STUDY OF IN-STREAM STORAGE SYSTEM USING PIANO KEY WEIR

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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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled "**STUDY OF IN-STREAM STORAGE USING PIANO KEY WEIR**" in partial fulfilment of the requirements for the award of the degree of Master of Technology with specialization in Water Resources Development, submitted in the Department of Water Resources Development and Management, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out during a period from November 2009 to June 2010 under the supervision of Dr. Nayan Sharma, Professor, Department of WRD&M, Indian Institute of Technology Roorkee, Roorkee,

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

Dharpale

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

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I am very thankful to my friends for their help and suggestions during the preparation of this report.

Last but not the least; it is my good fortune to express my love and gratitude to my parents, who have always been a source of inspiration, guidance and strength to me.

Finally I am grateful to God to provide me strength and patience for the completion and submission of my thesis.

Date: 30, June 2010 Dipali Ramesh Harpale

Place : Roorkee

ABSTRACT

In-stream storage is the storage volume of water at a given time in the channel or over the flood plain of the streams in a drainage basin or river reach. It stores the water into the stream itself. In this type of storage creation, there is no or very little spilling of the water is involved. Therefore it doesn't cause any submergence which is the greatest advantage of In-stream storage.

In the present scenario of climate change, we need to create more storage for rising demand of water in rural and urban areas. Construction of dams, weirs result in submergence which leads to so many problems. Amongst which submergence is the major issue which leads to rehabilitation and settlement problems. Hence In-stream storage using Piano Key weir has a good prospects to help create storage without submergence and reduced sedimentation.

Due to ongoing climate change effect, the water resources sector is facing increasing demand to meet various uses by the society. For e.g.

- In agriculture the crop water requirement are increasing due to increasing evapotranspiration.
- In hydropower projects, the evaporation increases the required amount of water.
- The demand of water for navigation as well for drinking water is increasing.

The conventional storage dams encounter stiff resistance from environment lobby because of R & R (Relief and Rehabilitation) and submergence.

In the above context, the new concept of Piano Key weir is being researched to provide cost effective solution for creation of required storage volume in the channel itself without involving submergence and R&R (Relief and Resettlement), by cascade development of Piano Key weir infrastructure in a river as per feasibility.

The Piano Key Weir is a particular geometry of weirs associating with a labyrinth shape and the use of overhangs, which reduces the basis length. This new shape of weir baptized Piano Key Weir represents an effective alternative for conventional barrages and dams and can increase in low cost the capacity of evacuation and\or the stocking of several existing dams.

In-stream storage and Piano Key weir are newly developing concepts. Almost no work is done to create In-stream storage. So there is a great need of further study to explore application of this concept.

Hence to study the emerging concept of In-stream storage and Piano Key weir, a case study of Brahmani River in Orissa is under taken. A Piano Key weir is proposed near Kharagprasad in Brahmani River to create the In-stream storage. This storage volume of water is created for the Industrial and drinking water requirement in the upstream of Kharagprasad. The daily requirement of 2.1 cumec of water is to be satisfied with the help of created storage.

For the present study, the satellite imagery data are procured from NRSC to study the river stability and channel plan form. The hydrological data for Samal barrage on the upstream of Kharagprasad sourced from Orissa Government are used to analyse the design flood and the low flow for the design of Piano Key weir. The cross-sections, soil sample data for Brahmani River was used to develop a mathematical model of Brahmani River for the study area. The mathematical model was constructed mainly to work out the storage volume of water during low flood and design flood. The bank spilling information and the backwater length during low flow is also derived from the developed mathematical model. From the present study the various design parameters for Piano Key are designed. The storage volume due to creation of In-stream storage was deduced and the cost function for the study area was developed.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

In-stream storage is the storage volume of water at a given time in the channel or over the flood plain of the streams in a drainage basin or river reach. River water infiltrates into a hydraulically connected aquifer during a rising flood stage and its reverse motion during a recession period recharges the river. The volume of water so stored and released after flood is referred to as bank storage. It is a significant hydrologic process because it can attenuate the flood wave in a river that has permeable bank materials, and can contribute substantial discharge to rivers during base flow.

There is a tremendous increase in the water consumption due to great increase in population. Moreover, lot of industries and hydropower projects have also come up and expected to come in future also. The standard of living of people is also going high day by day. To meet this water demand, there is a great need to store water in the river. In the present scenario of climate change, we need to create more storage for rising demand of water in rural and urban areas.

Construction of dams, weirs result in submergence which leads to so many problems. Amongst which submergence is the major issue which leads to rehabilitation and settlement problems.

Significantly in the middle of last century, the gated barrages concept came into practice replacing the weirs with falling shutters for hydraulic and cost advantages. But Conventional barrages face the following problems.

- Due to the gated structure, operation, maintenance and constant supervision is required.
- Due to poor operation and maintenance, considerable bed aggradations occur in the upstream.
- The cost is also increasing sharply due to the hike in the steel prices
- Significant sediments are deposited in the upstream pool hence retrogression also occurs at the downstream as most of the significant.

Therefore day by day, the need has been felt to bring about further innovation in the conventional barrages so an answer to this is a recent innovative research which is ongoing to develop Piano Key weir. The submergence problem can be envisaged by using emerging concept of In-stream storage systems. Low flow storage can be created in the stream itself without any submergence by using Piano Key Weir. It is relatively cheaper; no submergence is involved and can be implemented quickly.

The piano key weir doesn't have any gates. So it will involve very little maintenance and would be cheaper also. Moreover no manpower is required to operate the gates. No operation cost is involved hence. This also provides smooth passage of the incoming sediments along its ramps with minimum river channels.

Obviously the storage created by this spillway would be less. So we need to create cascades of piano keys. So that more than one piano key spillway can be constructed and then the required storage can be created. The backwater generated due to obstruction would result in the storage.

This new Piano Key Weir Technology has already been used by the Himachal Pradesh Power Corporation Ltd. (HPPCL) on Sawra Kuddu Barrage. The new technology is reported to have a number of hydraulic advantages.

As an example for use of Piano Key Weir as in-stream storage, 5 Piano Key Weir type barrages can be set up over about 100 km stretch of Brahmani between Kharagprasad in Dhenkanal district and Jenapur in Jajpur district. The barrages can be 25 kms apart from each other and the volume of the in-steam storage in low flow condition from one barrage is estimated to be about 17.701 million cubic metres.

Accordingly, the In-steam storage from 5 barrages is expected to be 88.505 million cubic metres. The cost of the project will range from Rs 40 crore to Rs 80 crore each, depending on the site conditions. Since 50 percent of the Brahmani water remains untapped, this can be a viable proposition.

While the discharging capacity of a Piano Key Spillway is much higher than that of a conventional Ogee spillway, for a given maximum reservoir level, the live storage can be more in case of the former compared to gated spillway. From a review of the status of water resources use of Brahmani River as far as Orissa state is concerned, there is ample scope to harness its share through creation of storages. The salient information pertaining to water uses of the Brahmani river is given below which will indicate the above observation:

- □ Average yield at head of delta = 18,318Mm^3
- □ Annual yield of Orissa portion = 10,661Mm^3
- □ 75% dependable yield For Orissa portion = 8277Mm^3
- □ 75% dependable water resources at Rengali dam = 9140Mm^3
- □ Total water withdrawal = 4696 Mm^3 /yr

For Irrigation = 4049 Mm^3 /yr

For Domestic, Rural = 260.74 Mm^3/yr

For Domestic Urban = 63.98 Mm^3/yr

Industrial = 322.44 Mm^3/yr

- □ Volume of water resources yet to be utilized = 4444Mm^3
- □ About 50% share of Orissa remains to be utilised

1.2 OBJECTIVES

- The objective is to create In-stream storage in the river. The In-stream storage in Brahamani River will be created using Piano Key weir.
- To calculate backwater effect and the storage volume of water stored as In-stream storage using mathematical models such as HEC-RAS 4 model.
- To Study the river channel stability.
- To investigate the stability with regard to channel plain form directly from satellite, an indirect assessment of depth and other information like deposition and scouring.
- To identify potential sites for the barrage near Kharagprasad.

To study the In-stream storage system for drinking water and industrial requirement, a case study of Brahamani River in Orissa will be undertaken. To create in-stream storage in the river, the Piano key weir design will be done.

1.3 DESCRIPTION OF A STUDY AREA

The area under the consideration for the present study enclose a 125 km river stretch of Brahmani River encompassing 41 no of different cross-sections including cross-section at Kharagprasad which is the proposed Piano Key weir location. The prime data, the study utilize is the survey data of the cross sections taken in the year 2009. The other hydrologic data of the river at different locations (Samal barrage and Rengali Dam) for 18 years from 1990 to 2007 sourced from Orissa Government. Photographic images derived from different digital satellites imageries processing in the earlier works are incorporated to draw the idea of the area at a glance.

