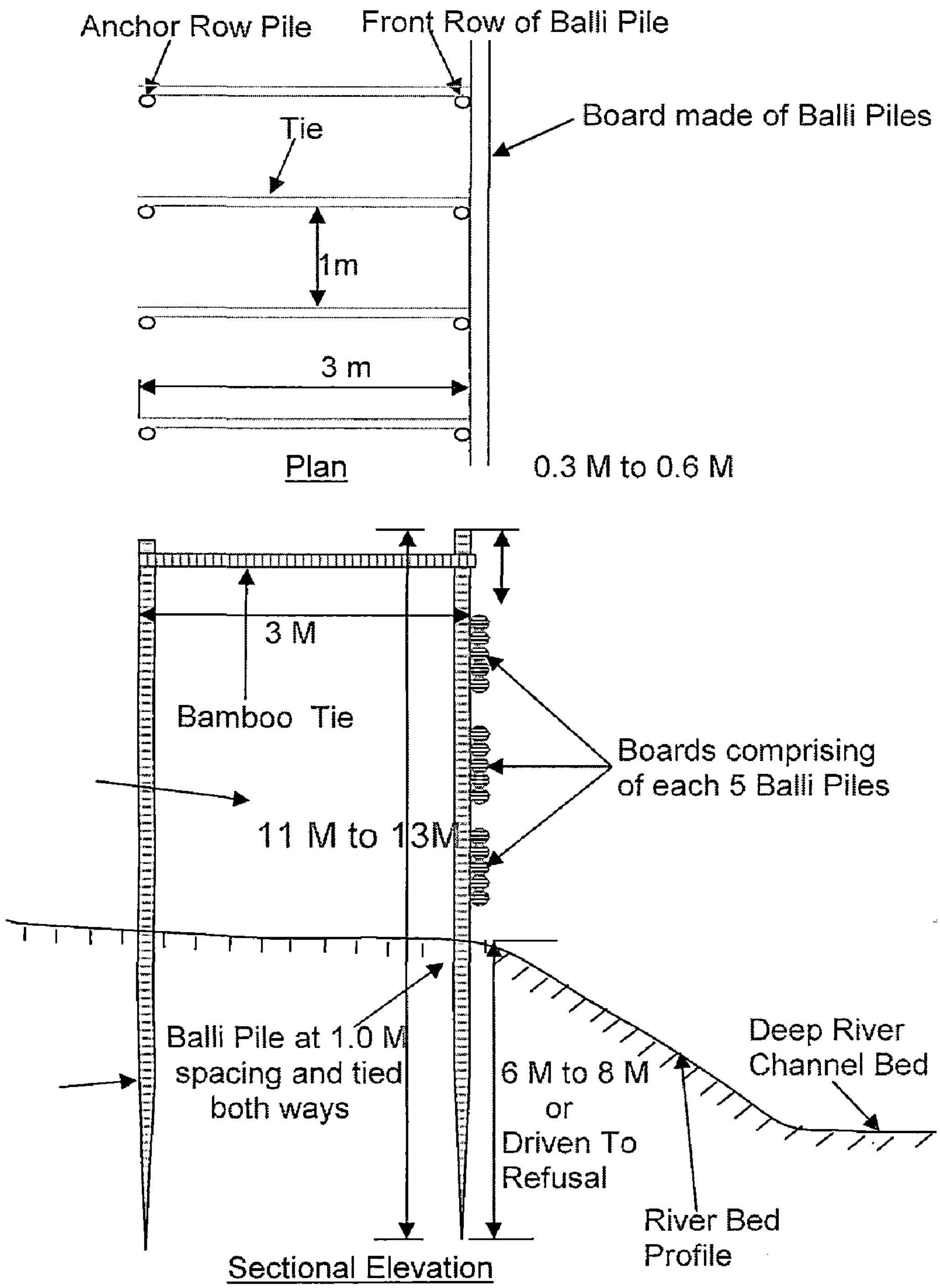


Fig 5.5 Typical Sketch Showing Bank Protection Works (Board Fencing)

5.6 Kellner Jetty

Drawing and specification the Kellner Jetty is attached here. It is composed of three equally long RCC bars of Length = 3.0m, thickness = 10cm, Spacing of Holes = 30 cm. These three bars are bolted together at their mid points. The bars are placed back to back with their longitudinal axis at right angle to each other. The bars are fastened into place and form three sets of intersecting planes, their common point at the center. The planes are maintained by lacing them with wire (Diameter of rope = ¾ inch) at 30cm interval. Then they are linked together in a line with a thick cable to form a jetty. The cables extend in a continuous line through the units and fastened at each end of the jetty which are called deadman anchors. These are standard creosoted 8-foot railroad ties buried about halfway into the ground

Kellner Jetty of 3m length and 10 cm thickness

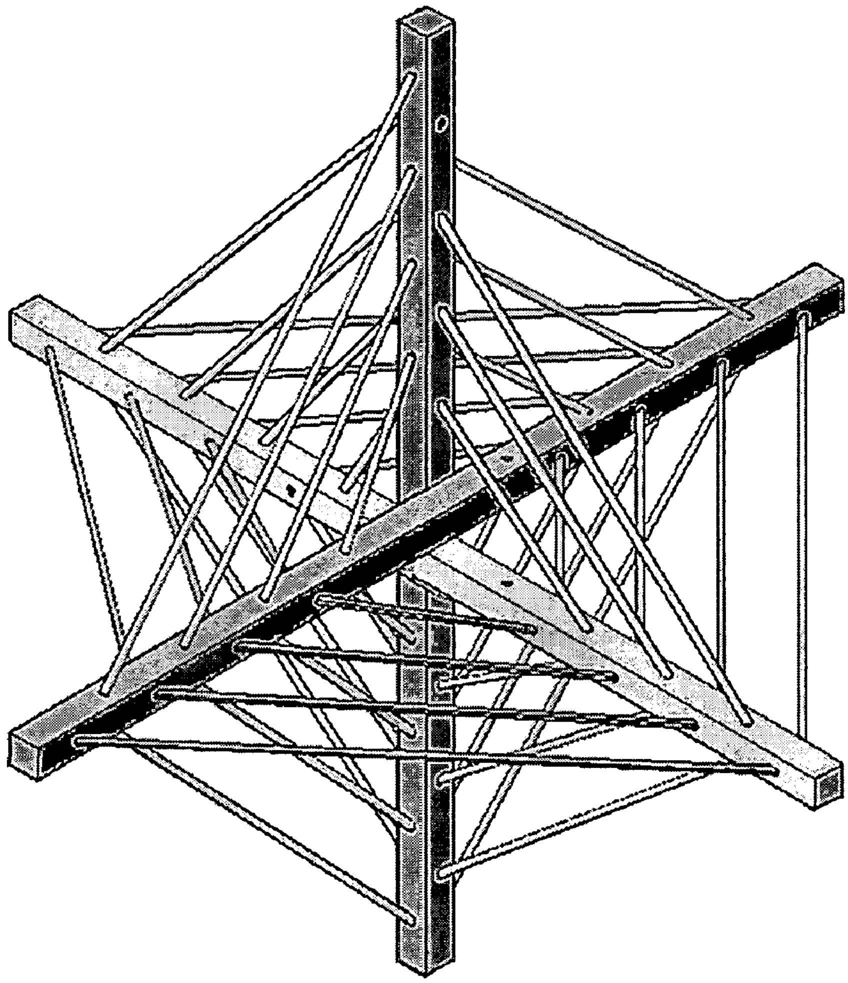
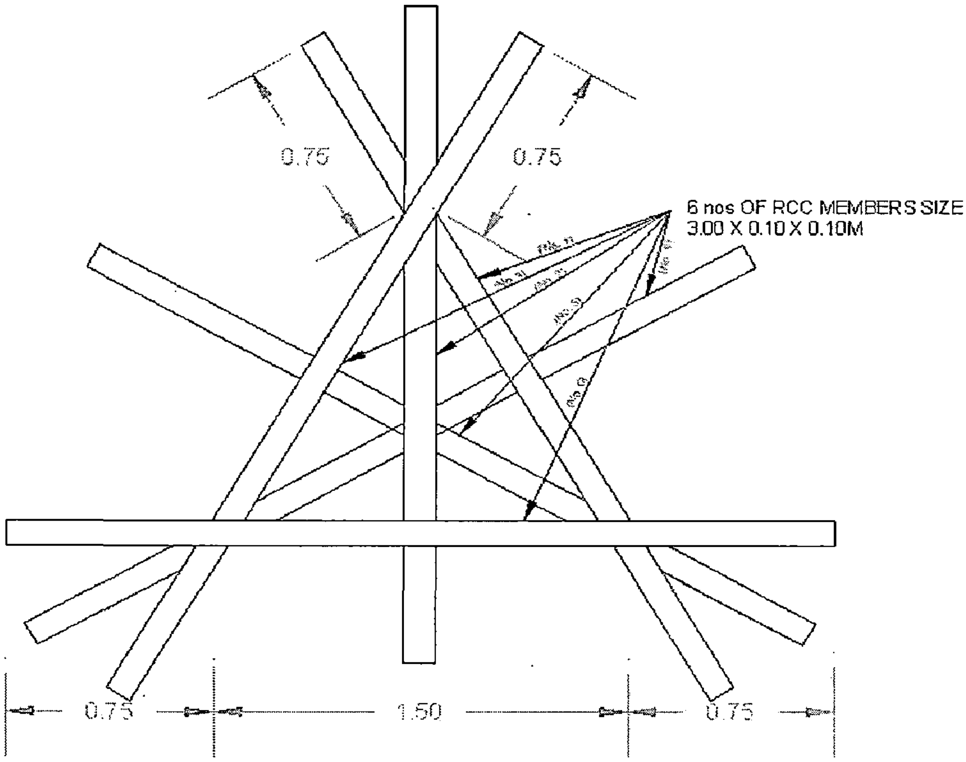


Fig 5.6

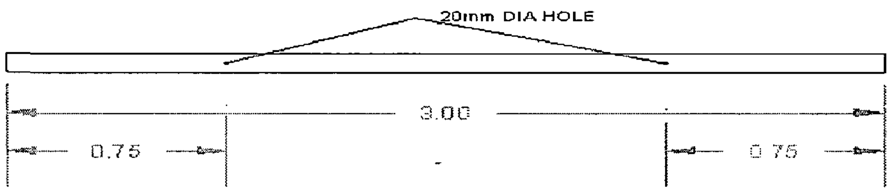
5.6 Porcupines

Alluvial rivers are well known for their sediment load and frequently changing course. Most of the rivers in this class are notorious for overflows and breaching their banks, resulting in the floods. River training works are required to stabilize the river channel along a certain alignment and with a certain cross section so that the river does not cause the damage to the land and property adjacent to its bank)

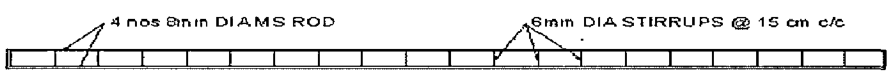
Porcupines are triangular structure made of three poles joined together at the top and braced near the bottom. The geometry of the porcupine is such that it will remain the same in any the position of the porcupine just like a pyramid having three slanting faces. Bamboo porcupines filled with the boulders have been used for training the river flow in the central and eastern Uttar Pradesh and Bihar. But the concept of permeable RCC porcupine is latest and recently used in river training in Brahmaputra River in Assam with a modified design. RCC porcupines, used in Brahmaputra River are made of 3 metre long straight, rectangular RCC members having10cm X10cm cross-section and reinforcement as shown in the Figure 2.2. Two holes.are provided at a suitable distance from both ends of the member to facilitate the fastening of members with each other to give the shape of pyramid to the arrangement of the straight members. All the porcupines used for training the river are joined together with a steel cable wire and anchored at the bank to withstand the forces due to water current. In latest development it has been decided to use RCC porcupine with greater height and hence bigger cross section. RCC porcupines induce sedimentation by breaking and dissipation f erosive vortex current. It works well in sediment carrying river with good amount of suspended sediments.



FRONT ELEVATION OF
3.0M SIZE PORCUPINE



FRONT ELEVATION OF PORCUPINE BAR



DETAILS OF REINFORCEMENT IN PORCUPINE BAR

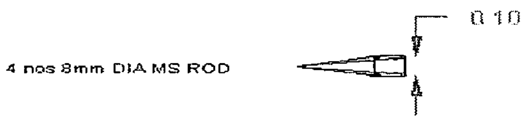
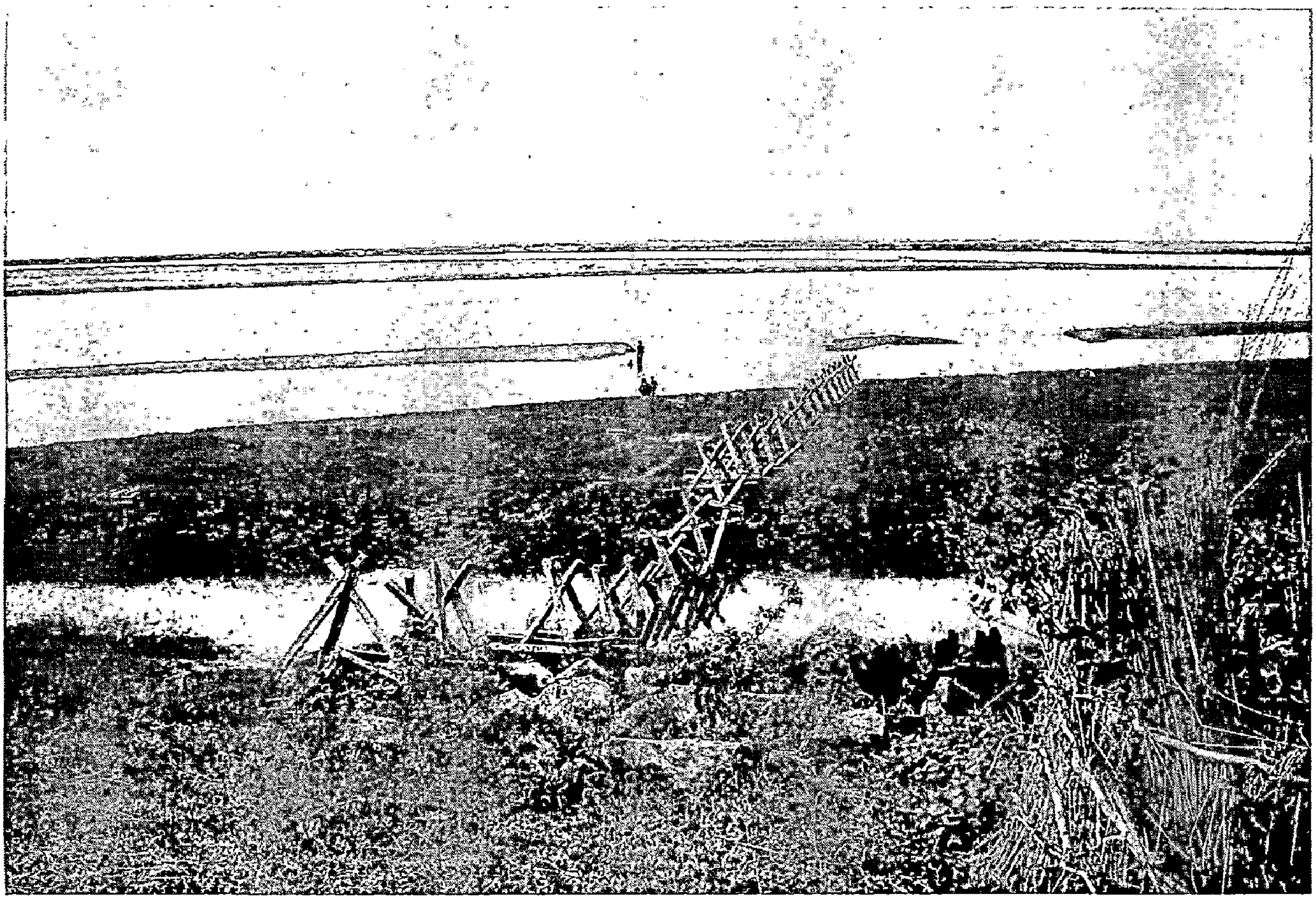
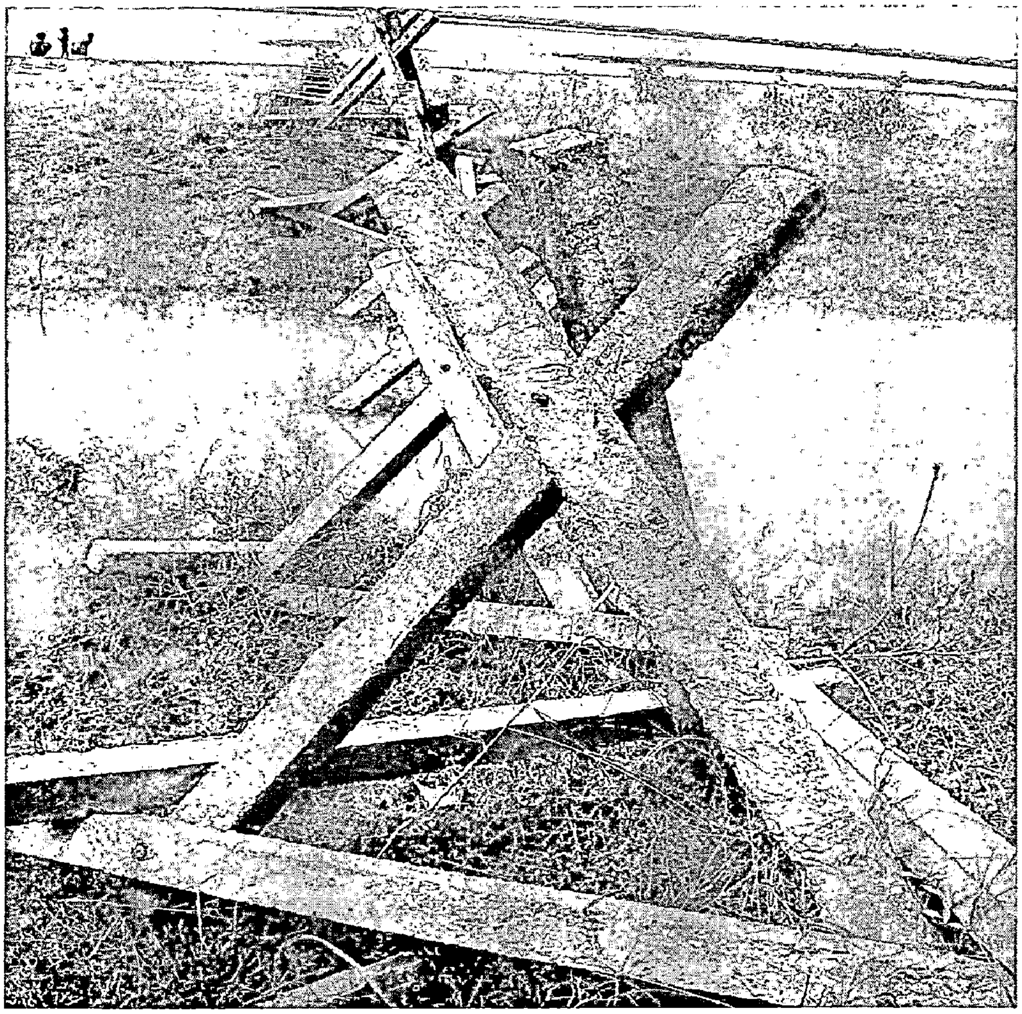