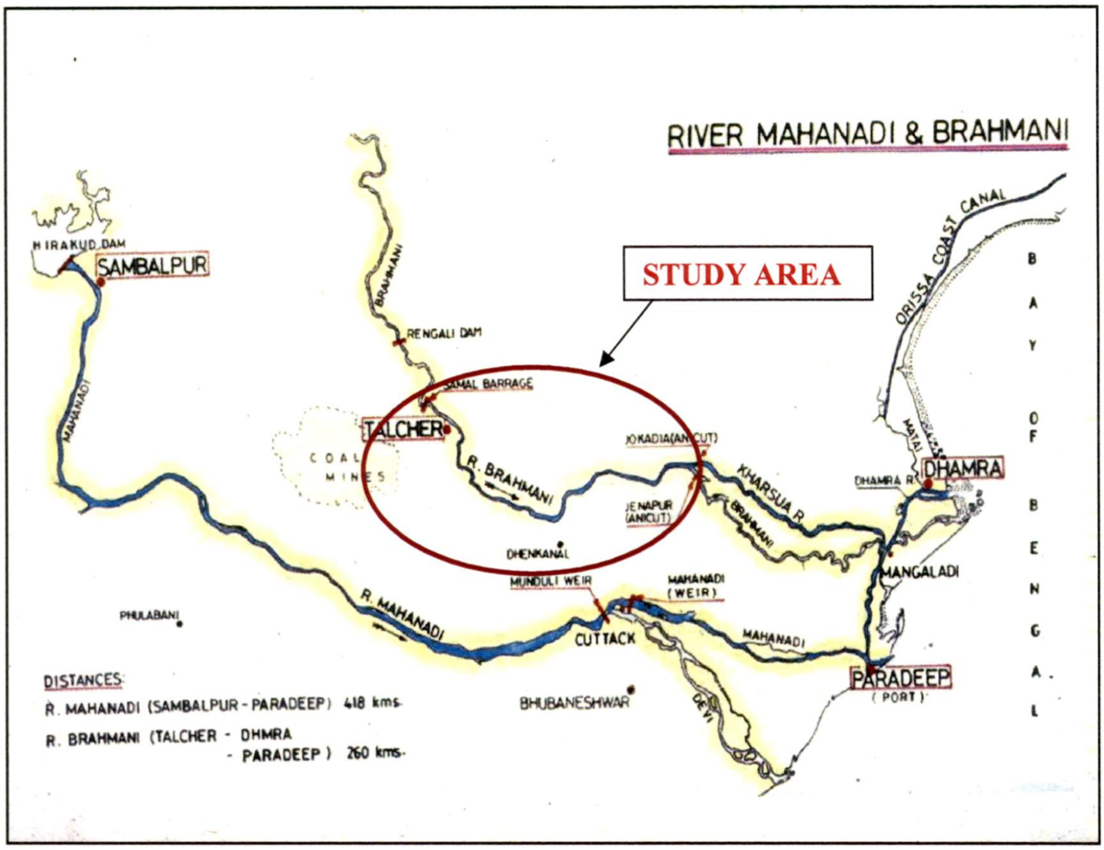


Fig 1.1 Map showing study area

1.4 STRUCTURE OF THE DISSERTATION

The dissertation contains study of In-stream storage using Piano Key weir in Brahmani River near Kharagprasad. The chapter 1 deals with the introduction of the studies. The chapter 2 contains a brief review of literature with respect to methods to calculate in-stream storage and back water length. The chapter 3 contains the study of the satellite imageries procured from the NRSC. In chapter 4, the preliminary design of Piano Key weir is carried out. All the design parameters of Piano Key weir are worked out. Chapter 5 deals with the mathematical modelling of Brahmani River for 17.25 km reach on upstream of Kharagprasad to get the In-stream storage volume. In chapter 6, the cost function for the proposed Piano Key weir and In-stream storage was developed. Chapter 7 briefly describes the results and discussions of the various analysis carried out in chapters 3, 4, 5 and 6. The last chapter 8 contains conclusions and the scope for future study.

2.1 Calculation of channel storage based on Horton-strahler-Rzhanisyn:

This is a method for calculating water storage in a channel network and its separate link.It is based on statistical laws of channel network structures suggested by Horton, Strahlerand Rzhanistsyn. The use of these laws makes it possible to divide the channel network into groups of rivers (the so-called 'streams' according to Horton) and river reaches (the 'segments' according to Strahler) of a certain order. These segments have similar morphological and hydrological characteristics, typical only for the given group, such as mean length / and mean cross section w at periods of low flow.

If the number of segments n is known for a group of given order o, it is possible to determine the total length of segments of this order in the channel network (In), and the total water storage (Vo) in the segments of order o, and in the entire channel network (V%) as the sum of storages in the segments of all orders from 1 to S:

$$£V = (ln_{1})\, w_{1} + (ln_{2})\, w_{2} + \ldots\ldots\ldots + (ln_{0})\, w_{0} \ldots\ldots\ldots + (ln_{s})\, w_{s} \qquad (1)$$

In calculating water storage using formula (1) for large channel networks (of tenth and higher order), it is usually impossible to determine the number and length of segments of small orders since their number is too great. Further, observational data on stages of small rivers are inadequate to calculate the mean cross sectional area of segments of small orders. However, these difficulties may be overcome by extrapolating /, n and w known for the segments of larger orders (o) to the segments of smaller orders (o - x). Extrapolation was also based on Horton-Strahler-Rzhanitsyn laws and was made with formulae resulting directly from these laws:

The formula for extrapolating the cross sectional area of segments in low flow periods may be expressed in the general form (Rzhanitsyn, 1960);

$$w_{o-x} = w_{o}\, C_{1}^{-x}\, C_{2}^{C_{3}^{o-x} - C_{3}^{x}} \qquad (2)$$

Parameters Cx, C2 and C3 are described in Gorbunov (1971).

Routine observations of water stages serve as initial data in calculations of water storage. The errors in the calculations of total water storage with adequate initial data are within 10 per cent. The dynamics of channel storage in different periods of spring flood are considered. The practical application of the method to forecast the inflow to the Cheboksary Reservoir on the Volga two months in advance is described.

2.2 Muskingum river routing to calculate dynamic bank storage:

The Muskingum unsteady flow routing method is well established in the hydrological literature and its modest data requirements make it attractive for practical use.

In the nonlinear form of Muskingum routing (Gill, 1978; Yoon and Padmanabhan, 1993), storage is related to weighted exponential functions of the discharges flowing into and out of a river reach. The nonlinear function arises from assumed power relationships between channel storage and river stage, and discharge and river stage. Consequently, Muskingum procedures account explicitly for channel storage only, and not total storage along a river reach which may include lateral inflows (tributary or difffuse inputs) or outflows, losses (recharge of groundwater aquifers or evapotranspiration), and temporal changes in bank storage. Parameters in the Muskingum models are typically derived by calibration using measured inflow and outflow discharge hydrographs which do, however, relate to changes in total storage along a river reach. Traditionally, total storage is, therefore, implicitly accounted for in the calibration of the routing parameters for the river channel.

Birkhead and James (1998) modified the traditional nonlinear Muskingum routing equations to synthesise the rating relationship (relationship between stage and discharge) based on a measured short-term local stage hydrograph at the site of interest, and a corresponding discharge hydrograph at a remote site along the river. Application of the procedures to a section of the Sabie River (South Africa) showed that neglecting bank storage resulted in poor estimates of the storage weighting factor. The procedures were successfully modified to account for bank storage by assuming instantaneous response of seepage in the alluvial bank zone, this being justified by the high hydraulic conductivity of the fluvial sediments. The procedures described by Birkhead and James (1998) are further developed here to explicitly account for the interaction between channel flow and bank storage in rivers with permeable river banks of any hydraulic conductivity. The effects of accounting for bank storage exxplicitly and implicitly (as in traditional Muskingum routing) on the estimation of the nonlinear Muskingum routing parameters from measured discharge hydrographs are also considered.

The approach is verified by resynthesising a rating function used in rigorous routing in a hypothetical channel with permeable banks. The method is also applied to determine routing parameters for a reach along the Sabie River in South Africa by explicitly accounting for bank storage.

2.3 Incorporating transient storage in conjunctive stream-aquifer modelling:

There has been growing interest in incorporating the transient storage effect into modelling solute transport in streams. In particular, for a smaller mountain stream where flow is fast and the flow field is irregular (a favourable environment to induce dead zones along the stream), long tails are normally observed in the stream tracer data, and adding transient storage terms in the advection–dispersion transport equation can result in more accurate simulation. As per the authour previous studies on transient storage modelling account for temporary, localized exchange between the stream and the shallow groundwater in the hyporheic zone, larger scale exchange with the groundwater in the underlying aquifer has rarely been included or properly coupled to surface water modelling. In this study, the researchers complement previous modelling efforts by incorporating the transient storage concept in a conjunctive stream–aquifer model. Three well-documented and widely used USGS models have been coupled to form the core of this conjunctive model: MODFLOW handles the groundwater flow in the aquifer; DAFLOW accurately computes unsteady streamflow by means of the diffusive wave routing technique, as well as stream–aquifer exchange simulated as streambed leakage; and MOC3D computes solute transport in the groundwater zone. In addition, an explicit finite difference package was developed to incorporate the one-dimensional transient storage equations for solute transport in streams. The quadratic upstream interpolation (QUICK) algorithm is employed to improve the accuracy of spatial differencing. An adaptive step size control algorithm for the Runge–Kutta method is incorporated to increase overall model efficiency. Results show that the conjunctive stream–aquifer model with transient storage can handle well the bank storage effect under a flooding event. When it is applied over a stream network, the results also show that the stream–

aquifer interaction acts as a strong source or sink along the stream and is too significant to be ignored. The adaptive step size control for stream solute transport improves overall model performance.

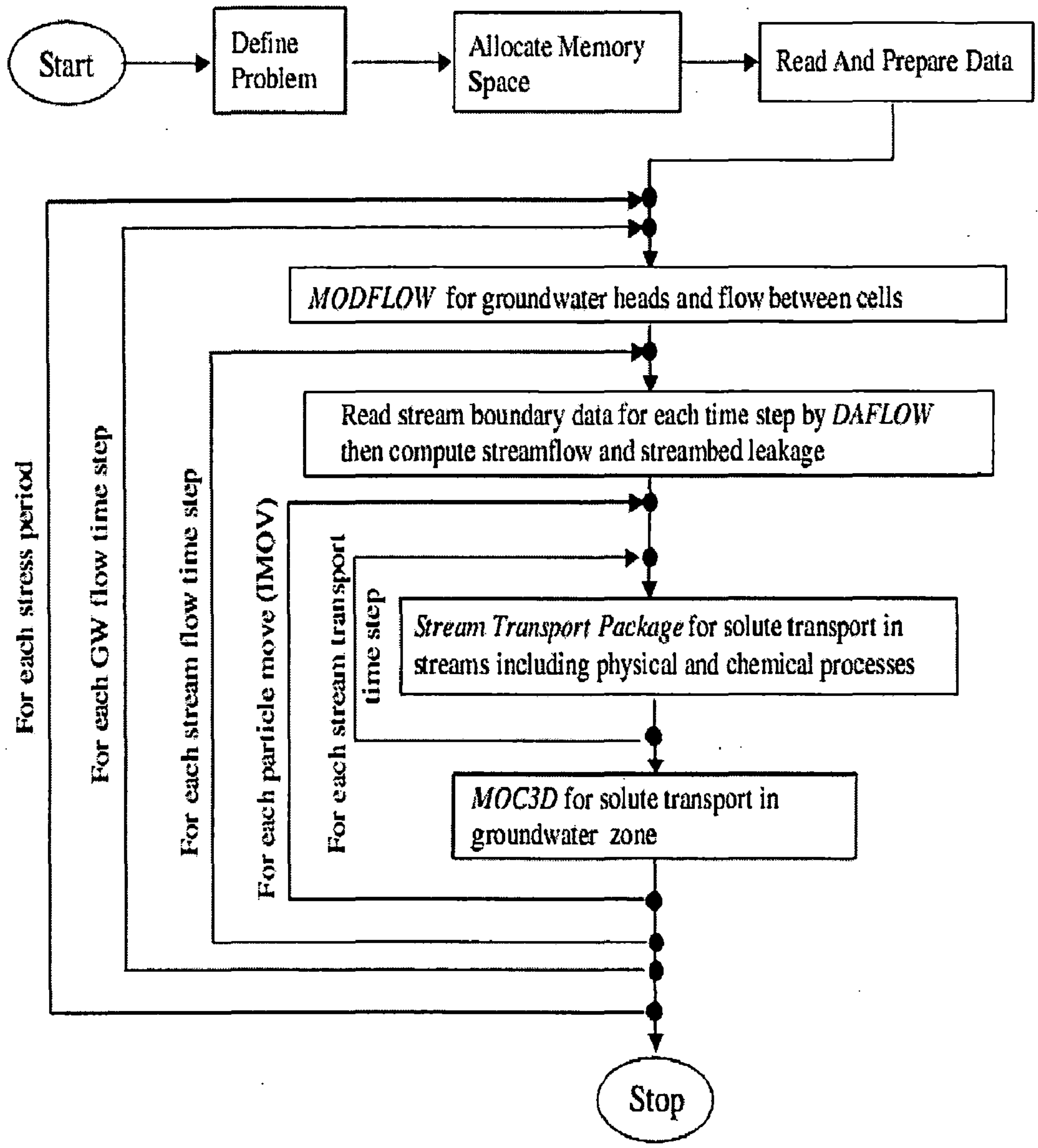


Fig. 2.1 General Flow Chart For Stream-Aquifer Modelling

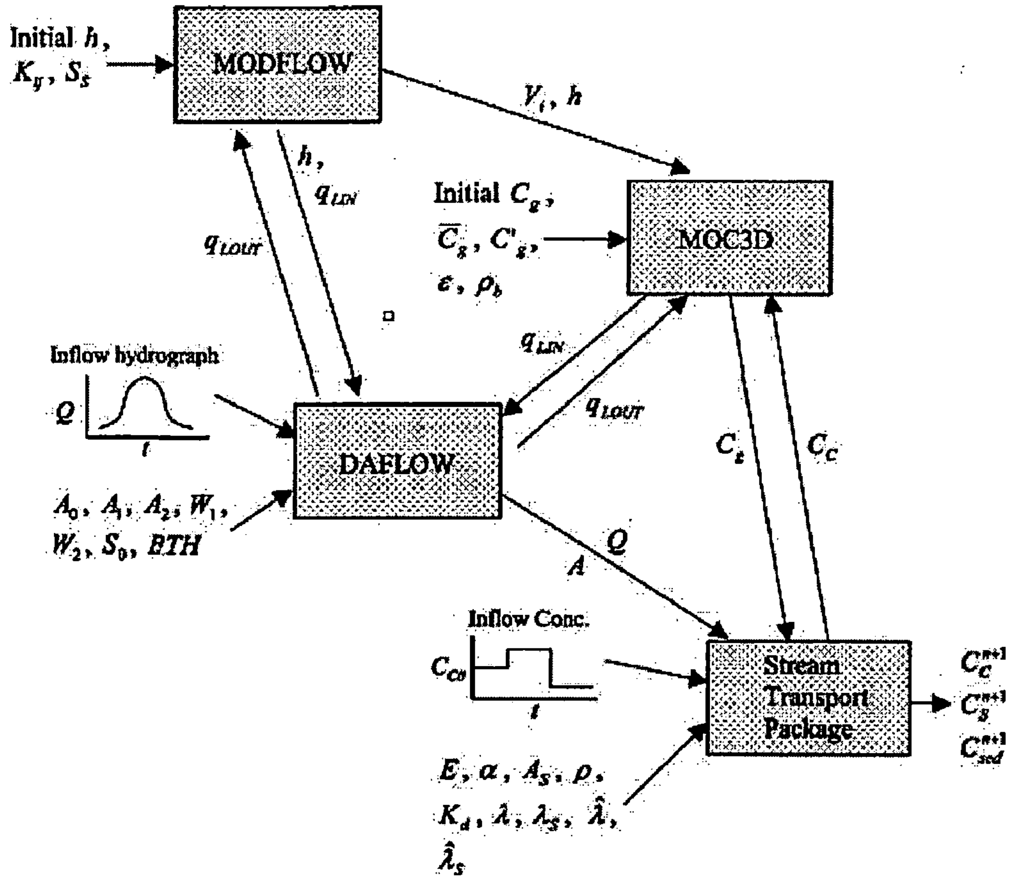


Fig. 2.2 Framework for the conjunctive stream-aquifer model showing input, output, and shared parameters.