Fig 5.7





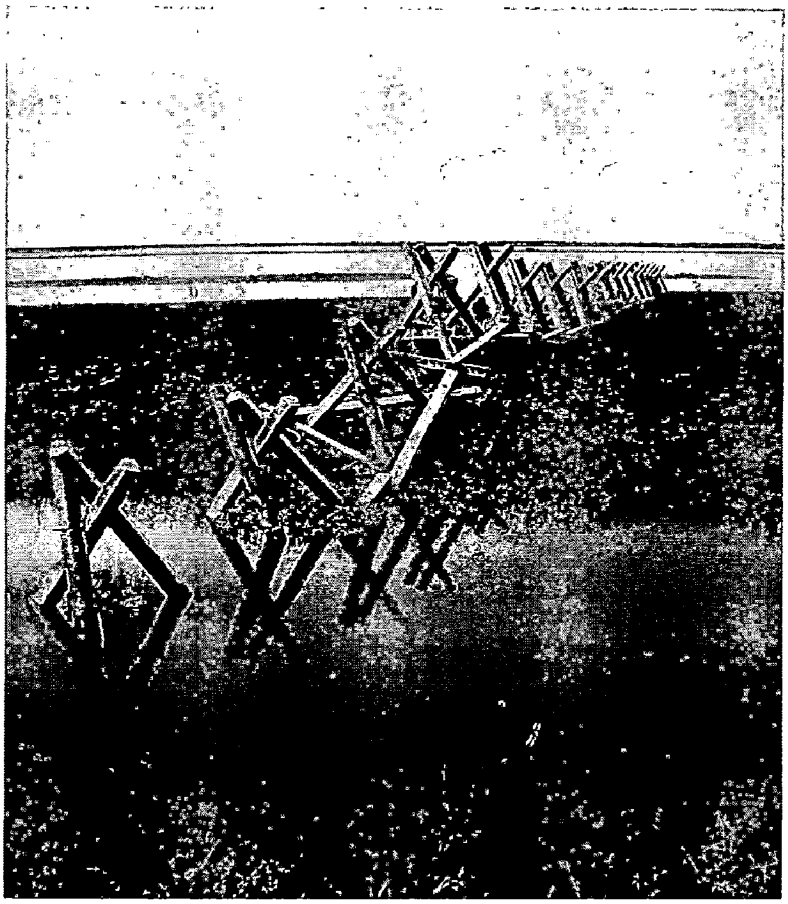


Fig 5.8

5.7 Results and Discussion

Unlike any other civil engineering structures, the river training structures are highly susceptible to unforeseen failures due to considerable inherent uncertainties in the magnitude and nature of fluvial forces, river behavior, as because the science of river hydraulics is still in a developing stage and its critical underlying channel processes are yet to be fully understood. Thus to minimize the possibility of failure of the river training structures, their designs in the present case have been based on the conservative .side without dilution of the measure, but at the same time not raising the costs unreasonably.

Chapter6
Experimental Study of Selected River Training Structures

6.1 General

The experimental programme was organized in one phases. In this phase, experiments were performed in a 50cm wide flume using physical models of porcupines and kellner jetty at one blockage ratio with varying degrees of submergence. In this phase of laboratory experiments, primarily it was endeavored to gain an insight into the physical behavior of different models for assessment of its suitability for installation in field to be followed subsequently. The flow conditions, bed material, degree of submergence and the channel blockage ratio was realistically varied which yielded the desired experimental data for undertaking the required analysis of data.

The main aim of this study is to investigate the scour depth and deposition, velocity field change before and after installation of models with different percentages of submergence which are founded in uniform sediment conditions. Therefore, experiments were conducted to examine the effects on scour depth and deposition pattern of a number of parameters including flow velocity, flow depth, discharge intensity, and sediment size, in clear water conditions. The experiments were conducted to supplement existing data for particular effects, e.g. on banks, while other experiments were designed to produce comprehensive data sets for the effect of parameters either not previously investigated or for which only very limited data were available. The experiments was accomplished using the laboratory flumes which is described in the subsequent sections. With this background, the details of the experimental programme are presented below.

6.2 LABORATORY FLUMES AND OTHER ACCESSORIES

Flume Used In First Phase

This 50 cm wide, 350 cm long flume is fitted with an inbuilt upstream tank along with a separate down stream tank. All three parts of the flume are made up of painted mild steel. The down stream tank is attached with a pump which is 10HP (horse power) in capacity. 10cm diameter pipes connect this pump to the upstream tank of the flume which is deeper than the flume in 1st 30 cm length. The width and depth of the upstream tank are 50 cm and 100 cm respectively while the width and depth of the flume are 50 cm x 50 cm respectively. This flume has side walls made up of transparent Perspex sheet in 240 cm length up to 25 cm before the tail gate.

An evenly perforated Perspex sheet of one cm dia holes and lcm thickness separates the

upper tank from the channel portion of flume at 50 cm distance from the upstream face of the up stream tank. The purpose of this sheet is to stabilize the flow and make it uniform. The upstream pipe drops the water right at the base of the upstream tank to minimize all disturbances. A V-notch is fixed in the down stream tank for discharge measurements. The down steam tank 15.90 cm wide, 90 cm deep, 250 cm long internally. The tail end of the flume is placed roughly 15 cm above the top of the upstream end of the tank This
tank is bifurcated into two stories in first 160cm length on the flume end side, the depth of the upper portion being 40 cm. A V-notch is fixed at the end of upper storey with its vertex placed 15 cm form the base of upper portion at the end of upper portion. A baffle wall is placed to curb all the disturbances of the water falling from the flume into the tank. It is 90cm away from upstream end of the tank. In the bottom portion is placed the opening of the pump close the base of tank 5 cm above it. The flume is mounted on two R.C.C. blocks 65 cm in height from the floor, 66 em wide and 39 cm in length. A threaded rod is grouted in the lower-end block.

The lower end of the flume is mounted over this threaded rod with the help of a very strong nut with handles in such away that the lower portion of flume could be moved up or down max. of 4% for creating the desired slope. The plan of this flume is given in Fig. 3.1Two railings are mounted on the top edges, i.e. left and right sides of the flume over which are placed all the required equipment for measurement of velocity, depths, and levels, etc. Pitot tubes and pointer gauges are placed on separate graduated trolleys. Both the railings were graduated in cm with permanent marker pens and with paint.

For clear water experiments, the working section of the flume commenced 30 cm away from the evenly perforated perspex sheet. Three energy dissipaters were placed a little distance from the sheet in a triangular fashion in this 30cm distance, to make the flow more evenly distributed and break its turbulence and extra energy.

After that 30cm length, 15 cm thick, bed of fine sand is placed which is glued to it in order to make the roughness coefficient of this section same as that of subsequent test bed section. Subsequent to this section, was introduced the test section of uniform sand bed of grain size d_{50} = 0.25mm, 100 cm in length and 50 cm in depth. Subsequent to this section was placed same arrangement of bricks, with the sand cover over 30 em length, as was placed prior to sand bed section.

6.3 Materials Used

The raw material was procured from near by places. This raw mixture was sieved through sieves of 0.85mm size and 0.075mm sieve. For sieve analysis the weight of coarse sand taken was equal to 1000 gm. Sieve analysis of the sediment material of d_{50} = 0.25mm mm is given in Table 3.1.

6.4 Experimental procedure

- First flume was adjusted to required slope
- The depths of sediment test bed layers were fixed as per already calculated depths at 15 cm. Pitot tubes, V -notch were calibrated by the standard procedures and their Cd is 0.8.
- The models were fixed in the layer of sand bed at right angle to the side wall of the flume.
- The required discharges were adjusted against required depth by hit and trial adjustment of the tail-gate of flume and the gate valve of pump and readings V-notch.
- Prior to conducting of each experimental run, the test bed section was leveled.
- Velocities were measured at a grid of 5cm x 10cm for a cross section of 50cm x 100cm. At each section following grid points, were choosen for taking measurements of velocities.
- After equilibrium scour had reached, the flow was stopped and the profile of whole scour hole and the deposition bar so developed were taken with the help of levels measured by the pointer gauge at every lcm or 2cm grid in x and y directions.
- In order to trace the flow pattern of flow lines floats, either wax balls or aluminum foil chips were used. The trajectory of wax balls was mostly in close vicinity of models and through its slits and slots
- Photographs were taken of the developed profiles from both upstream side view and from downstream side view, after the termination of the run.
- Bed profile was plotted with the help of surfer software with actual dimensions in x, y and z direction measured. This was done to ascertain the geometry of the scour hole and the deposition bar.

6.5 Model Experiments

In this phase of experiment two types of models , one porcupine of 10cm length and 2mm thickness and the other Kellner Jetty of the same dimension were deployed to investigate the change in velocity in the test section before and after installation of models. The experiments were run for 6 hours. All the experiments were conducted in clear water conditions. The flow was adjusted to less than critical conditions.

This experiment was done in three stages. In the first stage the flow depth was adjusted to 30cm with a discharge of 0.026cumecs for 30% submergence. Similarly in subsequent stages the flow depth was changed to 20 cm with a discharge of 0.0154cumecs for 50% submergence and 10 cm with a discharge of 0.0057cumecs for 0% submergence. All these experiments were carried out for a blockage ratio of 20% for both types of models.