2.4 GRADUALLY VARIED FLOW

2.4.1 UNIFORM AND NON-UNIFORM FLOW:

1) **Uniform flow:** If for a given length of tthe channel, the velocity of flow, depth of flow, slope of the channel and cross-section remain constant, it is said to be *uniform-flow.*

2) **Non-uniform flow:** If for a given length of channel, the velocity of flow, depth of flow etc. Do not remain constant, the flow is said to be *non-uniform flow.*

Mathematically, uniform and non-uniform flows are written as:

For uniform flow, $\frac{dy}{ds} = 0, \frac{dv}{ds} = 0$

For non-uniform flow, $\frac{dy}{ds} \neq 0, \frac{dv}{ds} \neq 0$

Non-uniform flow in open channels is also called varied flow, which is classified in the following two types as:

a) **Rapidly varied flow:** It is defined as that flow in which depth of flow changes abruptly over a small length of the channel. As shown in figure, when there is any obstruction in the path of flow of water, the level of water rises above the obstruction and then falls and again rises over a small lenfth of channel. Thus the depth of flow changes rapidly over a short length of the channel. For this short length of the channel the flow is called *rapidly varied flow(R.V.F.).*

b) **Gradually Varied Flow:** If the depth of flow in a channel changes gradually over a long length of the channel, the flow is said to be *gradually varied flow (G.V.F.).*

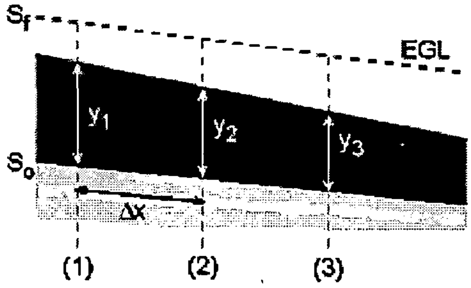


Fig. 2.3 Gradually Varied Flow Profile

Equation of gradually varied flow for steady state condition:

$\frac{dh}{dx} = \frac{(S_0 - S_f)}{1-(F_r)^2}$ Where,

h= depth of flow,

S_0 = slope of channel bed,

S_f = slope of energy line,

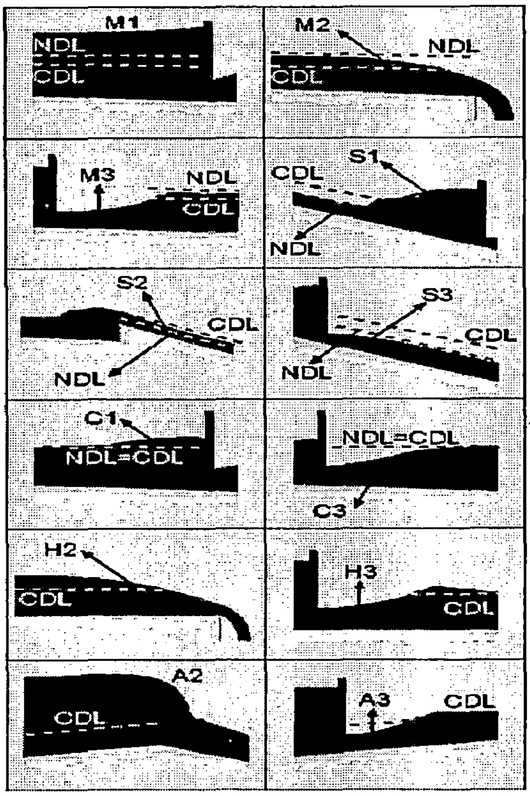
$F_r = \frac{V}{\sqrt{gh}}$

V = mean velocity of flow,

g = gravitational acceleration,

h= depth of flow

According to above equation, different water surface profiles can be derived, and there are altogether 12 possible shapes, as shown in the table and figure. The surface profiles can be clasified into 5 categories according to the channel bed slopes. They include mild(M), steep(S), Critical(C), horizontal(H) and adverse(A) slopes. The surface profiles can be further devided into different types accordint to its depth relative to the channel's normal depth(Y_0) and critical depth(Y_c).



Slope Type	Surface Type	Flow Depth	Flow Type
Mild (M) $(0 < S < S_c)$	M1	$y > y_o > y_c$	Subcritical (Fr < 1)
	M2	$y_o > y > y_c$	Subcritical (Fr < 1)
	M3	$y_o > y_c > y$	Supercritical (Fr > 1)
Steep (S) $(S > S_c > 0)$	S1	$y > y_c > y_o$	Subcritical (Fr < 1)
	S2	$y_c > y > y_o$	Supercritical (Fr > 1)
	S3	$y_c > y_o > y$	Supercritical (Fr > 1)
Critical (C) $(S = S_c = S_o)$	C1	$y > y_o = y_c$	Subcritical (Fr < 1)
	C3	$y_c = y_o > y$	Supercritical (Fr > 1)
Horizontal (H) $(S = 0)$	H2	$y > y_c$	Subcritical (Fr < 1)
	H3	$y_c > y$	Supercritical (Fr > 1)
Adverse (A) $(S < 0)$	A2	$y > y_c$	Subcritical (Fr < 1)
	A3	$y_c > y$	Supercritical (Fr > 1)

Fig. 2.4 Different water surface profiles

2.4.2 Back Water Curve and Afflux:

Consider the flow over a dam as shown in figure. On the upstream side of the dam, the depth of water will be rising. If there had not been any obstruction(such as dam) in the path of flow of wter in the channel, the depth of water would have been constant as shown by dotted line parallel to the bed of the channel in figure. Due to obstruction, the water level rises and it has maximum depth from the bed at some section.

Let, h_1= depth of water at the point, where the water starts rising up, and

h_2=maximum height of rising water from bed.

Then (h_2-h_1) =*Afflux*. The afflux is defined as the maximum increase in water level due to obstruction in the path of flow of water. The profile of the rising water on the upstream side of the dam is called *back water curve*. The distance along the bed of the channel between the sections where water starts rising to the section where water is having maximum height is known as *length of back water curve*.

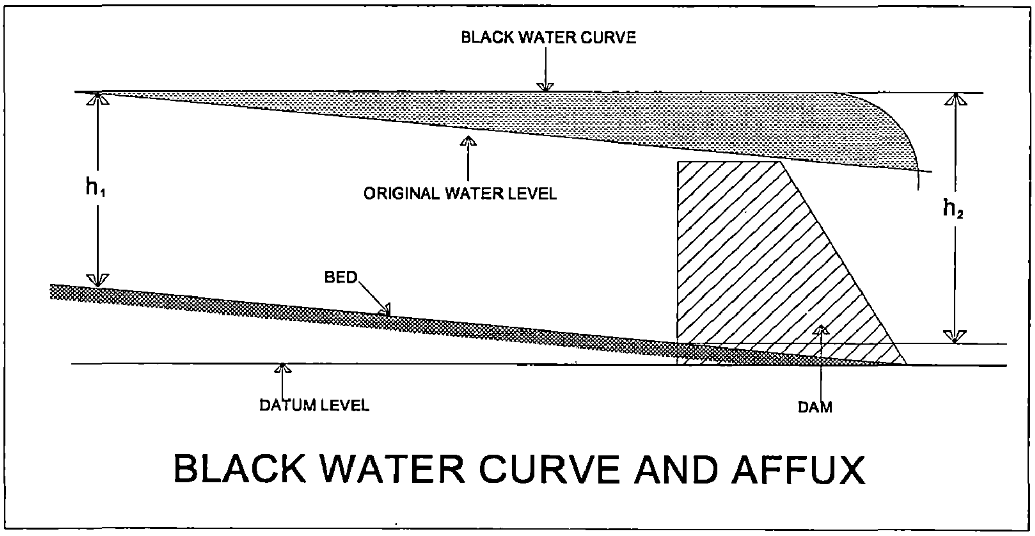


Fig. 2.5 Backwater curve and affux

Method for calculation of Backwater:

A new model called LBLR for Linear Backwater Lag-and-Route is proposed which approximates the Saint-Venant equations linearized around a non-uniform flow in a finite channel (with a downstream boundary condition).

This model takes into account the backwater effects also. We now consider the discharge and the water depth variations around a non-uniform steady flow. The equilibrium regime is described by Q(x) = Q and Y(x), solution of the following ordinary differential equation for a boundary condition defined by downstream elevation Y(X):

$$\frac{dY}{dx} = \frac{S_b - S_f}{1 - F^2}$$

Where, S_f and F can be expressed as functions of Y.

A channel with a backwater curve is approximated by the concatenation of two pools. This consists in approximating the backwater curve by a stepwise linear function: a line parallel to the bed in the upstream part (corresponding to the uniform part) and a line tangent to the free surface at the downstream end in the downstream part. Let x1 denote the abscissa of the intersection of the two lines (the discharge and the water depth variations at this point are denoted q1 and y1, respectively). The corresponding approximation of the backwater profile is schematized in Figure, SX represents the slope of the backwater curve at the downstream end of the reach and is computed using above equation no ( )

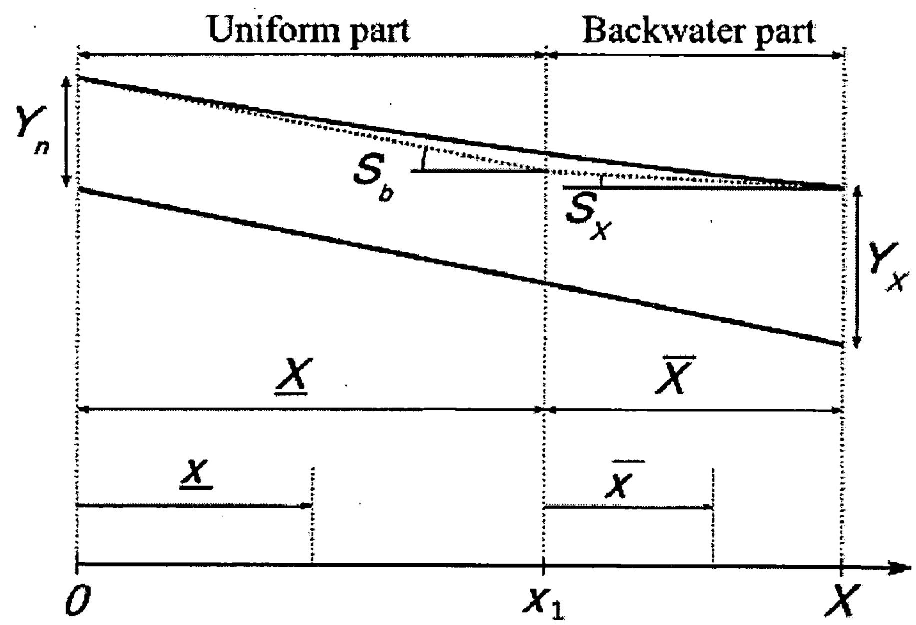


Fig. 2.6 Backwater curve approximation scheme

Validation:

For validation purpose a trapezoidal prismatic channel is considered.