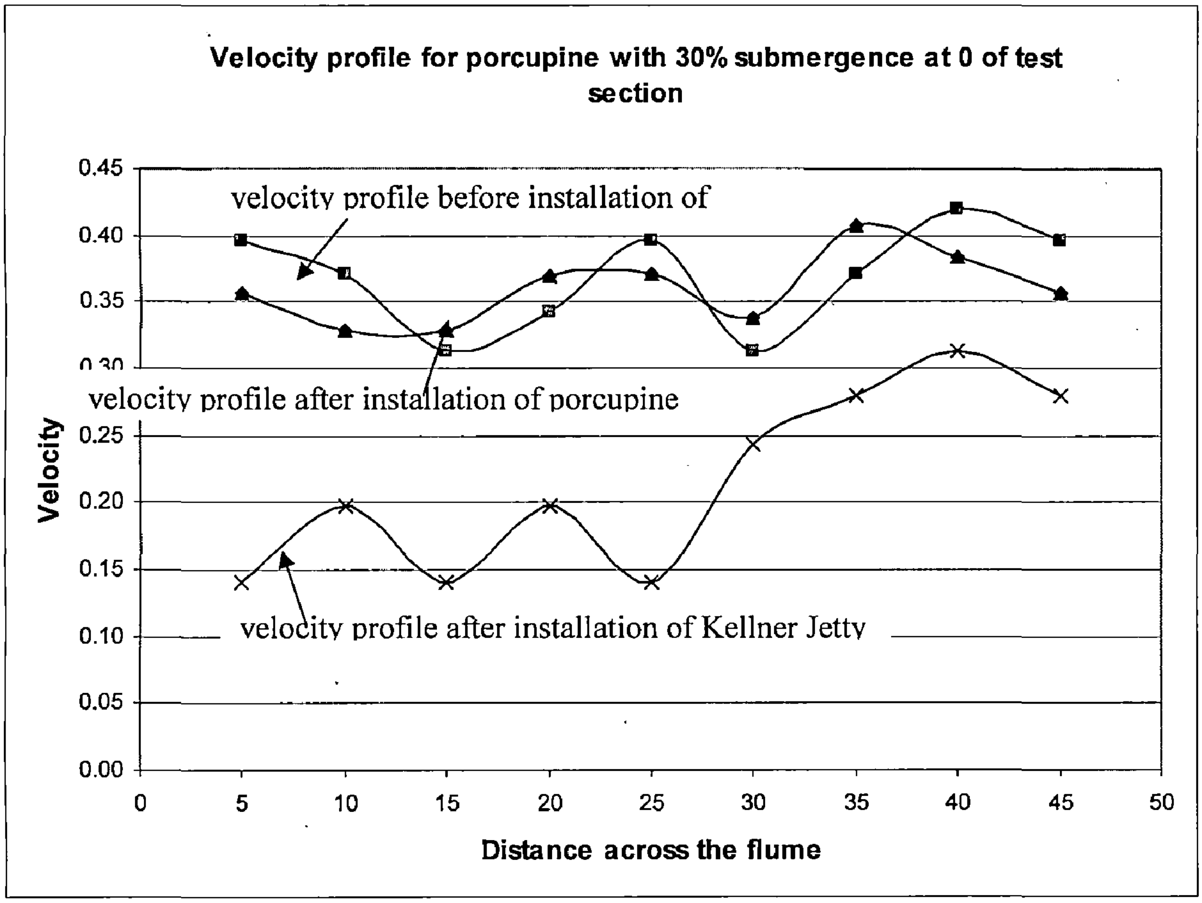


Fig 6.1 Velocity profile with 30% submergence at 0 of cross section

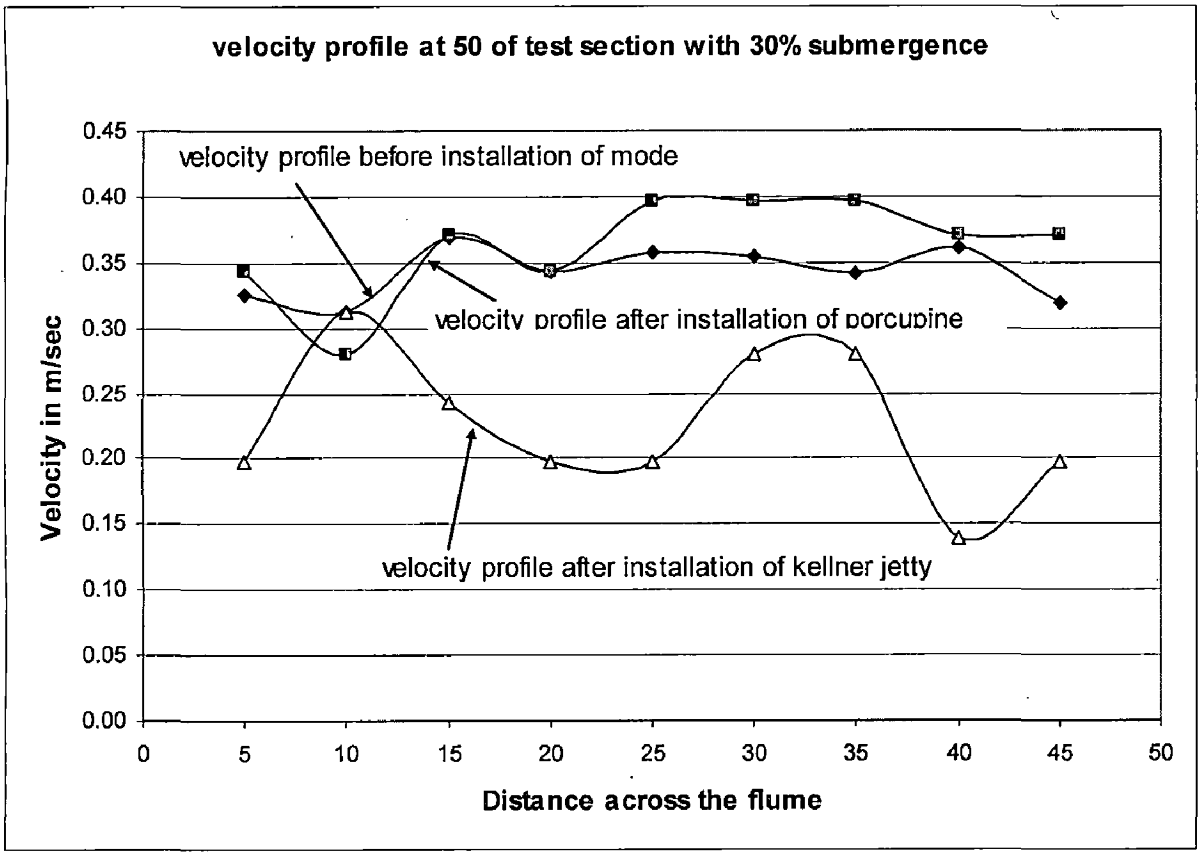


Fig 6.2 Velocity profile with 30% submergence at 50 of cross section

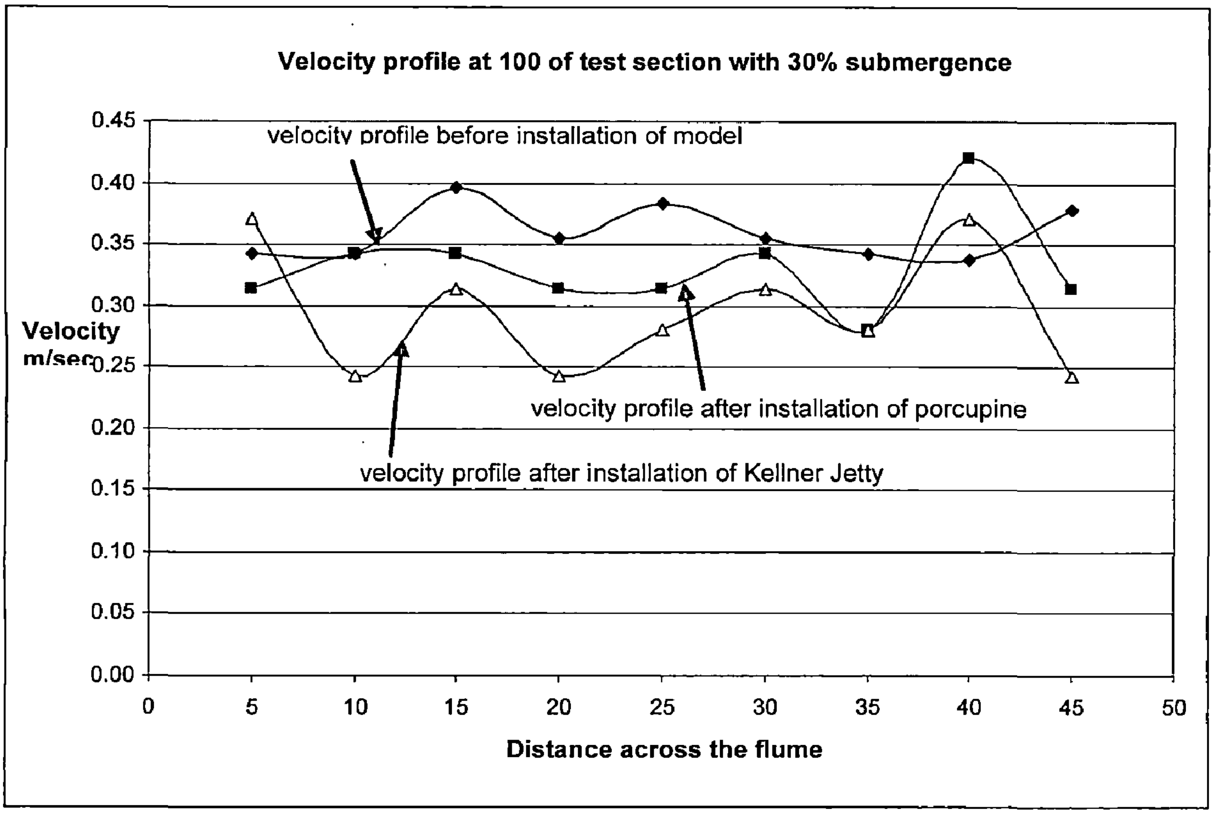


Fig 6.3 Velocity profile with 30% submergence at 100 of cross section

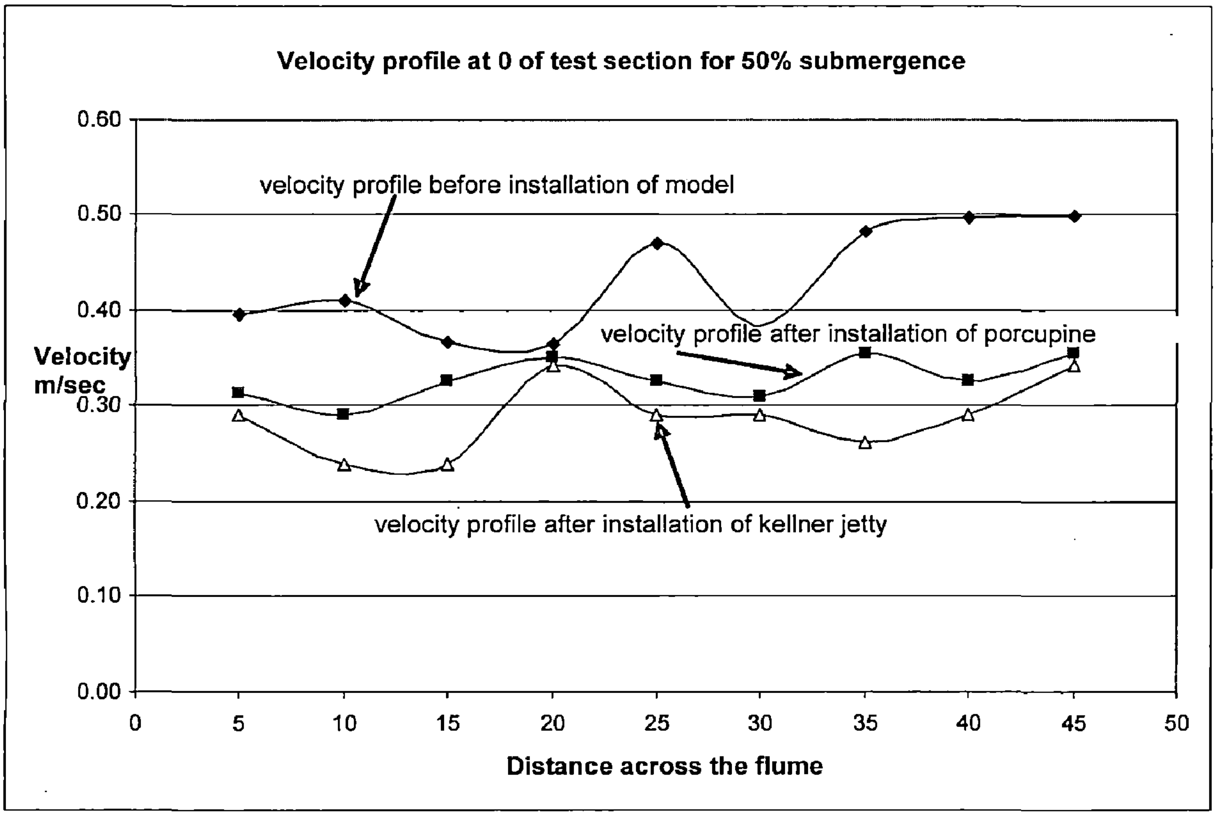


Fig 6.4 Velocity profile with 50% submergence at 0 of cross section

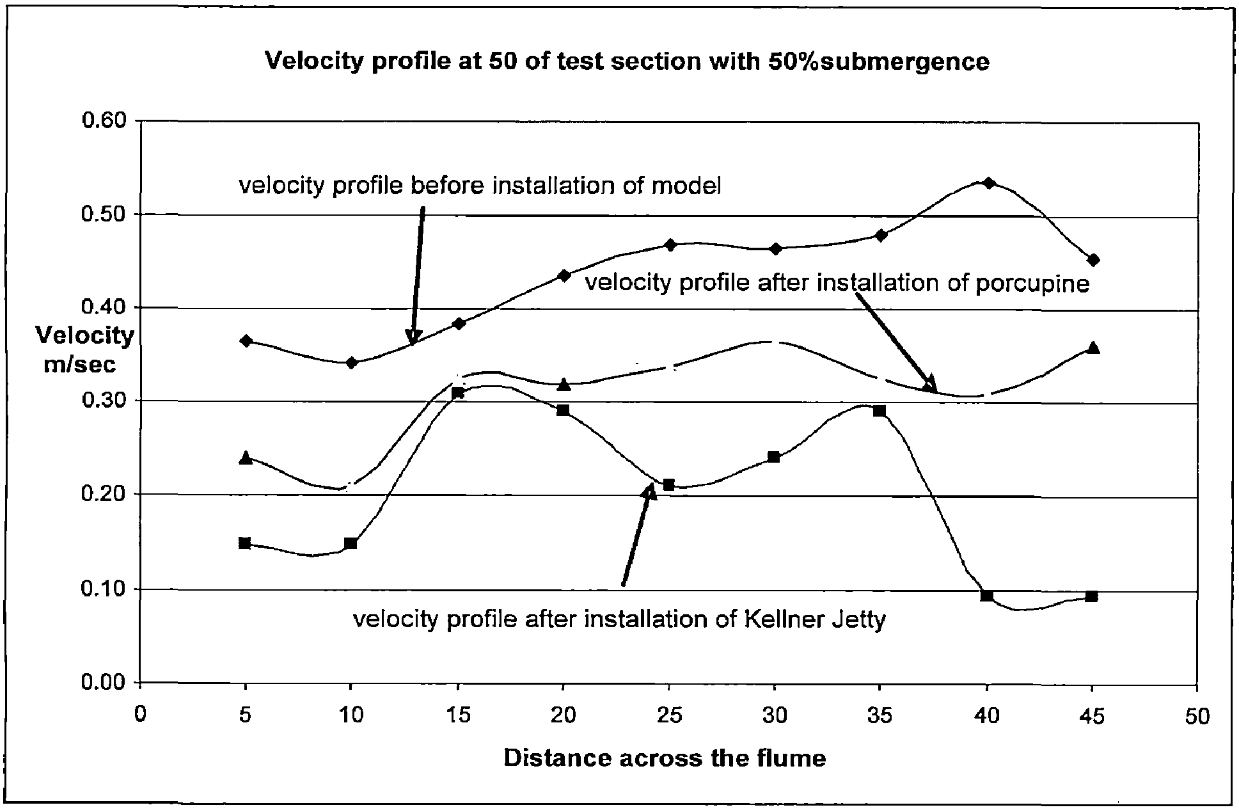


Fig 6.5 Velocity profile with 50% submergence at 50 of cross section

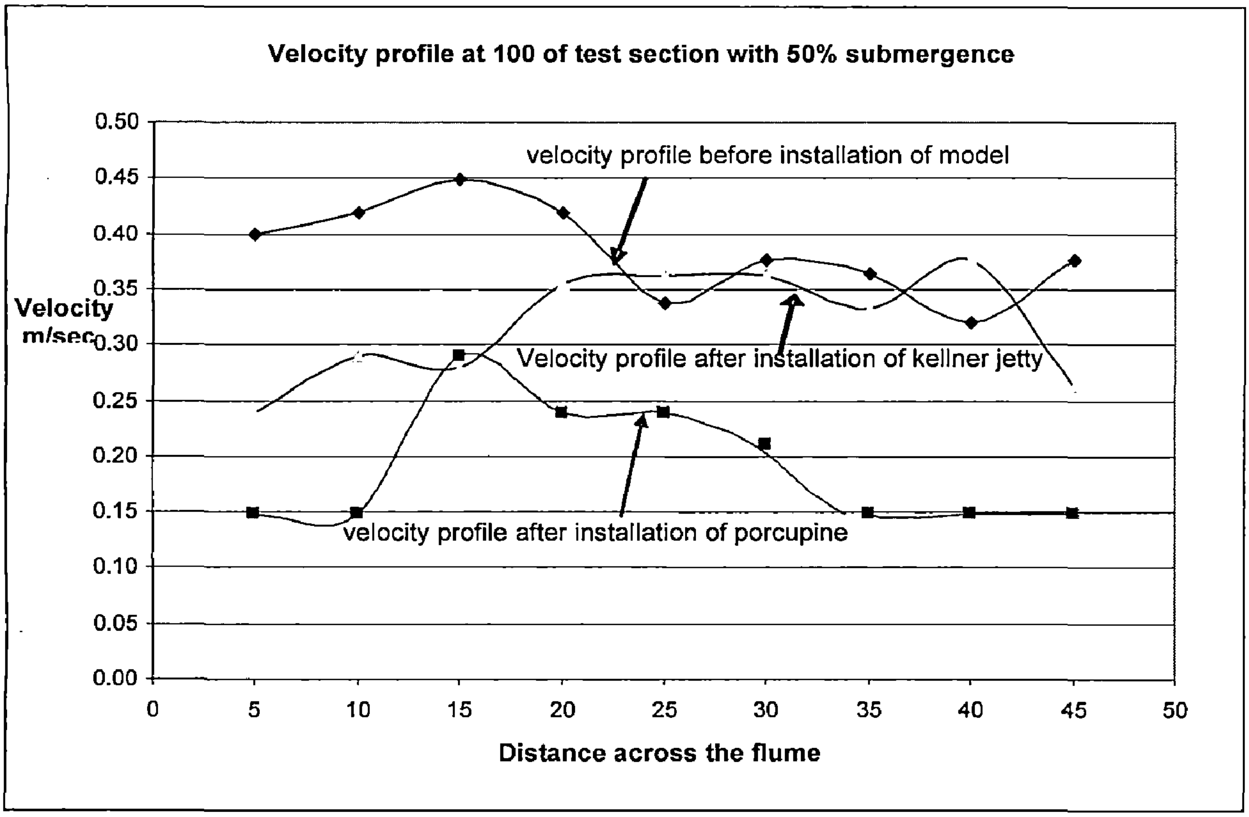


Fig 6.6 Velocity profile with 50% submergence at 100 of cross section

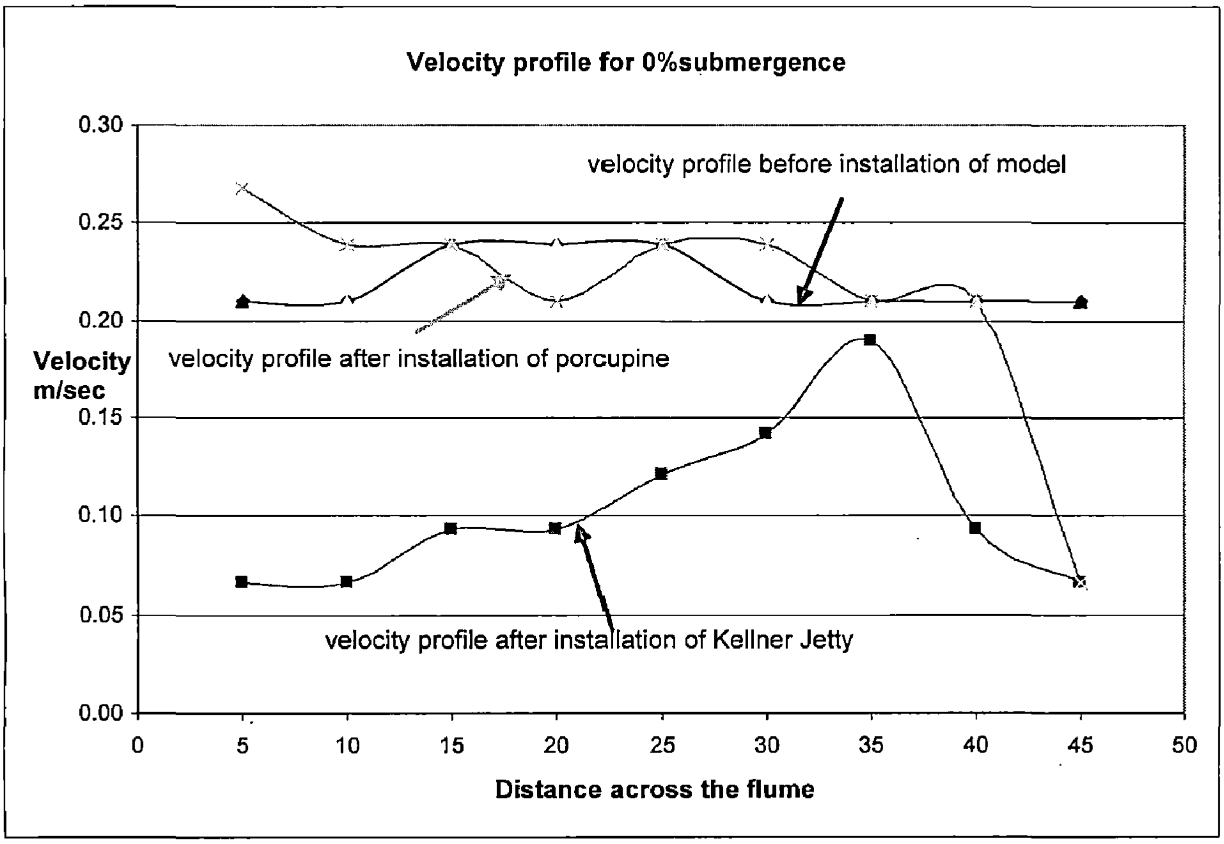


Fig 6.7 Velocity profile with 0% submergence at 0 of cross section

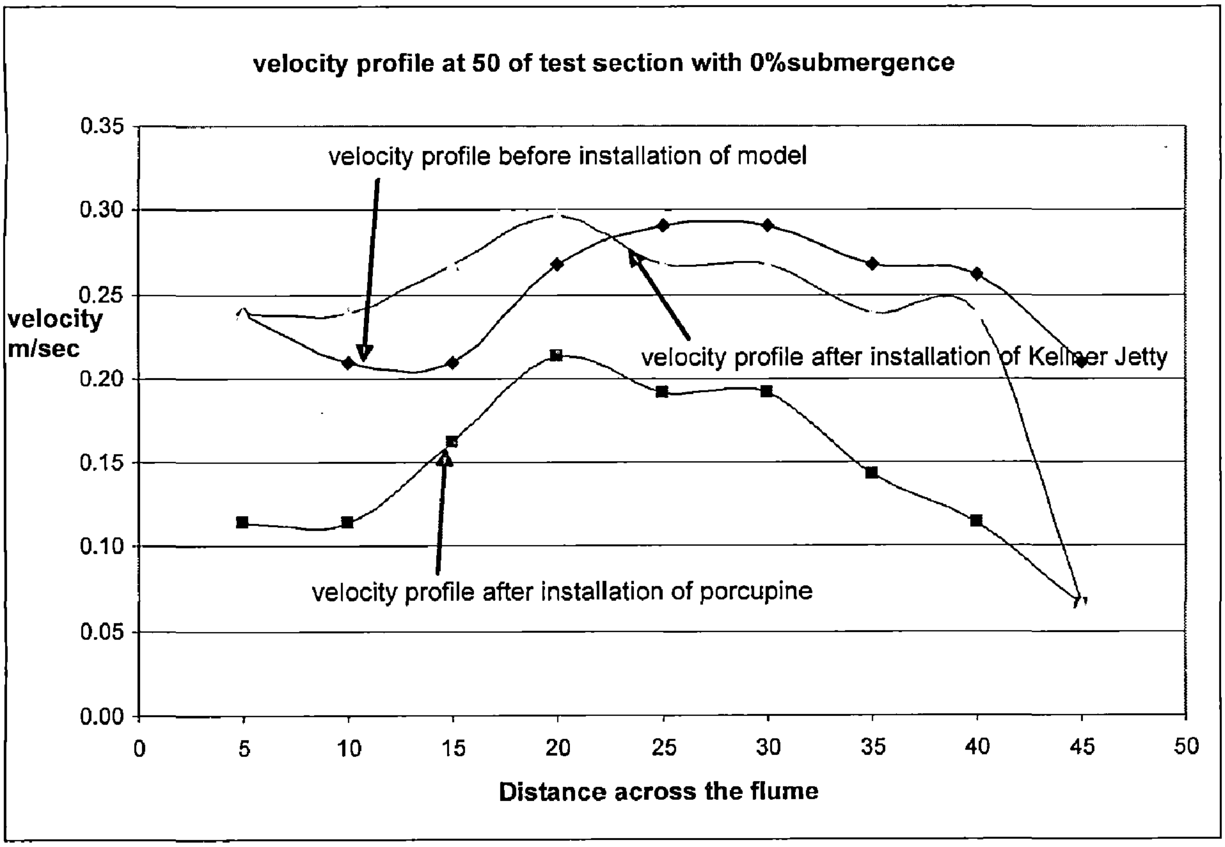


Fig 6.8 Velocity profile with 0% submergence at 50 of cross section

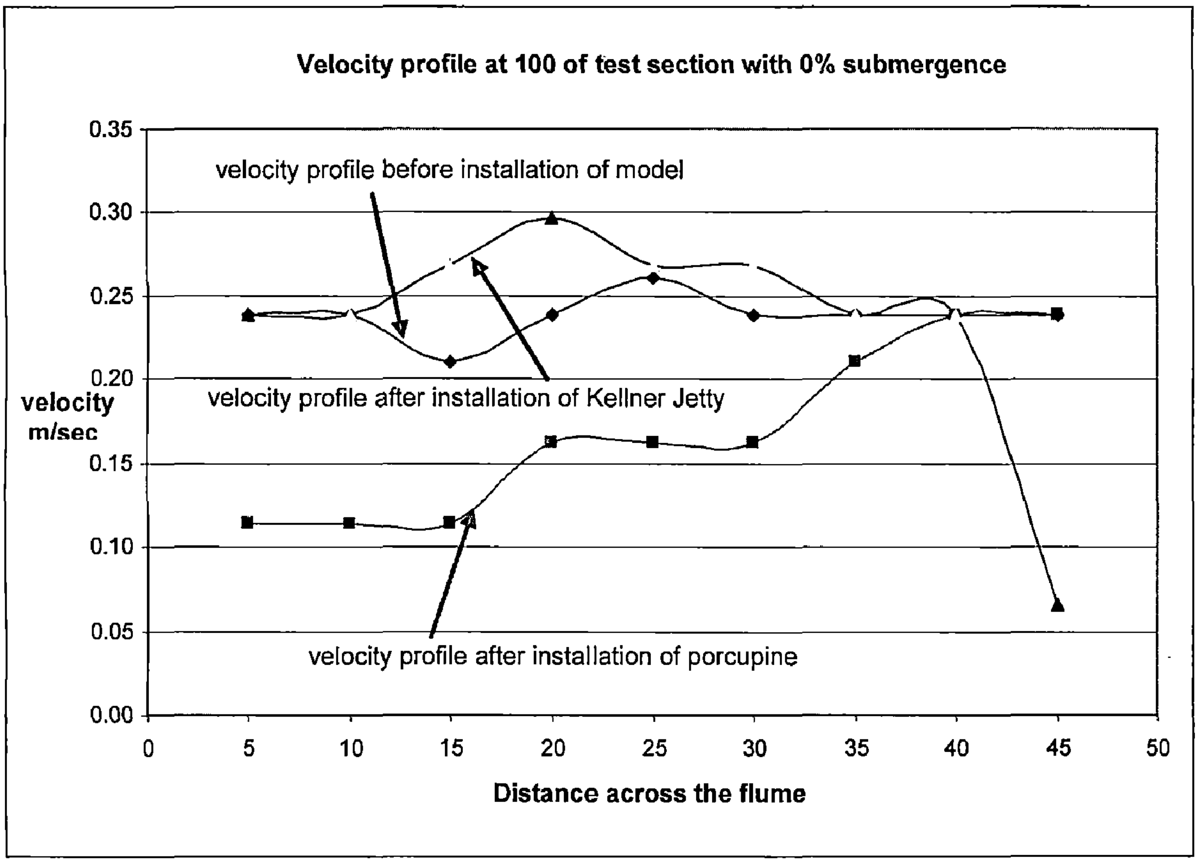


Fig 6.9 Velocity profile with 0% submergence at 100 of cross section

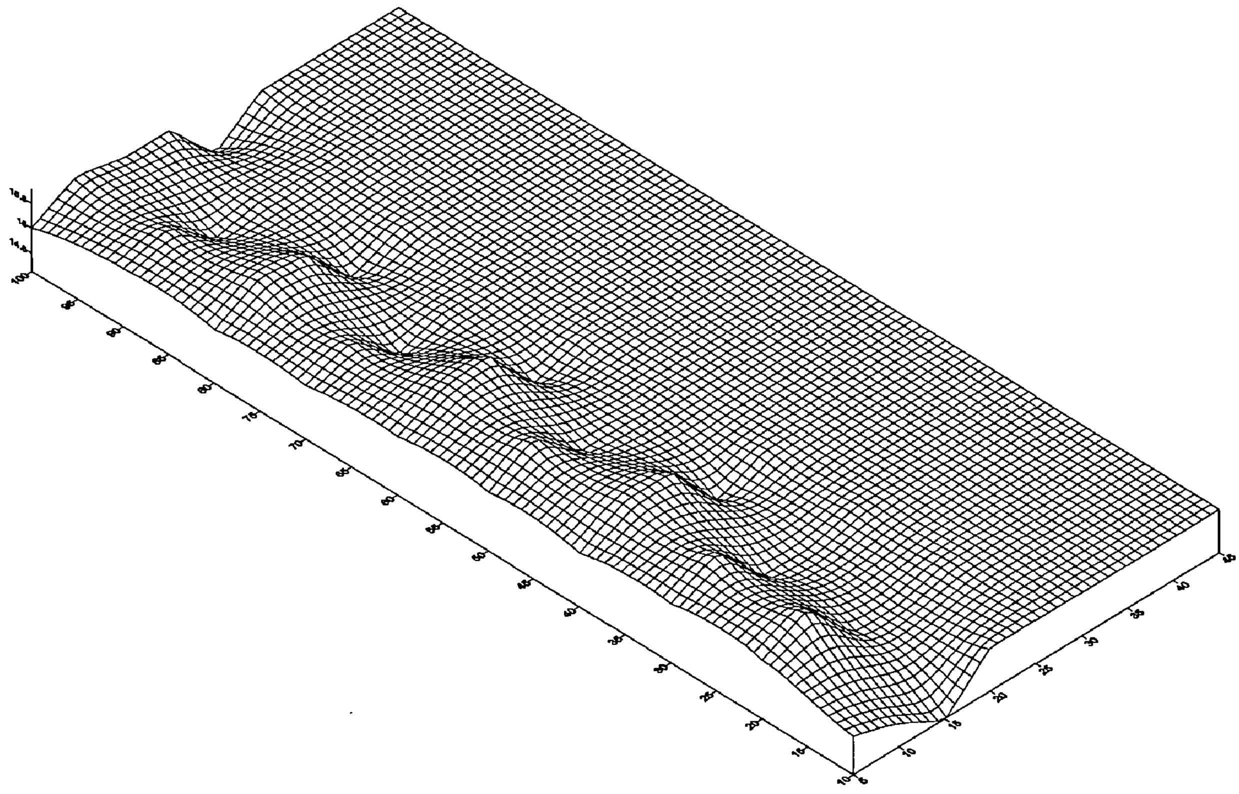


Fig 6.10 BedProfile after installation of Porcupine with 30% submergence

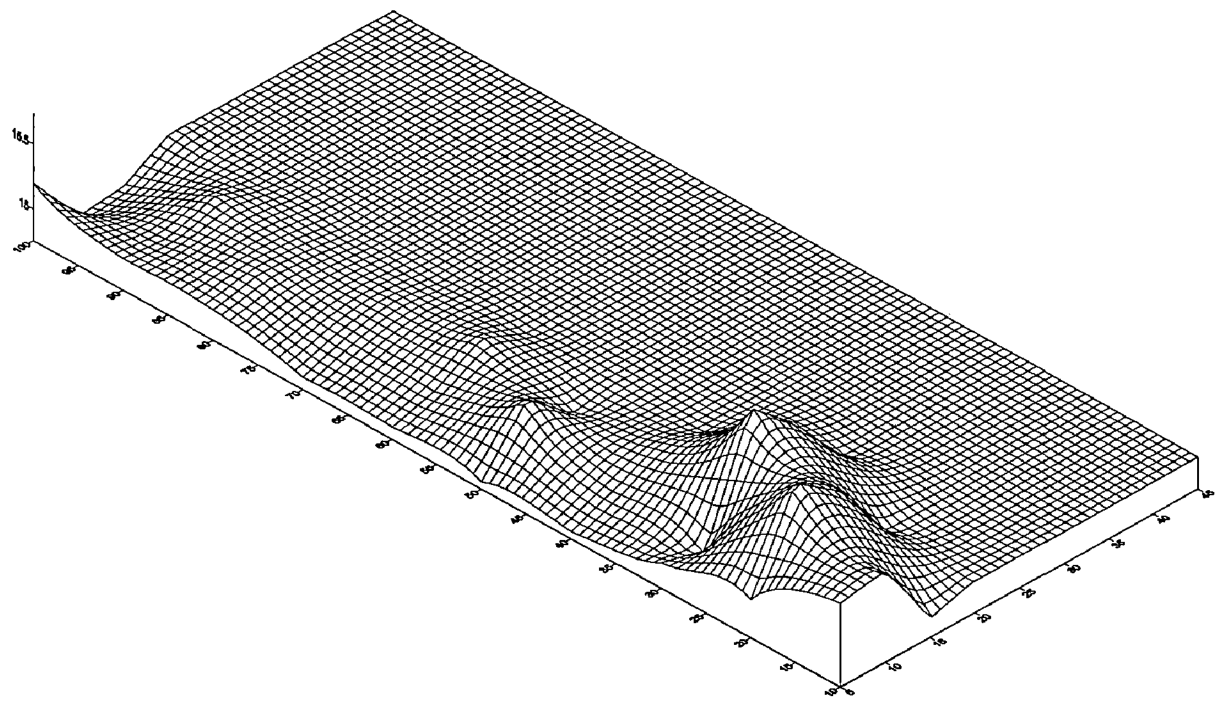