The real backwater curve, approximate backwater curve derived from the model and normal water are shown in the figures below for three cases:

1) Cross structure effect: a reservoir at the downstream end

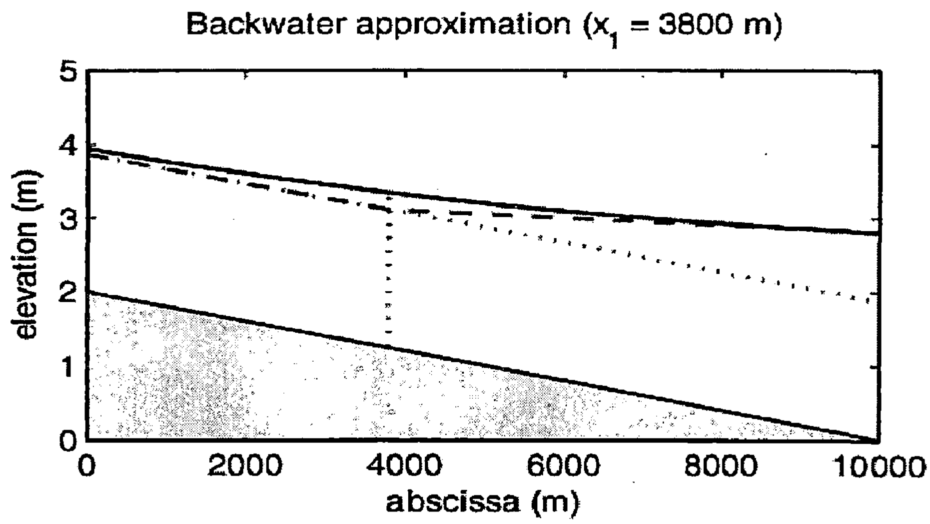


Fig. 2.7 Backwater curve and its approximation with a reservoir at the downstream end. Real backwater curve (-), uniform normal depth (...) and approximate backwater curve(- - -)

2) Cross structure effect: a gate at the downstream end

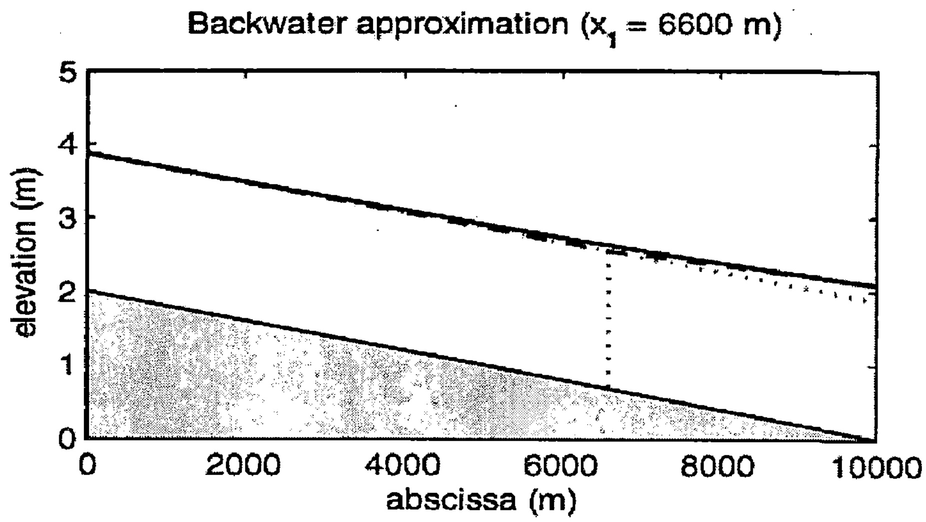


Fig. 2.8 Backwater curve and its approximation with a gate at the downstream end. Real backwater curve (-), uniform normal depth (...) and approximate backwater cvrve(- - -)

3) Cross structure effect: a weir at the downstream end

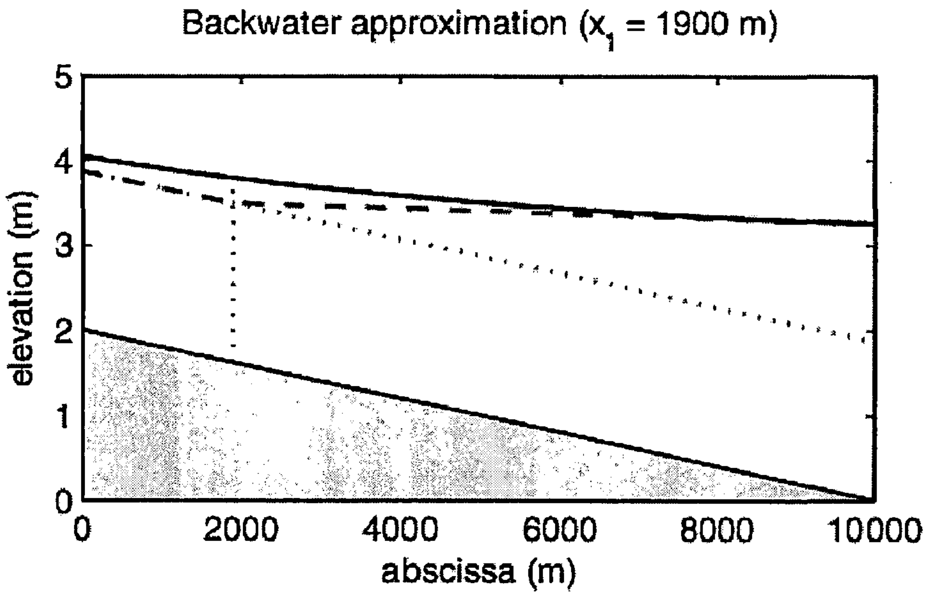


Fig. 2.9 Backwater curve and its approximation with a weir at the downstream end. Real backwater curve (-), uniform normal depth (...) and approximate backwater curve(- - -)

In this author has presented an example of hydraulic modelling: the numerical integration of the energy equation for gradually varied flows (GVFs) in open channel. It is one of the simplest forms of numerical model: i.e. the numerical integration of a steady one-dimensional flow.

The energy equation is also called the backwater equation. It is usually rewritten as:

$$\frac{\partial h}{\partial s} = -S_f$$

where S_f is the friction slope defined as:

$$S_f = \frac{\zeta_0}{\rho g DH} = f\frac{v^2}{DH\,2g}$$

Where ζ0 is the boundary shear stress and D^ is the hydraulic diameter.

BASIC ASSUMPTIONS:

1) The backwater calculations are developed assuming [HI] a non-uniform flow, [H2] a steady flow, [H3] that the flow is gradually varied and [H4] that, at a given section, the flow resistance is the same as for a uniform flow for the same depth and discharge, regardless of trends of the depth.
2) The GVF calculations (i.e. backwater calculations) neither apply to uniform equilibrium flows nor to unsteady flows nor to rapidly varied flows (RVFs). Furthermore, the last assumption [H4] implies that the Darcy, Chezy or Gauckler-Manning equations may be used to estimate the flow resistance, although these equations were originally developed for uniform flows only.

METHOD OF CALCULATION:

1) A sketch of the longitudinal profile of the channel must be drawn. Control structures (e.g. gate and weir) must be included.
2) The location of 'obvious' control sections^ must be highlighted as at these locations the flow depth is known. Classical control sections include sluice gate, overflow gate, broad-crested weir, hydraulic jump, over fall and bottom slope change from very flat to very steep (or from very steep to very flat).
3) The uniform flow properties must be computed at each position along the channel. In practice, uniform flow conditions must be re-computed after each change of bottom slope, total discharge, cross-sectional shape (e.g. fi-om rectangular to trapezoidal), cross-sectional characteristics (e.g. bottom width) and boundary roughness (i.e. bottom and sidewalls).
4) The uniform-flow properties must be computed at each position along the channel. In practice, uniform flow conditions must be re-computed after each change of bottom slope, total discharge, cross-sectional shape (e.g. fi-om rectangular to trapezoidal), cross-sectional characteristics (e.g. bottom width) and boundary roughness (i.e. bottom and sidewalls).