Fig 6.11 Bed Profile after installation of Kellner Jetty with 30% submergence

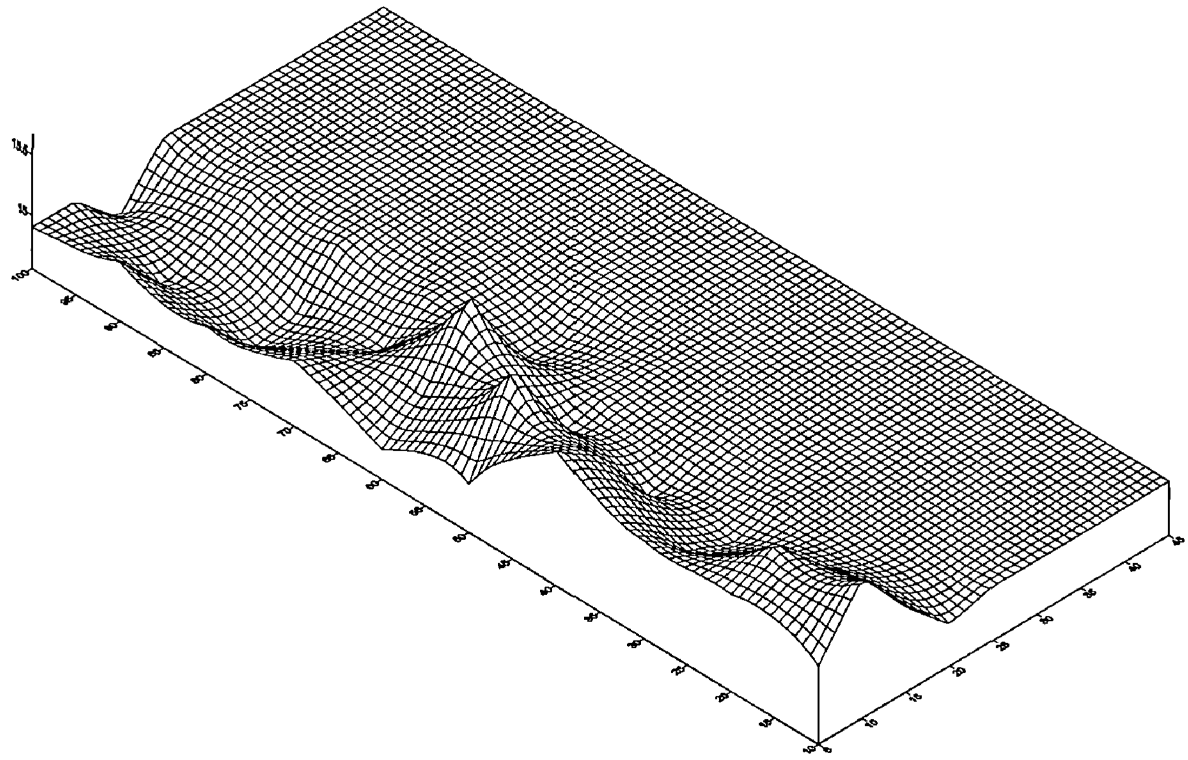


Fig 6.12 Bed Profile after installation of Porcupine with 50% submergence

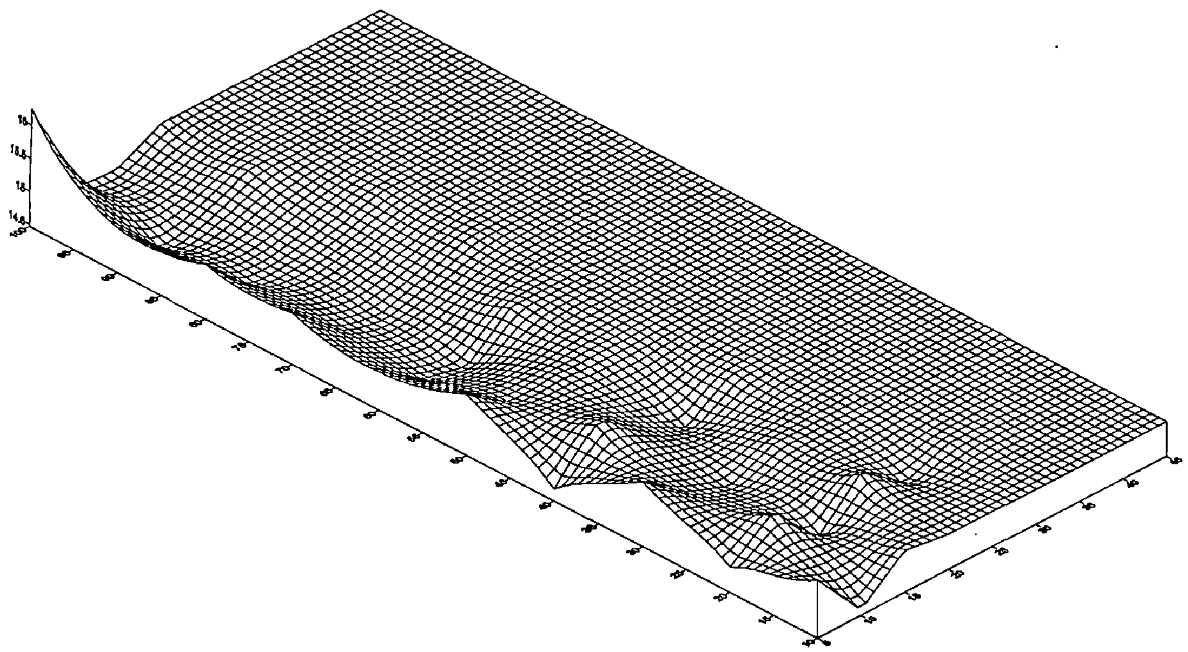


Fig 6.13 Bed Profile after installation of Kellner Jetty with 50% submergence

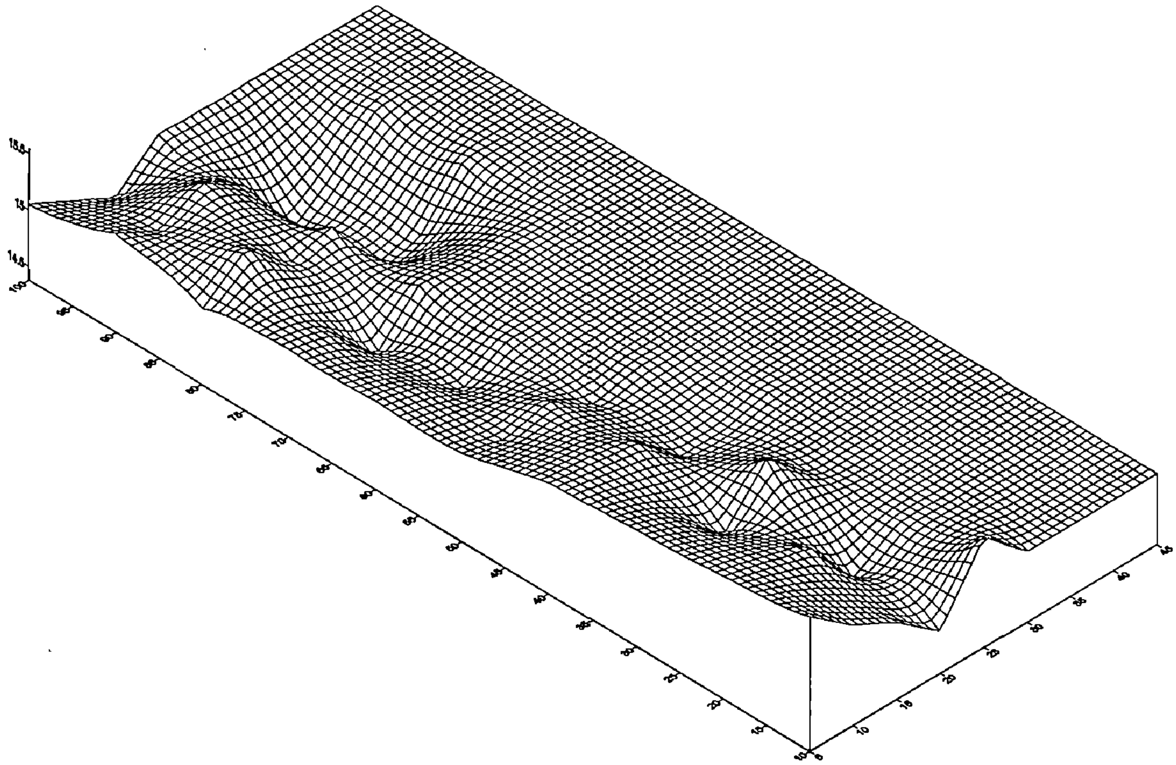
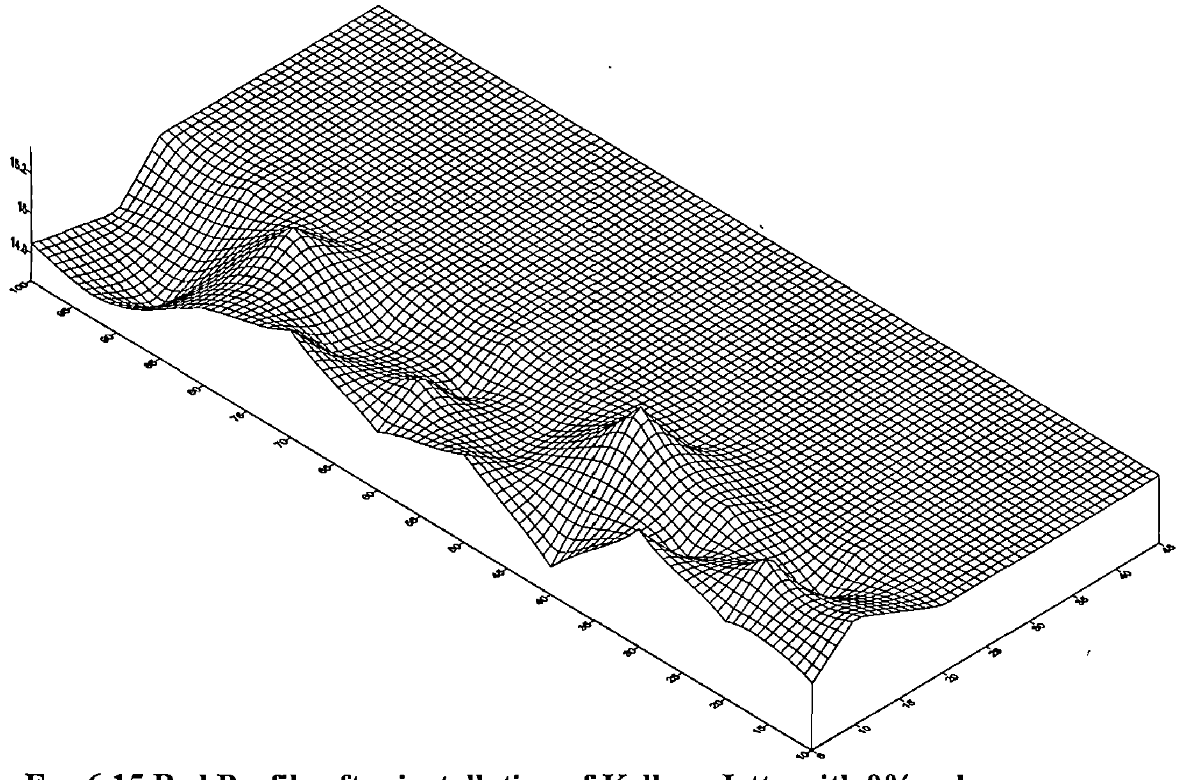
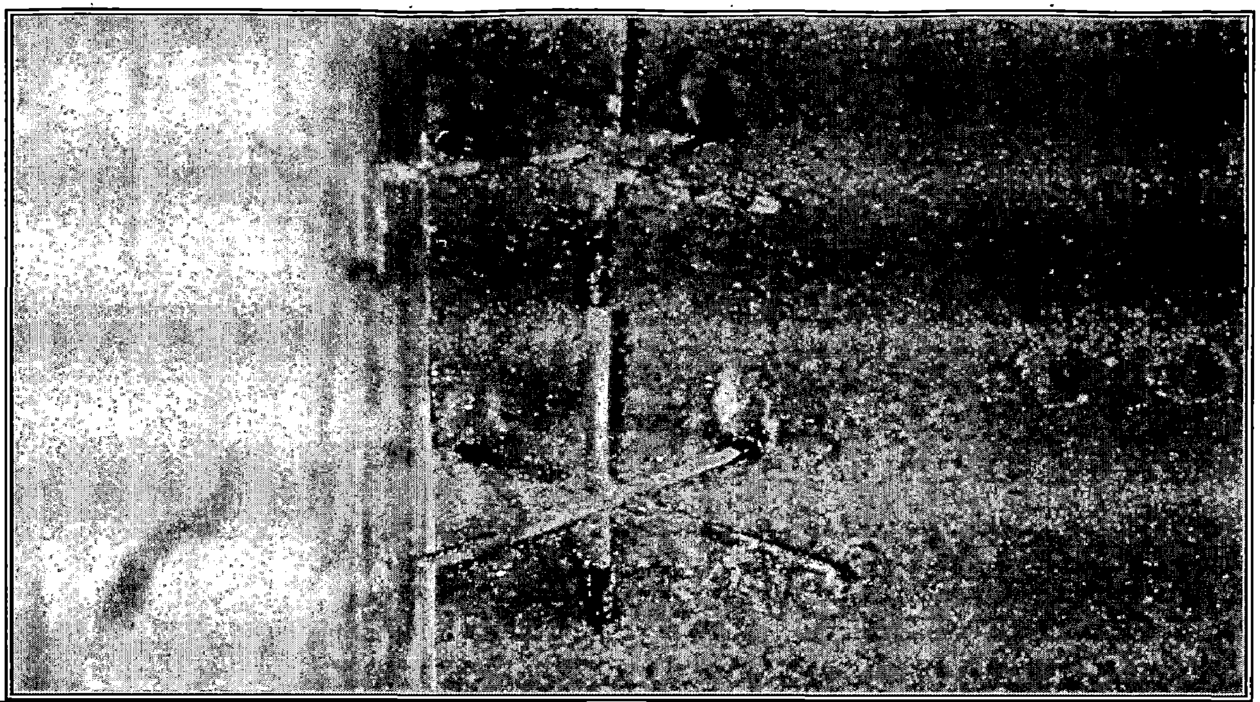
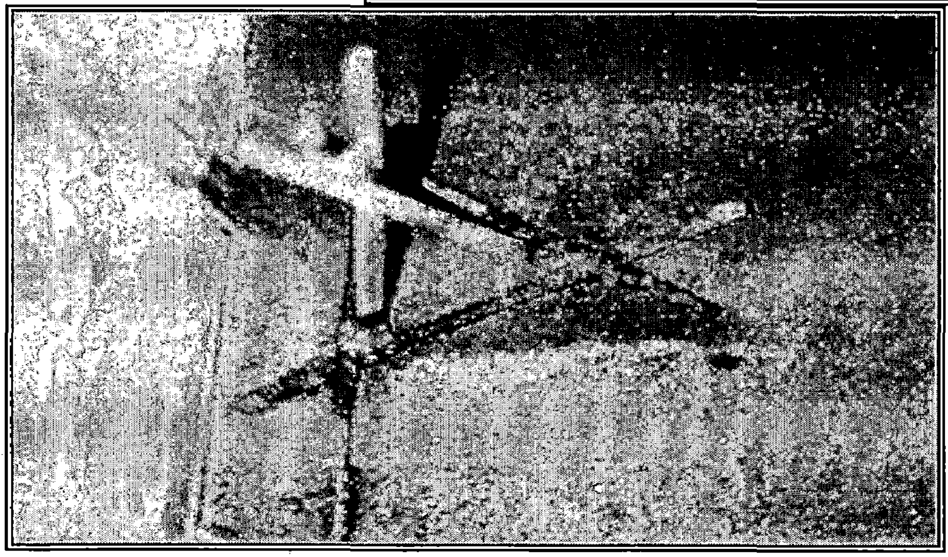


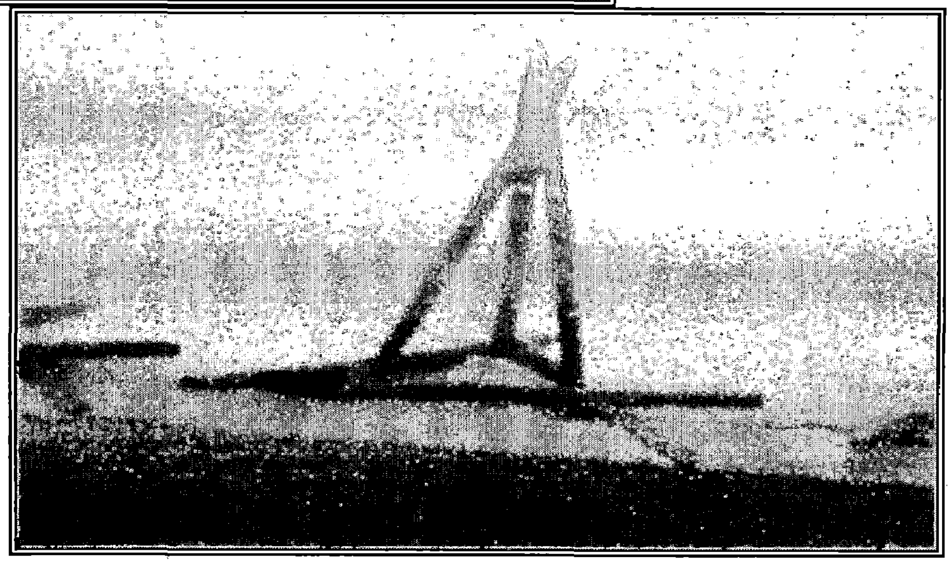
Fig 6.14 Bed Profile after installation of Porcupine with 0% submergence

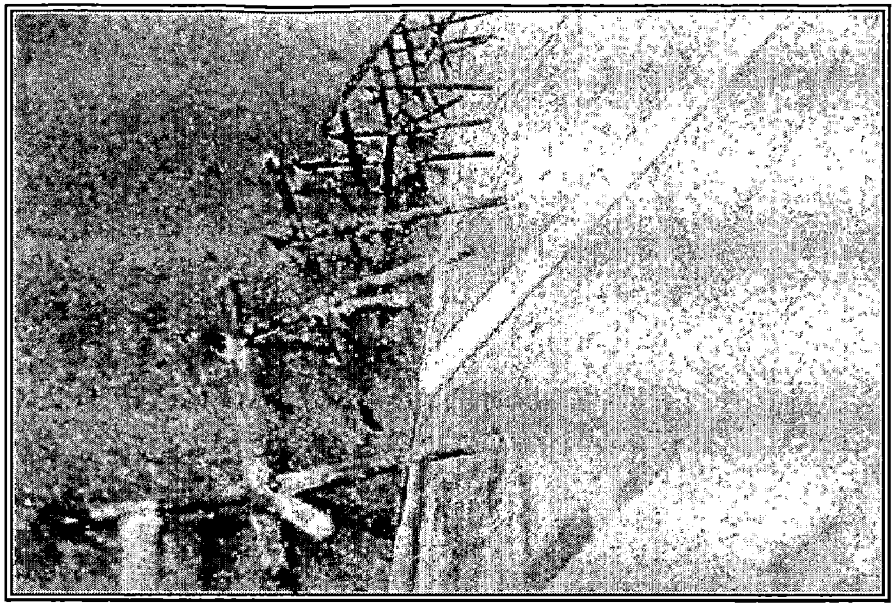
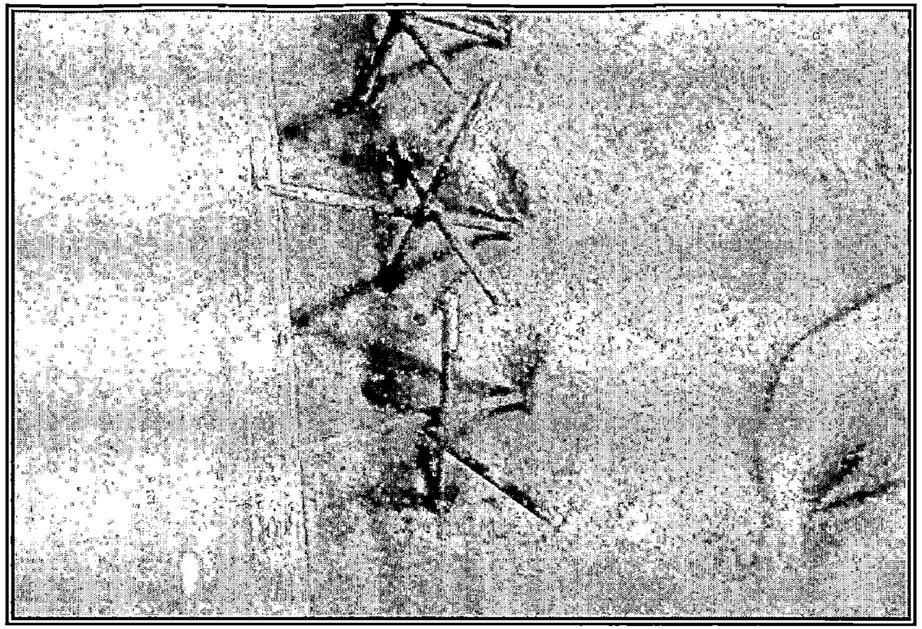
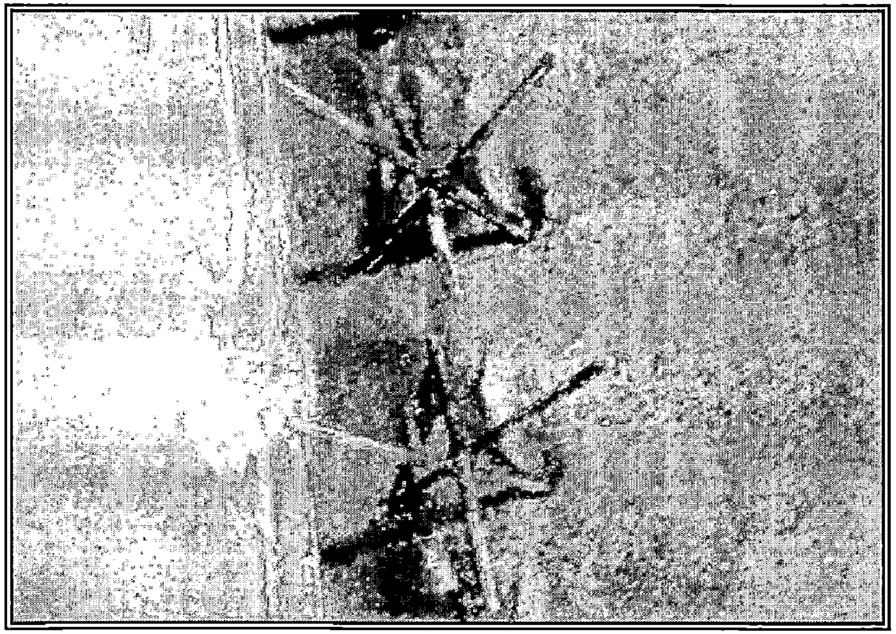
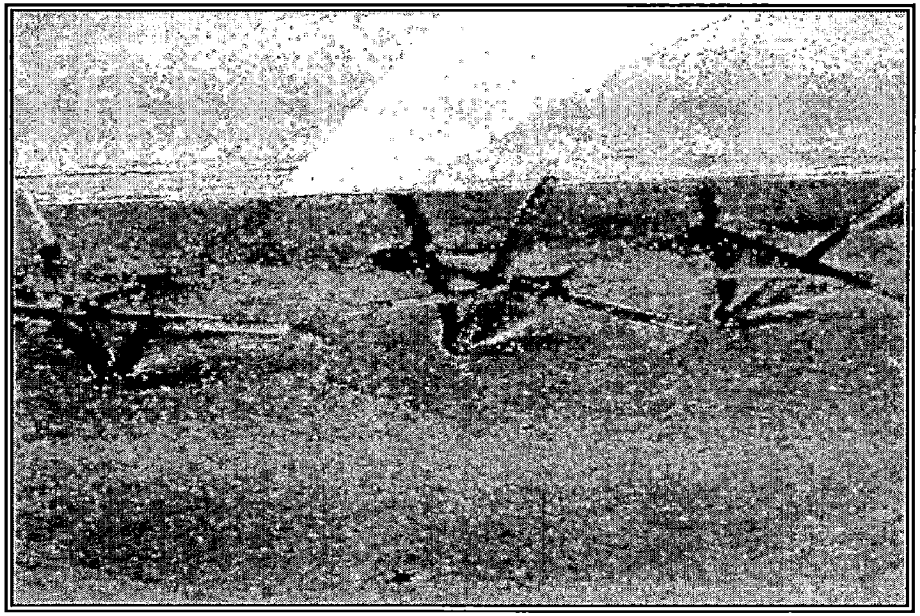


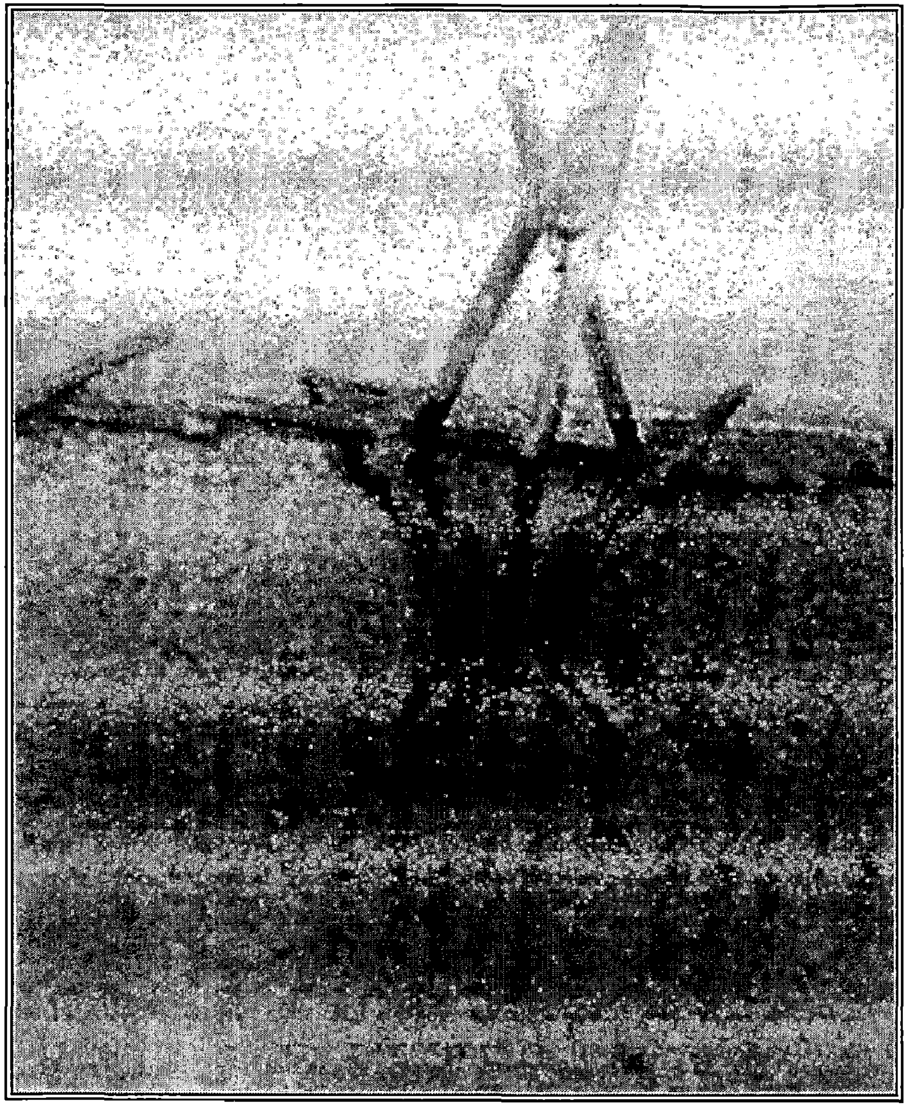
Fog 6.15 Bed Profile after installation of Kellner Jetty with 0% submergence

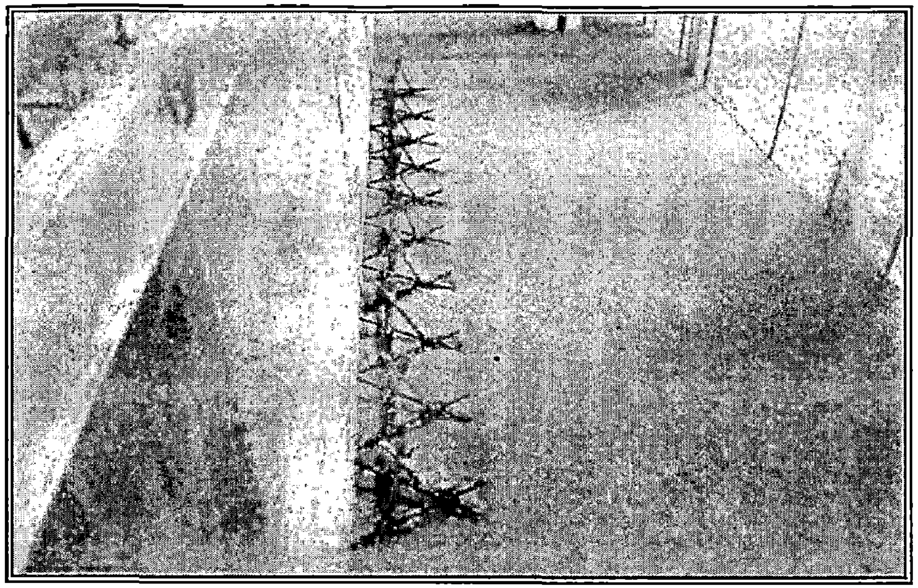
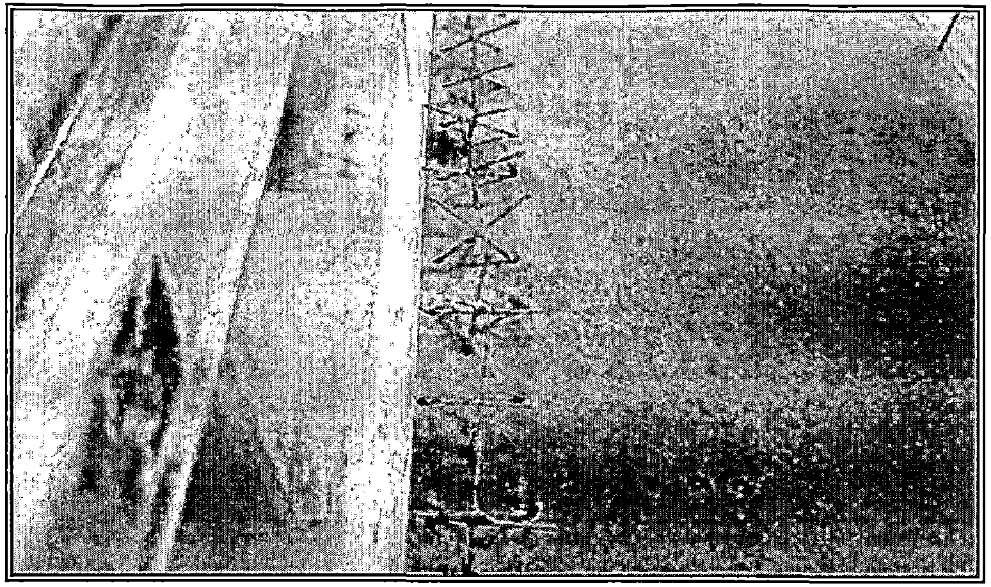
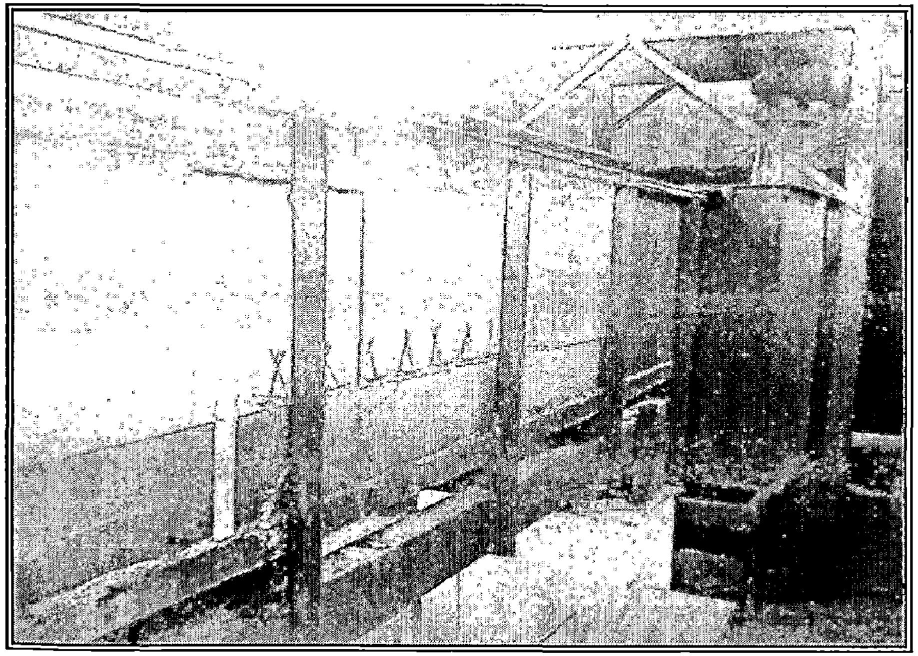
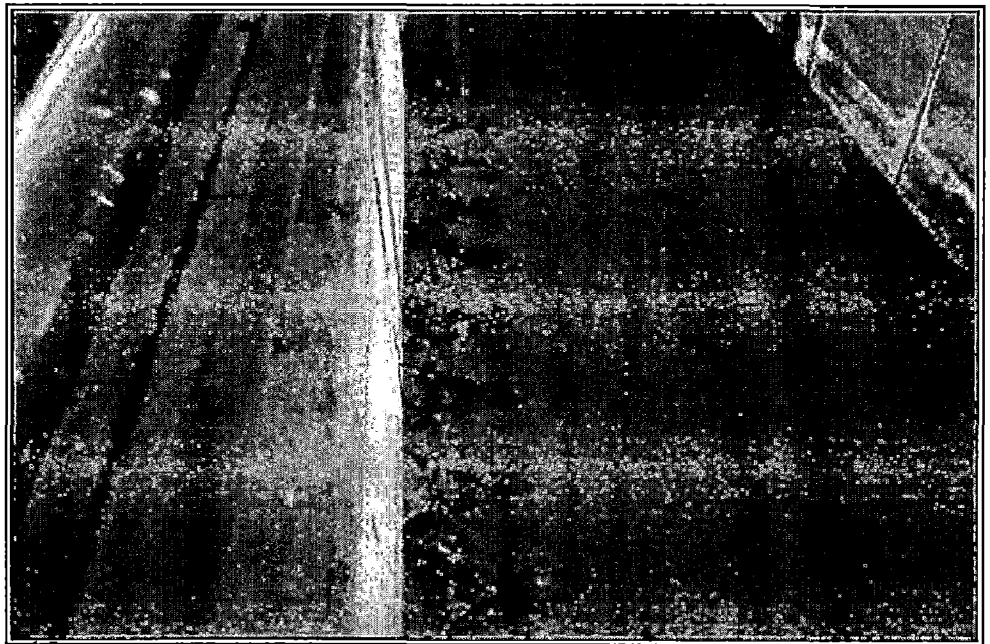


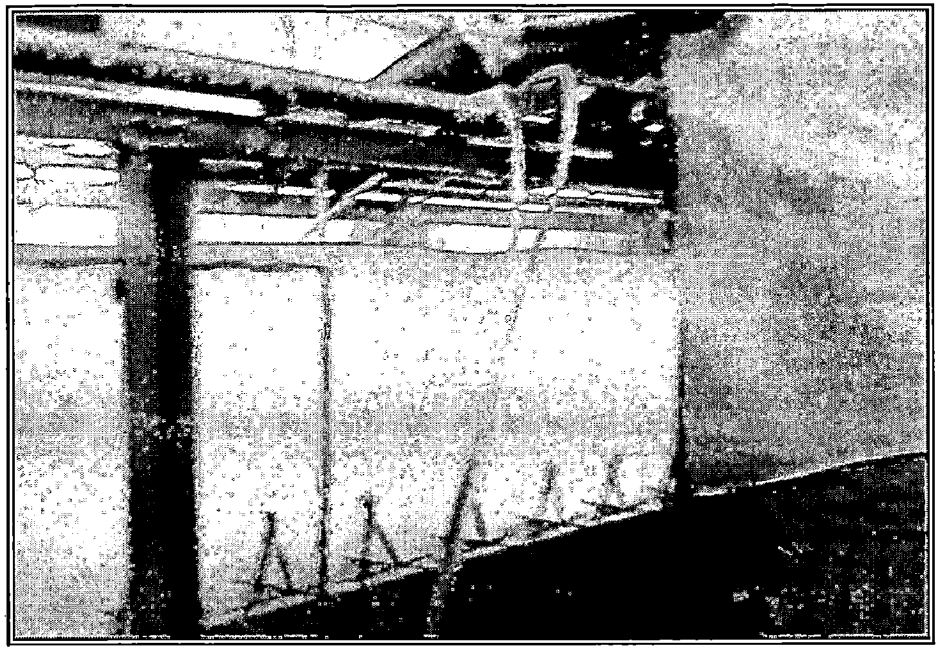


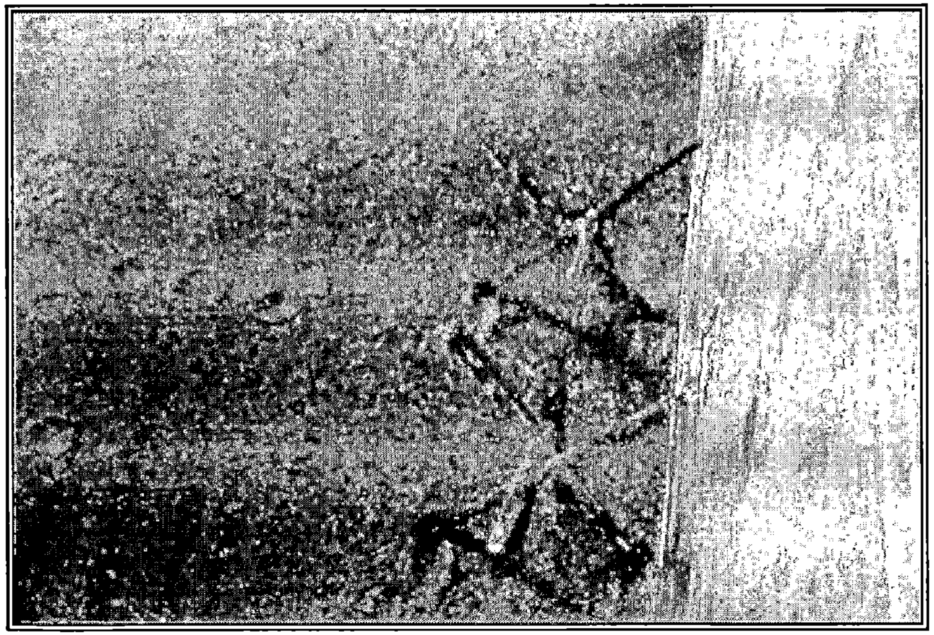








6.6 Results and Discussion

The details pertaining to experiments conducted is described to assess the findings of the present study. Details of the flume although not necessary is presented to highlight the fact which is conducted in a flume of smaller length are only trial runs to identify the effect of porcupine and Kellner jetty in the flow field. All the experiments has been conducted with clear water only and not with sediment laden water. Due to time constraint and some unavoidable circumstances the number of experiments has to be limited. The flume where all the experiments were run is a smaller one in which the turbulence was more which somehow affected the results.