Backwater calculations start from positions of known flow depth. Calculations may be performed in the upstream and downstream flow directions up to the next control sections. At a control section, the type of control affects the flow computation through the control section. That is, the Belanger equation (i.e. momentum equation) must be applied for a hydraulic jump; the Bernoulli equation (and specific energy concept) is applied for a gate (or a weir); for a smooth change from sub- to supercritical flow (e.g. spillway crest), calculations continue down-stream of the critical flow depth section. Sketch the composite free-surface profiles.

2.4.3 HEC-RAS-(Version-4.0, 2008)

This is the latest version developed by US Army Corps of Engineers at Hydrologic Engineering Center. This is Next Generation of hydrologic engineering software which encompasses several aspects of hydrologic engineering including; river hydraulics; reservoir system simulation; flood damage analysis; and real time river forecasting for reservoir operations. This system is comprised of a graphical user interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities. The HEC-RAS system will ultimately contain three one dimensional hydraulic analysis components (i) Steady flow water surface profile (ii) Unsteady flow simulations (iii) movable boundary sediment transport computations. Apart from this software contains several hydraulic design features. This is capable of importing GIS data or HEC-2 data (Brunner, 2002; HEC-RAS Manual, 2008).

It is an integrated system of software, designed for interactive use in a multi-tasking environment. The system is comprised of a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities.

The HEC-RAS system will ultimately contain three one-dimensional hydraulic analysis components for:

- Steady flow water surface profile computations
- Unsteady flow simulation
- Movable boundary sediment transport computations

A key element is that all three components will use a common geometric data representation and common geometric and hydraulic computation routines. In addition to the three hydraulic analysis components, the system contains several hydraulic design features that can be invoked once the basic water surface profiles are computed (HEC-RAS Manual, 2008).

The review of existing models indicates that several models are available with different features. All the models use St. Venant's equations and have different sediment predictors, energy slope relations and distribution of aggradation/degradation equations. A natural river has many complexities due to its size, flow variation, concentration of sediment and its properties, engineering work carried out on the river and other geographical, meteorological, social factors. Due to these reasons, cannot have universal applicability. Hence, for modelling a particular river one should be very careful to choose a model, which is applicable according to the characteristics of that river.

2.4.3.1 Overview of Hydraulic Capabilities

HEC-RAS is designed to perform one dimensional hydraulic calculation for a full network of natural and constructed channels. The following is a description of the major hydraulic capabilities of HEC-RAS.

- Steady flow water surface profiles: This component of the modelling system is intended for calculating water surface profiles for steady gradually varied flow. The system can handle a single river reach, dendritic system, or a full network of channels. The steady flow component is capable of modelling subcritical, 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 (i.e. hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junctions).

  The effects of various obstructions such as bridges, culverts, weirs, spillways and other structures in the flood plain may be considered in the computations. The steady flow system is designed for application in flood plain management and flood insurance studies to evaluate floodway encroachments. Also, capabilities are available for assessing the change in water surface profiles due to channel improvements, and levees.

- Unsteady Flow Simulation
- Sediment Transport/Movable Boundary Computations

CHAPTER 3

STUDY OF SATELLITE DATA FOR CHANNEL STABILITY AND IDENTIFICATION OF BARRAGE SITE IN BRAHMANI RIVER NEAR KHARAGPRASAD

3.1 General

A suitable barrage site is to be identified near Kharagprasad village for the purpose of creating in-stream storage using piano key weir technology. The proposed creation of in-stream storage is envisaged to meet the water requirement primarily for industrial uses besides possible uses for drinking water etc. In the newly proposed piano key weir type barrage would facilitate enhancement of discharging capacity along with movement of incoming sediment charge in the river flow. This type of piano key barrage will be much cheaper than conventional barrages and would require hardly any man power for O&M. The piano key weir type barrage will create the desired in-stream storage within the channel cross section of the Brahmani River which will also help in replenishment of sub soil aquifer along both the banks.

The first task for locating the piano key weir type barrage relates to carefully analyse the suitability from the consideration of river channel stability and also assessing the fulfilment of physical attributes of the potential sites.

The study area for satellite imagery based analysis is considered from the downstream of the Samal Barrage up to the Jajpur. The present report pertains to the study of channel stability for identifying suitable barrage sites near Kharagprasad village using multi date satellite imageries.

3.2 Satellite Data Used in the Study: The satellite data of the year 1998 and 2009 have been procured from NRSA Hyderabad. The list of detailed remote sensing data collected is given in Table 3.1 below. The following procedure has been adopted for analysis using remote sensing technique.

Table 3.1: Details of Satellite Data

S. No.	Satellite	Sensor	Path/Orbit	Row/Sector	Date of Pass	Year
1	IRS 1D	LISS III	106	57	12th Apr	1998
2	IRS P6	LISS III	106	57	18th Mar	2009
3	IRS P6	LISS III	105	57	6th Apr	2009