Depth of water in the flume 30cm
Submergence = 30%

	Velocity before installation of porcupine	Velocity after installation of porcupine	%change	Average% change	Velocity after installation of Kellner Jetty	%change	Average% change
0 of test section							
5	0.36	0.40	**-11**	-4	0.14	**61**	53
10	0.33	0.37	**-13**		0.20	**40**	
15	0.33	0.31	**5**		0.14	**57**	
20	0.37	0.34	**7**		0.20	**46**	ve
25	0.37	0.40	**-7**	-2	0.14	**62**	32
30	0.34	0.31	**7**		0.24	**28**	
35	0.41	0.37	**9**		0.28	**31**	
40	0.38	0.42	**-10**		0.31	**18**	
45	0.36	0.40	**-11**		0.28	**22**	
50 of Test Section							
5	0.33	0.34	**-5**	-1	0.20	**39**	32
10	0.31	0.28	**11**		0.31	**0**	
15	0.37	0.37	**0**		0.24	**34**	
20	0.34	0.34	**0**		0.20	**42**	
25	0.36	0.40	**-11**	-11	0.20	**45**	37
30	0.35	0.40	**-12**		0.28	**21**	
35	0.34	0.40	**-16**		0.28	**18**	
40	0.36	0.37	**-3**		0.14	**61**	
45	0.32	0.37	**-16**		0.20	**38**	
100 of Test Section							
5	0.34	0.31	**8**	10	0.37	**-8**	20
10	0.34	0.34	**0**		0.24	**29**	
15	0.40	0.34	**13**		0.31	**21**	
20	0.35	0.31	**12**		0.24	**31**	
25	0.38	0.31	**18**	7	0.28	**27**	17
30	0.35	0.34	**3**		0.31	**12**	
35	0.34	0.28	**18**		**0.28**	**18**	
40	0.34	0.42	**-24**		0.37	**-10**	
45	0.38	0.31	**17**		0.24	**36**	

	Velocity before installation of porcupine	Velocity after installation of porcupine	%change	Average% change	Velocity after installation of Kellner Jetty	%change	Average% change

Depth of water in the flume 20cm
Submergence = 50%

0 of Test Section							
5	0.40	0.31	**21**	19	0.29	**27**	29
10	0.41	0.29	**29**		0.24	**42**	
15	0.37	0.33	**11**		0.24	**35**	
20	0.36	0.35	**4**		0.34	**6**	
25	0.47	0.33	**31**	28	0.29	**38**	36
30	0.38	0.31	**19**		0.29	**24**	
35	0.48	0.35	**26**		0.26	**46**	
40	0.50	0.33	**35**		0.29	**42**	
45	0.50	0.35	**29**		0.34	**31**	

50 of Test Section							
5	0.36	0.15	**59**	45	0.24	**34**	29
10	0.34	0.15	**57**		0.21	**39**	
15	0.38	0.31	**19**		0.33	**15**	
20	0.43	0.29	**33**		0.32	**26**	
25	0.47	0.21	**55**	61	0.34	**28**	29
30	0.46	0.24	**49**		0.36	**22**	
35	0.48	0.29	**39**		0.33	**32**	
40	0.54	0.09	**82**		0.31	**42**	
45	0.45	0.09	**79**		0.36	**21**	

100 of Test Section							
5	0.40	0.15	**63**	47	0.24	**40**	23
10	0.42	0.15	**64**		0.29	**31**	
15	0.45	0.29	**35**		0.28	**38**	
20	0.42	0.24	**43**		0.35	**15**	
25	0.34	0.24	**29**	49	0.36	**-7**	4
30	0.38	0.21	**44**		0.36	**4**	
35	0.36	0.15	**59**		0.33	**9**	
40	0.32	0.15	**54**		0.38	**-18**	
45	0.38	0.15	**61**		0.26	**31**	

	Velocity before installation of porcupine	Velocity after installation of porcupine	%change	Average% change	Velocity after installation of Kellner Jetty	%change	Average% change

Depth of water in the flume 10cm
Submergence = 0%

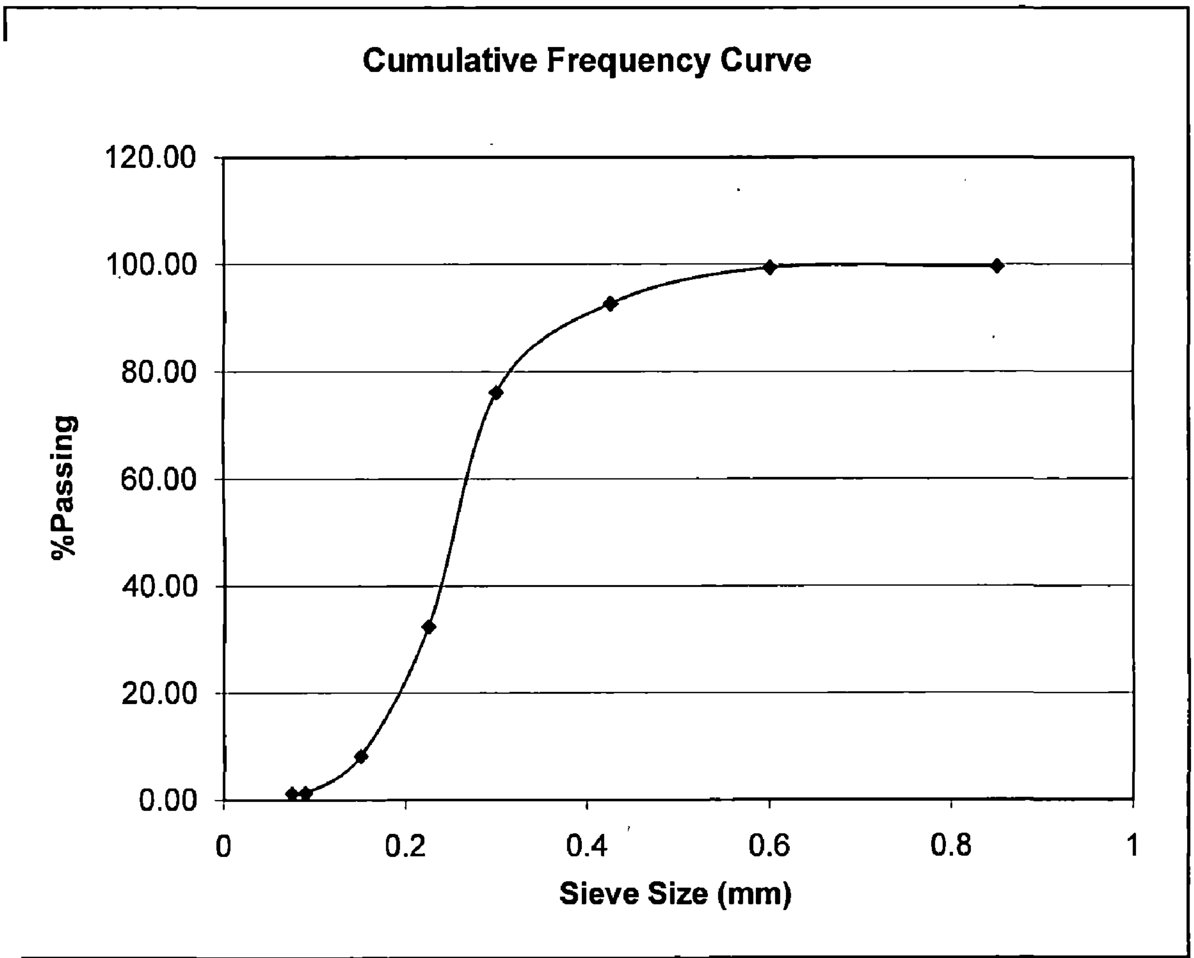
0 of test section							
5	0.21	0.07	**69**	62	0.09	**55**	48
10	0.21	0.07	**69**		0.08	**64**	
15	0.24	0.09	**61**		0.09	**61**	
20	0.24	0.09	**61**		0.15	**38**	
25	0.24	0.12	**49**	43	0.19	**21**	52
30	0.21	0.14	**33**		0.14	**32**	
35	0.21	0.19	**10**		0.07	**69**	
40	0.21	0.09	**55**		0.07	**69**	
45	0.21	0.07	**69**		0.07	**69**	

50 of Test Section							
5	0.24	0.11	**52**	35	0.07	**72**	41
10	0.21	0.11	**46**		0.07	**69**	
15	0.21	0.16	**23**		0.07	**69**	
20	0.27	0.21	**20**		0.30	**-11**	
25	0.29	0.19	**34**	48	0.27	**8**	43
30	0.29	0.19	**34**		0.27	**8**	
35	0.27	0.14	**47**		0.09	**65**	
40	0.26	0.11	**56**		0.09	**64**	
45	0.21	0.07	**69**		0.07	**69**	

100 of Test Section							
5	0.24	0.11	**52**	44	0.07	**72**	30
10	0.24	0.11	**52**		0.07	**72**	
15	0.21	0.11	**46**		0.14	**32**	
20	0.24	0.16	**32**		0.30	**-24**	
25	0.26	0.16	**38**	16	0.27	**-2**	41
30	0.24	0.16	**32**		0.20	**16**	
35	0.24	0.21	**12**		0.09	**60**	
40	0.24	0.24	**0**		0.09	**60**	
45	0.24	0.24	**0**		0.07	**72**	

\nalysis of the Sediment Materialof d_{50} = 0.25

Size mm	Weight Retained gm	Cum weight Retained gm	Cum % Retained	% Passing
.85	1.86	1.86	0.37	99.63
0.6	1.19	3.05	0.61	99.39
.425	33.52	36.57	7.32	92.69
0.3	82.69	119.26	23.86	76.14
0.225	218.86	338.13	67.63	32.37
0.15	121.17	459.29	91.87	8.13
0.09	34.39	493.69	98.75	1.25
0.075	0.52	494.21	98.85	1.15
pan	5.73	499.94	100.00	0.00
otal	499.94			



Chapter7

Observations, Conclusions and Suggestions for Further Study

7.1 Observations and Conclusions

1. In this preliminary phase of the study, a prima-facie analysis was conducted to get an insight into the basic causes of insufficient navigation draught in the six locations on the Ganga river falling between Varanasi & Farakka.
2. This initial preliminary study phase has brought out the occurrence of channel flow bifurcation and shoal formation behaviour due to heavy sedimentation at these sites as the root cause of navigation bottlenecks. Stream bank erosion and heavy sediment deposition due to inadequate transport capacity could be identified as major underlying causes leading to channel instability and resultant reduction in draught.
3. Basic prerequisite to development of reasonably durable navigation channel in the Ganga river can be cited as effective sediment management both in the stream as well as in the watershed areas and effectively control the predominant channel instability processes to achieve concentration of stream flow to generate desired draught. The problem of reduced draught in several locations of the Ganga river can be primarily attributed to channel instability phenomenon triggered by heavy sediment inflows and channel widening process due to erosion which invariably cause erratic drifting of the thalweg channel.
4. From the preliminary analysis of the satellite data of the Ganga river along with available information on LAD, discharge etc., it could be surmised that in the identified six locations the draught reduction problem is primarily an off-shoot of the stream flow division into multiple channels with resulting braided configuration. This river behaviour of stream channel bifurcation in the Ganga is actuated by bank erosion accompanied with random deposition of sediments in heaps during the receding stages of flood flows when sediment transport capacity in the stream sharply declines.

7.2 Suggestions for Further Study

1. For developing durable fairway in the Ganga river, field trial experiments of new cost-effective and hydraulically efficient river training structures are essentially require to be conducted at few selected sites to evolve their suitable design for their large scale application ultimately.
2. Due to aforesaid inherent uncertainties and complexities of the Ganga river behavior, it will be desirable as well as prudent to adopt step-by-step field experimental procedure based on incremental construction approach. This approach involves limited intervention in doses into the river environment in steps of field trials, wherein the response of the stream channel to the river training structures is, erratically analyzed for introducing further refinements / improvements for achieving desired results.

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OUTPUT TABLE OF HEC-RAS

Bateswarsthan
Profile output table

Qtotal	Min ch Elv	W.S. Elv	Critical W.S.	EG Elv	EG slope	Vel Ch	Flow area	Top Width	Froude No
880.00	15.01	17.33	17.33	17.97	0.008464	3.55	248.17	199.50	1.01
880.00	14.10	17.13		17.41	0.002421	2.35	374.81	218.68	0.57
880.00	14.01	16.45	16.45	17.17	0.008021	3.76	233.89	165.22	1.01
880.00	13.09	15.59	15.59	16.33	0.007743	3.83	229.88	154.09	1.00
880.00	12.59	15.41		15.96	0.004916	3.30	267.03	159.37	0.81
880.00	12.08	15.36		15.73	0.002723	2.70	326.08	168.56	0.2
880.00	11.58	15.34		15.59	0.001654	2.22	396.12	188.65	0.49
880.00	11.08	15.32		15.50	0.001471	1.90	463.53	255.97	0.45
880.00	10.57	15.29		15.43	0.000964	1.60	549.94	285.79	0.37
880.00	10.42	15.26		15.38	0.000854	1.52	579.55	297.56	0.35
880.00	9.15	15.27		15.34	0.000354	1.16	761.89	304.54	0.23
880.00	9.29	15.25		15.32	0.000417	1.16	757.00	338.54	0.25
880.00	7.12	15.26		15.30	0.000158	0.85	1033.52	356.24	0.16
880.00	5.73	15.26		15.29	0.000084	0.72	1216.03	331.11	0.12
880.00	11.01	14.70	14.70	15.23	0.009124	3.20	274.81	271.97	1.02

Cross section output
Plan: Rbateshwarsthan Ganga Bateshwarsthan RS: 683.717

Profile: 10Apr1986 00:00

E.G. Elev (m) 17.97	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (m) 0.64	Wt. n-Val.		0.030	
W.S. Elev (m)17.33	Reach Len. (m)	48.54	48.54	48.54
Crit W.S. (m) 17.33	Flow Area (m2)		248.17	
E.G. Slope (m/m)0.008464	Area (m2)		248.17	
Q Total (m3/s)880.00	Flow (m3/s)		880.00	
Top Width (m)199.50	Top Width (m)		199.50	
Vel Total (m/s)3.55	Avg. Vel. (m/s)		3.55	
Max Chl Dpth (m) 2.31	Hydr. Depth (m)		1.24	
Conv. Total (m3/s)9565.4	Conv. (m3/s)		9565.4	
Length Wtd. (m)48.54	Wetted Per. (m)		199.59	

Min Ch El (m)15.01	Shear (N/m2)		103.20
Alpha 1.00	Stream Power (N/m s)		365.95
Frctn Loss (m 0.20	Cum Volume (1000 m3)		360.25
C & E Loss (m) 0.11	Cum SA (1000 m2)		171.41