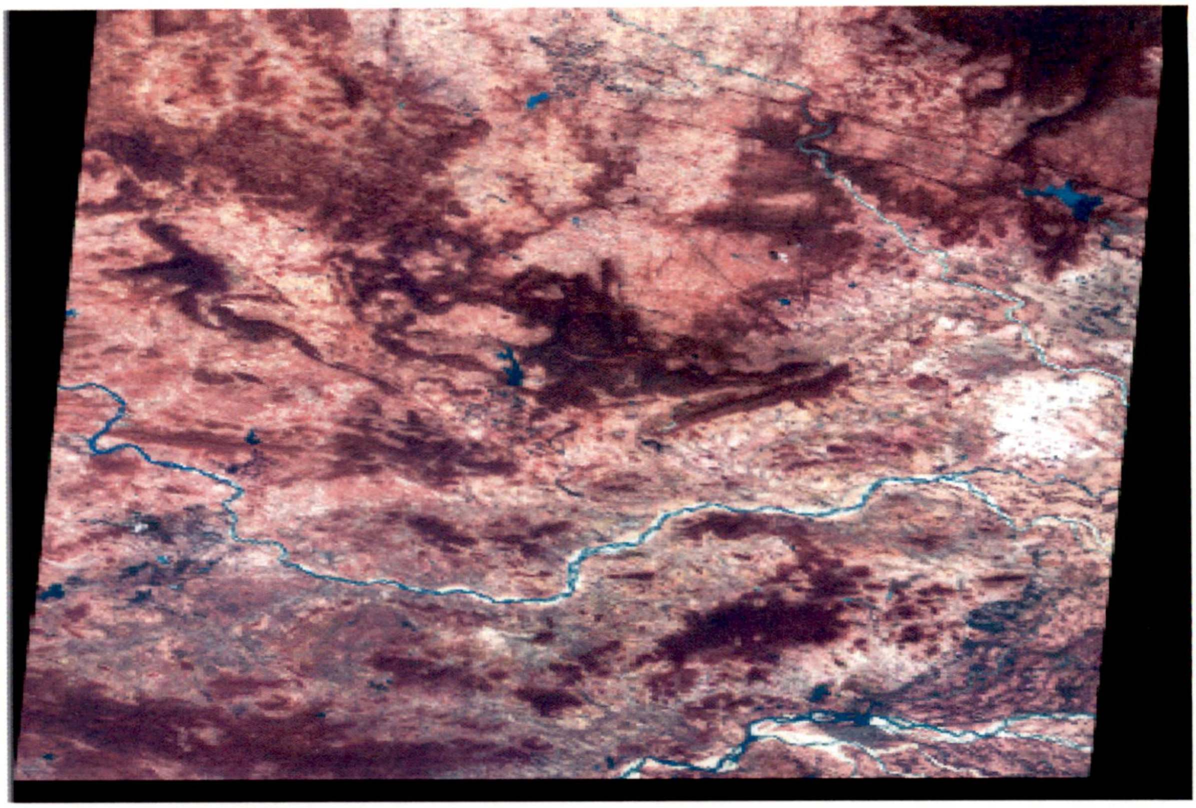


Fig 3.1 Satellite Image of Brahmani River on 12th April, 1998

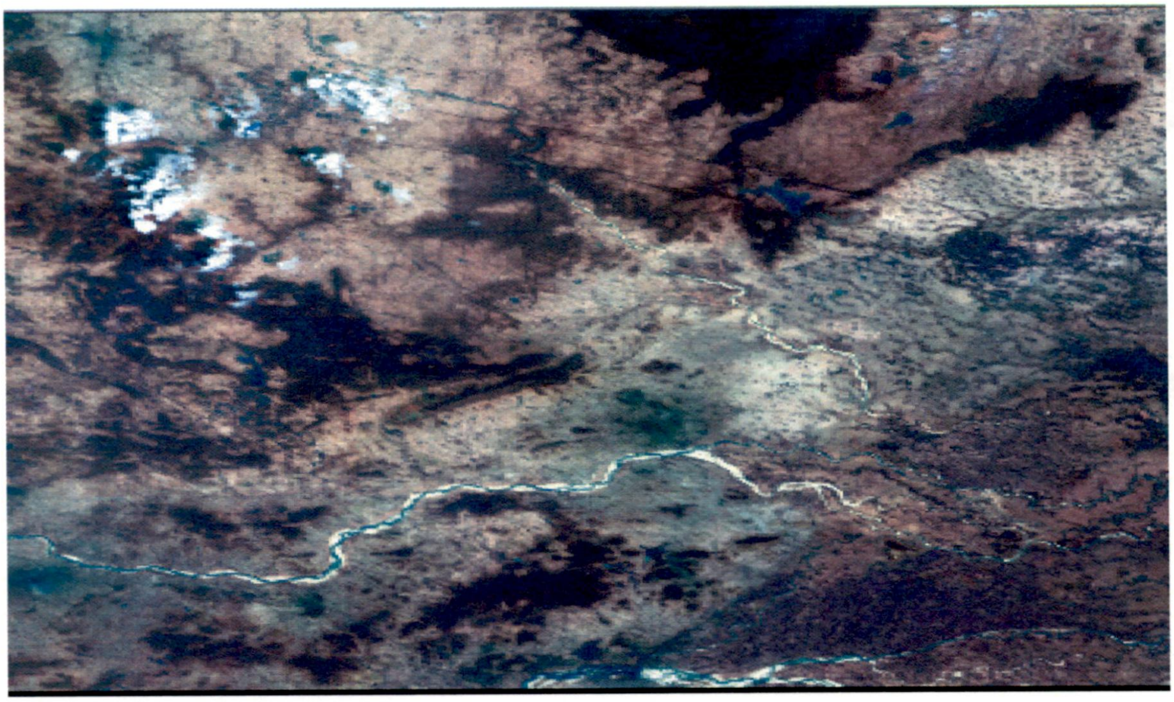


Fig 3.2 Satellite Image of Brahmani River on 18th March, 2009

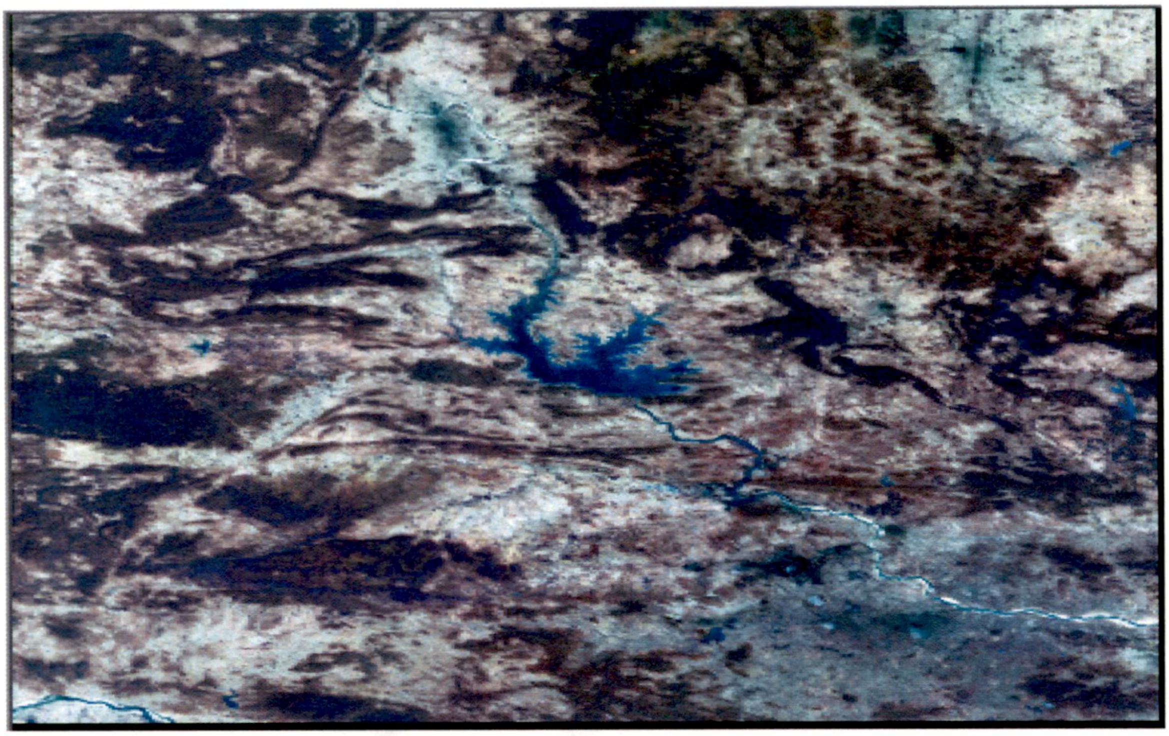


Fig 3.3 Satellite Image of Brahmani River on 6th April, 2009.

3.3 Data Processing:

Remote sensing data have been used to extract information regarding the morphological behaviour and the bank shifts, island formation, deposition, erosion etc of the Brahmani River implicitly. For this purpose, ERDAS-IMAGINE 8.5 software has been used as indicated below:

3.3.1 Data Decoding

ERDAS IMAGINE 8.5 has in-built routines to read IRS LISS data directly. However, it requires input regarding the header file length. For IRS LISS III the length of header file is 0 bytes. The images are first imported to the software. Then the images are displayed using the Image viewer.

3.3.2Data registration

As the Earth and Satellite are both dynamic systems, hence as the data are collected some geometric disturbance creeps in. This affects the scale and leads to errors in measurement. To get accurate measurements, the image has to be registered to the actual earth. This process is known as data registration.

Data registration consists of two basic processes of geocoding and data resampling.

For geocoding, the image is to be correlated to the salient ground features such as roads, bridges, river junctions or by defining the exact latitude and longitude values of the four corner points of the image. In case of river Brahmaputra, there are very few salient ground points which can be clearly identified on the satellite image. ERDAS IMAGINE 8.5 has the capability to geocode satellite data to the ground by defining the latitude and longitude values of the 4 corner points of the image using Polyconic projection system. These values of the 4 corner points are available in the Leader file available with each data set. With help of this input each image is geocoded, using WARP function in ERDAS IMAGINE 8.5.

After geocoding is performed, resampling or intensity interpolation of the geocoded image is carried out using Bi-linear interpolation. This is performed by RESAMPLE function or ERDAS IMAGINE 8.5. After the registration process, the image for year 1998 looks as shown in Fig. 3.4.

3.3.3 Mosaicking

As the study area is partly covered in the satellite imageries, hence all these images have to be merged together to for m one complete mosaicked image. This will help in adopting a single base line for making measurement. The line joining all the imageries of the study area for one year has been carried out by MOSAIC function of ERDAS IMAGINE 8.5. After mosaicking the two scenes of year 2009, the image looks as shown in Figure 3.5.

3.3.4 Digitisation

After all the images of one year have been mosaicked to form one single image, to find the position of bank line, onscreen digitization of the bank line has to be carried out. Each bank line, the position of live channels and reverine feature are digitised and stored with help of ARC GIS.