Punarak

Profile output table

Qtotal	Min ch Elv	W.S. Elv	Critical W.S.	EG Elv	EG slope	Vel Ch	Flow area	Top Width	Froude No
2652.00	36.10	36.92		37.11	0.005627	1.96	1353.41	1950.78	0.75
2652.00	35.69	36.65		36.84	0.005400	1.92	1378.09	1978.89	0.74
2652.00	35.30	36.44		36.60	0.004020	1.75	1513.96	2006.18	0.64
2652.00	31.30	36.24		36.39	0.003946	1.72	1541.02	2067.45	0.64
2652.00	29.91	36.18		36.26	0.001471	1.27	2080.87	2089.45	0.41
2652.00	29.70	36.05		36.16	0.002526	1.49	1775.55	2108.37	0.52
2652.00	29.86	36.06	34.93	36.10	0.000406	0.86	3077.51	2115.60	0.23
2652.00	29.55	34.26	34.26	35.89	0.006058	5.66	468.80	144.93	1.00
2652.00	23.71	34.74		34.74	0.000001	0.12	21838.76	2160.25	0.01
2652.00	28.82	33.23	33.16	34.60	0.005850	5.19	511.40	175.52	0.97
2652.00	27.14	34.19		34.19	0.000004	0.20	12948.97	2200.87	0.03
2652.00	27.26	34.15		34.19	0.000135	0.83	3182.95	1005.89	0.15
2652.00	27.47	34.17		34.18	0.000015	0.31	8480.80	2205.12	0.05
2652.00	27.96	34.18		34.18	0.000006	0.24	11197.47	2217.84	0.03
2652.00	28.13	34.18		34.18	0.000004	0.22	12302.59	2246.32	0.03
2652.00	27.91	34.18		34.18	0.000004	0.21	12851.47	2272.72	0.03
2652.00	27.69	34.17		34.18	0.000003	0.20	13323.87	2303.65	0.03
2652.00	27.52	34.17		34.18	0.000003	0.19	13781.57	2328.55	0.03
2652.00	27.39	34.17		34.18	0.000003	0.19	14249.29	2346.52	0.02
2652.00	27.30	34.17		34.18	0.000003	0.18	14710.75	2364.31	0.02
2652.00	27.23	34.17	28.10	34.18	0.000002	0.17	5187.20	2381.98	0.02

Cross section output

Plan: 160508 Ganga Punarak RS: 880 Profile: 01Jan2004 00:00

E.G. Elev (m) 37.11	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (m) 0.20	Wt. n-Val.		0.030	
W.S. Elev (m)36.92	Reach Len. (m)	50.00	50.00	50.00
Crit W.S. (m)	Flow Area (m2)		1353.41	
E.G. Slope (m/m)0.005627	Area (m2)		1353.41	
Q Total (m3/s) 2652.00	Flow (m3/s)		2652.00	
Top Width (m) 1950.78	Top Width (m)		1950.78	
Vel Total (m/s) 1.96	Avg. Vel. (m/s)		1.96	
Max Chl Dpth (m) 0.82	Hydr. Depth (m)		0.69	

Conv. Total (m3/s)	35355.2	Conv. (m3/s)	35355.2
Length Wtd. (m)	50.00	Wetted Per. (m)	1950.80
Min Ch El (m)	36.10	Shear (N/m2)	38.28
Alpha	1.00	Stream Power (N/m s)	75.01
Frctn Loss (m)	0.28	Cum Volume (1000 m3)	7974.30
C & E Loss (m)	0.00	Cum SA (1000 m2)	1925.24

Arjunpur

Profile output table

Qtotal	Min ch Elv	W.S. Elv	Critical W.S.	EG Elv	EG slope	Vel Ch	Flow area	Top Width	Froude No
928.00	45.22	46.88		47.06	0.003956	1.88	492.61	578.30	0.65
928.00	44.51	46.54		46.60	0.000941	1.11	836.16	739.25	0.33
928.00	44.86	46.15		46.25	0.002236	1.40	664.15	795.64	0.49
928.00	44.05	45.92		45.96	0.000648	0.94	987.39	847.22	0.28
928.00	44.26	45.62		45.70	0.001859	1.25	744.22	920.93	0.44
928.00	43.39	45.61		45.64	0.000403	0.82	1129.83	830.98	0.22
928.00	42.90	45.48		45.58	0.002043	1.40	662.26	738.27	0.47
928.00	42.66	45.49		45.51	0.000226	0.66	1398.92	917.21	0.17
928.00	44.23	45.33		45.44	0.003463	1.48	629.14	964.87	0.58
928.00	41.96	45.40		45.40	0.000027	0.34	2724.48	976.60	0.07
928.00	44.28	45.15		45.37	0.007462	2.09	443.86	717.34	0.85
928.00	41.30	45.30		45.30	0.000021	0.33	2814.93	876.05	0.06
928.00	42.87	45.02	45.02	45.27	0.013504	2.19	424.47	1001.02	1.07
928.00	38.49	43.02		43.03	0.000016	0.30	3110.52	917.96	0.05
928.00	41.70	42.78	42.78	43.00	0.011912	2.06	450.18	1055.49	1.01
928.00	38.51	42.64		42.64	0.000013	0.26	3600.40	1177.43	0.05
928.00	41.18	42.39	42.39	42.61	0.011920	2.08	446.81	1036.39	1.01
928.00	38.47	41.88		41.90	0.000094	0.58	1610.40	676.83	0.12
928.00	40.20	41.40	41.40	41.80	0.009746	2.79	333.17	427.81	1.01
928.00	38.47	41.42		41.45	0.000221	0.80	1166.13	573.71	0.18
928.00	39.43	41.16		41.34	0.002930	1.91	484.79	443.56	0.58
928.00	38.47	41.07		41.13	0.000527	1.09	851.16	500.57	0.27
928.00	39.06	40.68		40.92	0.003504	2.18	425.86	366.90	0.65
928.00	38.46	40.32		40.49	0.001710	1.82	510.95	337.79	0.47
928.00	36.49	40.37		40.40	0.000115	0.72	1289.10	449.50	0.14
928.00	38.84	40.27		40.35	0.001481	1.29	719.75	714.19	0.41
928.00	37.29	40.26		40.28	0.000101	0.56	1649.12	758.43	0.12
928.00	38.32	40.09		40.21	0.002013	1.56	596.07	561.26	0.48
928.00	36.64	40.13		40.14	0.000071	0.50	1856.14	777.41	0.10
928.00	38.16	40.00		40.09	0.001767	1.38	673.32	690.22	0.45
928.00	36.95	40.00		40.02	0.000102	0.52	1774.33	913.25	0.12
928.00	37.07	39.95		39.99	0.000294	0.82	1125.97	650.08	0.20
928.00	36.69	39.35	39.35	39.80	0.009446	2.99	310.56	350.51	1.01
928.00	35.51	37.15		37.33	0.001977	1.87	495.15	348.13	0.50
928.00	35.02	36.93	36.10	37.06	0.001208	1.61	577.43	353.38	0.40
928.00	35.02	36.11	36.11	36.59	0.009056	3.09	299.89	311.20	1.01
928.00	45.21		39.66					0.00	

Cross section output

Plan: 150508 Ganga Arjunpur RS: 1121 Profile: 01Jun2003 00:00

E.G. Elev (m) 47.06	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (m) 0.18	Wt. n-Val.		0.030	
W.S. Elev (m)46.88	Reach Len. (m)	250.00	250.00	250.00
Crit W.S. (m)	Flow Area (m2)		492.61	
E.G. Slope (m/m) 0.003956	Area (m2)		492.61	
Q Total (m3/s) 928.00	Flow (m3/s)		928.00	
Top Width (m) 578.30	Top Width (m)		578.30	
Vel Total (m/s) 1.88	Avg. Vel. (m/s)		1.88	
Max Chl Dpth (m) 1.66	Hydr. Depth (m)		0.85	
Conv. Total (m3/s) 14755.0	Conv. (m3/s)		14755.0	
Length Wtd. (m) 250.00	Wetted Per. (m)		578.31	
Min Ch El (m) 45.22	Shear (N/m2)		33.04	
Alpha 1.00	Stream Power (N/m s)		62.25	
Frctn Loss (m) 0.43	Cum Volume (1000 m3)		6156.88	
C & E Loss (m) 0.04	Cum SA (1000 m2)		4105.73	

Debchandpur

Profile output table

Qtotal	**Min ch Elv**	**W.S. Elv**	**Critical W.S.**	**EG Elv**	**EG slope**	**Vel Ch**	**Flow area**	**Top Width**	**Froude No**
576.00	54.06	54.98		55.11	0.004988	1.61	357.97	633.51	0.68
576.00	52.68	54.89		54.90	0.000113	0.46	1260.64	859.57	0.12
576.00	50.78	54.89		54.89	0.000010	0.21	2707.52	953.68	0.04
576.00	51.01	54.88		54.88	0.000014	0.24	2410.00	917.71	0.05
576.00	51.28	54.87		54.87	0.000022	0.28	2080.16	893.09	0.06
576.00	51.55	54.86		54.86	0.000036	0.33	1755.94	842.87	0.07
576.00	51.82	54.83		54.84	0.000068	0.40	1432.01	811.57	0.10
576.00	52.08	54.77		54.79	0.000157	0.52	1098.00	779.79	0.14
576.00	52.30	54.62		54.65	0.000552	0.78	736.36	737.70	0.25
576.00	52.04	54.30		54.36	0.000613	1.02	563.46	408.48	0.28
576.00	51.90	54.12		54.16	0.000707	0.91	635.22	613.51	0.28
576.00	51.89	53.85		53.91	0.000969	1.10	523.02	478.30	0.34
576.00	51.60	53.60		53.64	0.000779	0.98	586.55	540.81	0.30
576.00	50.91	53.38		53.46	0.001059	1.25	459.95	370.73	0.36
576.00	50.35	53.20		53.27	0.000821	1.15	500.52	378.19	0.32
576.00	50.02	52.99		53.07	0.001179	1.27	455.31	391.67	0.37
576.00	49.38	52.83		52.89	0.000723	1.05	547.68	430.72	0.30

576.00	48.90	52.55		52.69	0.001315	1.66	347.92	215.68	0.42
576.00	48.13	52.40		52.52	0.000622	1.54	374.84	148.60	0.31
576.00	48.27	52.22		52.38	0.000920	1.78	324.28	138.97	0.37
576.00	48.19	52.04		52.21	0.000956	1.79	321.13	139.60	0.38
576.00	48.47	51.96		52.05	0.000597	1.28	449.30	227.17	0.29
576.00	49.26	51.71		51.87	0.001746	1.72	334.47	243.05	0.47
576.00	49.08	51.60		51.66	0.000611	1.07	538.36	363.07	0.28
576.00	49.39	51.30		51.43	0.002607	1.58	365.19	409.13	0.53
576.00	48.82	51.15		51.19	0.000585	0.91	631.43	524.39	0.27
576.00	48.83	50.85		50.99	0.001936	1.66	347.39	288.91	0.48
576.00	47.82	50.35	49.57	50.49	0.001430	1.66	346.66	228.94	0.43
576.00	47.25	49.02	49.02	49.58	0.008586	3.32	173.62	155.81	1.00
576.00	54.06		53.79					0.00	

Cross section output

Plan: 150508 Ganga Debchandpur RS: 1253 Profile: 01Jan2004 00:00

E.G. Elev (m) 55.11	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (m) 0.13	Wt. n-Val.		0.030	
W.S. Elev (m) 54.98	Reach Len. (m)	500.00	500.00	500.00
Crit W.S. (m)	Flow Area (m2)		357.97	
E.G. Slope (m/m) 0.004988	Area (m2)		357.97	
Q Total (m3/s)576.00	Flow (m3/s)		576.00	
Top Width (m)633.51	Top Width (m)		633.51	
Vel Total (m/s)1.61	Avg. Vel. (m/s)		1.61	
Max Chl Dpth (m) 0.91	Hydr. Depth (m)		0.57	
Conv. Total (m3/s) 8155.6	Conv. (m3/s)		8155.6	
Length Wtd. (m) 500.00	Wetted Per. (m)		633.51	
Min Ch El (m) 54.06	Shear (N/m2)		27.64	
Alpha 1.00	Stream Power (N/m s)		44.47	
Frctn Loss (m) 0.17	Cum Volume (1000 m3)		8884.85	
C & E Loss (m) 0.04	Cum SA (1000 m2		5146.57	

Nakhwa

Profile output table

Qtotal	Min ch Elv	W.S. Elv	Critical W.S.	EG Elv	EG slope	Vel Ch	Flow area	Top Width	Froude No
378.00	55.42	59.16		59.16	0.000013	0.19	1971.13	1000.00	0.04
378.00	54.99	59.16		59.16	0.000007	0.16	2410.47	1010.77	0.03
378.00	54.97	59.16		59.16	0.000007	0.15	2442.77	1021.54	0.03
378.00	54.96	59.16		59.16	0.000007	0.15	2475.57	1032.31	0.03
378.00	54.94	59.16		59.16	0.000006	0.15	2508.24	1043.08	0.03

378.00	54.92	59.16		59.16	0.000006	0.15	2541.34	1053.85	0.03
378.00	54.91	59.16		59.16	0.000006	0.15	2574.33	1064.62	0.03
378.00	54.89	59.16		59.16	0.000006	0.14	2607.77	1075.39	0.03
378.00	54.88	59.16		59.16	0.000006	0.14	2640.96	1086.15	0.03
378.00	54.86	59.16		59.16	0.000005	0.14	2674.49	1096.92	0.03
378.00	54.85	59.16		59.16	0.000005	0.14	2708.32	1107.69	0.03
378.00	54.83	59.16		59.16	0.000005	0.14	2742.24	1118.46	0.03
378.00	54.82	59.16		59.16	0.000005	0.14	2776.45	1129.23	0.03
378.00	54.80	59.16		59.16	0.000005	0.13	2810.55	1140.00	0.03
378.00	54.76	59.16		59.16	0.000005	0.13	2874.30	1147.69	0.03
378.00	54.72	59.16		59.16	0.000004	0.13	2939.45	1155.39	0.03
378.00	54.69	59.15		59.16	0.000004	0.13	3005.25	1163.08	0.02
378.00	54.65	59.15		59.16	0.000004	0.12	3071.91	1170.77	0.02
378.00	54.61	59.15		59.15	0.000004	0.12	3139.41	1178.46	0.02
378.00	54.57	59.15		59.15	0.000003	0.12	3208.13	1186.15	0.02
378.00	54.53	59.15		59.15	0.000003	0.12	3277.34	1193.85	0.02
378.00	54.49	59.15		59.15	0.000003	0.11	3347.78	1201.54	0.02
378.00	54.45	59.15		59.15	0.000003	0.11	3418.62	1209.23	0.02
378.00	54.42	59.15		59.15	0.000003	0.11	3490.97	1216.92	0.02
378.00	54.38	59.15		59.15	0.000002	0.11	3563.80	1224.62	0.02
378.00	54.34	59.15		59.15	0.000002	0.10	3637.66	1232.31	0.02
378.00	54.30	59.15		59.15	0.000002	0.10	3712.23	1240.00	0.02
378.00	54.14	59.15		59.15	0.000002	0.10	3779.51	1257.50	0.02
378.00	53.98	59.15		59.15	0.000002	0.10	3877.20	1275.00	0.02
378.00	53.81	59.15		59.15	0.000002	0.09	4005.03	1292.50	0.02
378.00	53.65	59.15		59.15	0.000002	0.09	4163.42	1310.00	0.02
378.00	53.49	59.15		59.15	0.000001	0.09	4351.88	1327.50	0.02
378.00	53.33	59.15		59.15	0.000001	0.08	4570.80	1345.00	0.01
378.00	53.16	59.15		59.15	0.000001	0.08	4819.89	1362.50	0.01
378.00	53.00	59.15		59.15	0.000001	0.07	5099.39	1380.00	0.01
378.00	52.53	59.15		59.15	0.000001	0.07	5407.43	1412.00	0.01
378.00	52.06	59.15		59.15	0.000001	0.07	5741.16	1444.00	0.01
378.00	51.59	59.15		59.15	0.000001	0.06	6100.02	1476.00	0.01
378.00	51.12	59.15		59.15	0.000000	0.06	6484.71	1508.00	0.01
378.00	50.65	59.15		59.15	0.000000	0.05	6894.33	1540.00	0.01
378.00	50.18	59.15		59.15	0.000000	0.05	7329.66	1572.00	0.01
378.00	49.71	59.15		59.15	0.000000	0.05	7790.16	1604.00	0.01
378.00	49.24	59.15		59.15	0.000000	0.05	8276.19	1636.00	0.01
378.00	48.77	59.15		59.15	0.000000	0.04	8787.80	1668.00	0.01
378.00	48.30	59.15	49.71	59.15	0.000000	0.04	9324.52	1700.00	0.01

Cross section output

Plan: RNakhwa Ganga Nakhwa RS: 1298 Profile: 02Jan2003 00:00

E.G. Elev (m) 59.16	**Element**	**Left OB**	**Channel**	**Right OB**
Vel Head (m) 0.00	Wt. n-Val.		0.030	
W.S. Elev (m) 59.16	Reach Len. (m)	96.15	96.15	96.15

Crit W.S. (m)		Flow Area (m2)	1971.13
E.G. Slope (m/m)	0.000013	Area (m2)	1971.13
Q Total (m3/s)	378.00	Flow (m3/s)	378.00
Top Width (m)	1000.00	Top Width (m)	1000.00
Vel Total (m/s)	0.19	Avg. Vel. (m/s)	0.19
Max Chl Dpth (m)	3.74	Hydr. Depth (m)	1.97
Conv. Total (m3/s	103135.8	Conv. (m3/s)	103135.8
Length Wtd. (m)	96.15	Wetted Per. (m)	1002.29
Min Ch El (m)	55.42	Shear (N/m2)	0.26
Alpha	1.00	Stream Power (N/m s)	0.05
Frctn Loss (m)	0.00	Cum Volume (1000 m3)	20654.11
C & E Loss (m)	0.00	Cum SA (1000 m2)	6330.00