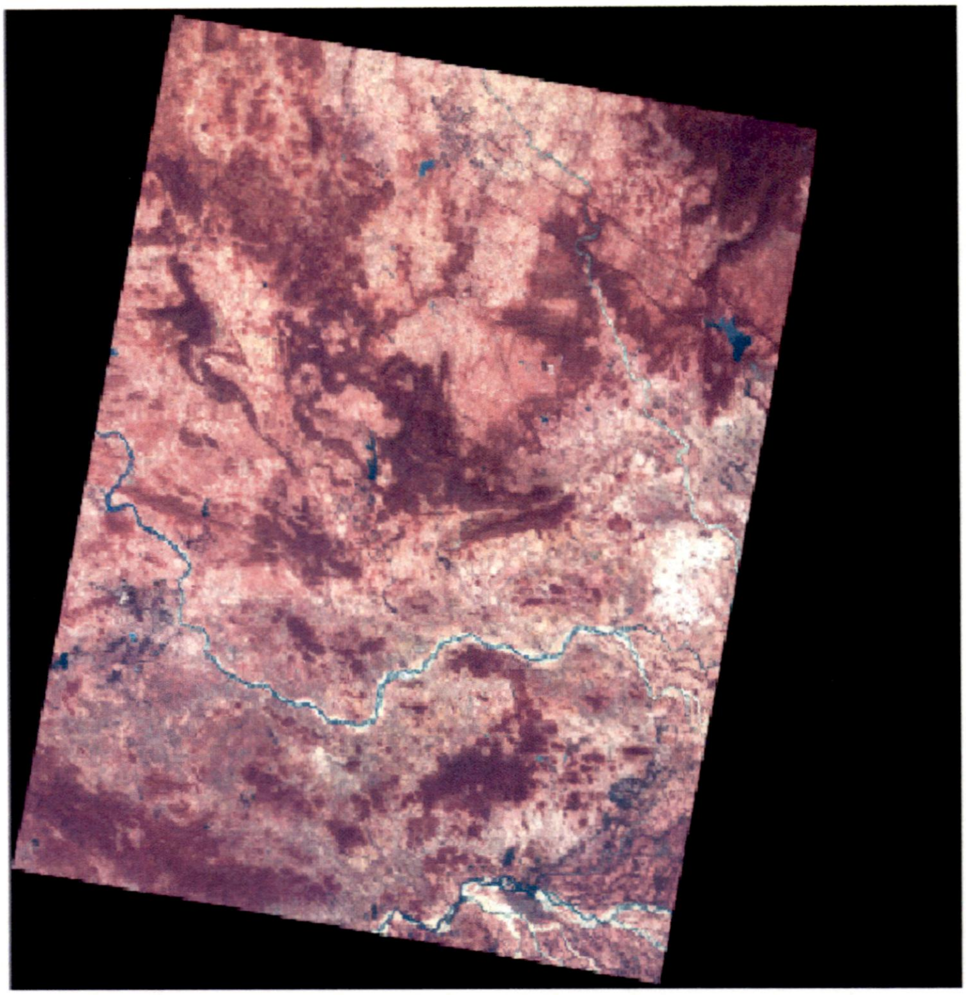


Fig 3.4 Geographically Registered Satellite Image of Brahmani River for 1998

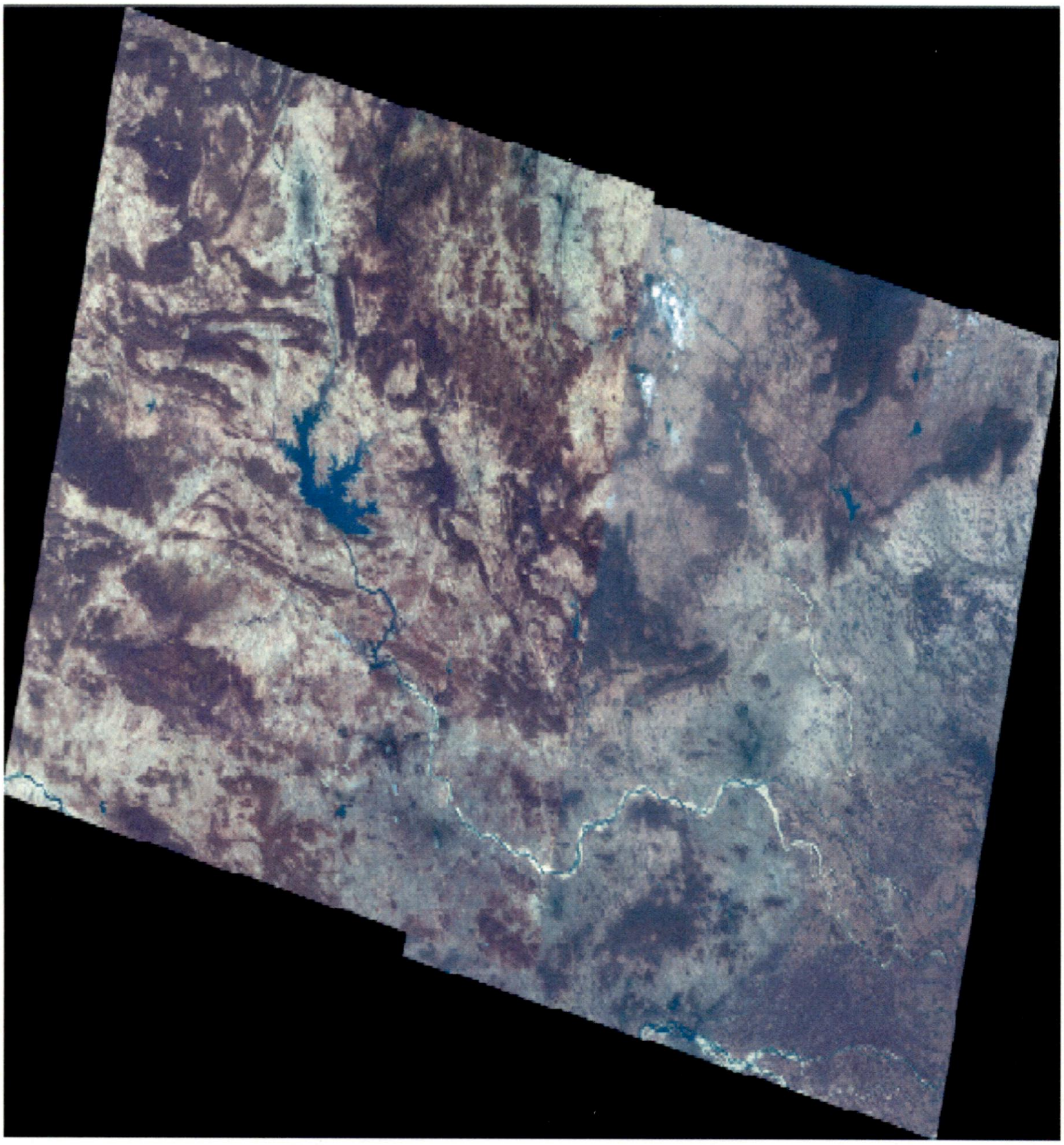


Fig 3.5 Geographically Registered and Mosaicked Image of Brahmani River for year 2009

The information digitized are in the form of small arcs and hence have to be converted into shape files for continuous and smooth shape, before any computations can be carried out. The digitization work with the help of ARC GIS is done. The shape files have been created. Following are the figures of the created shape files.

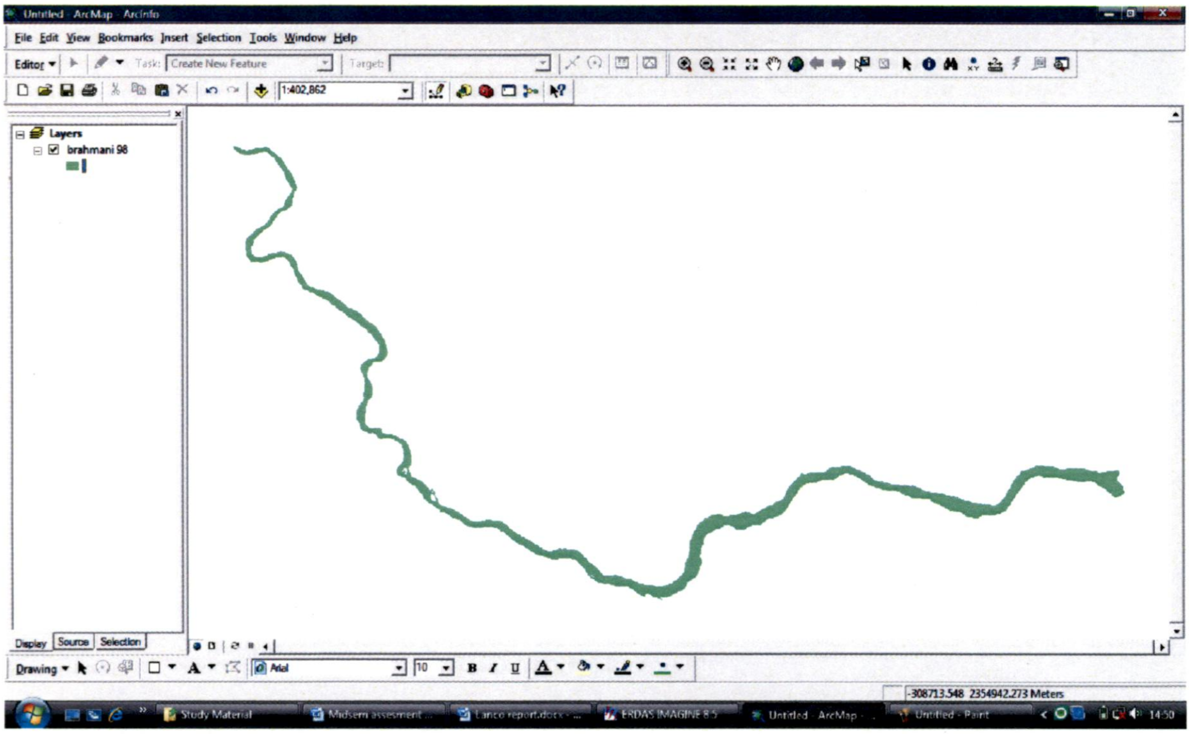


Fig 3.6 Shape file of Brahmani River for Year 1998

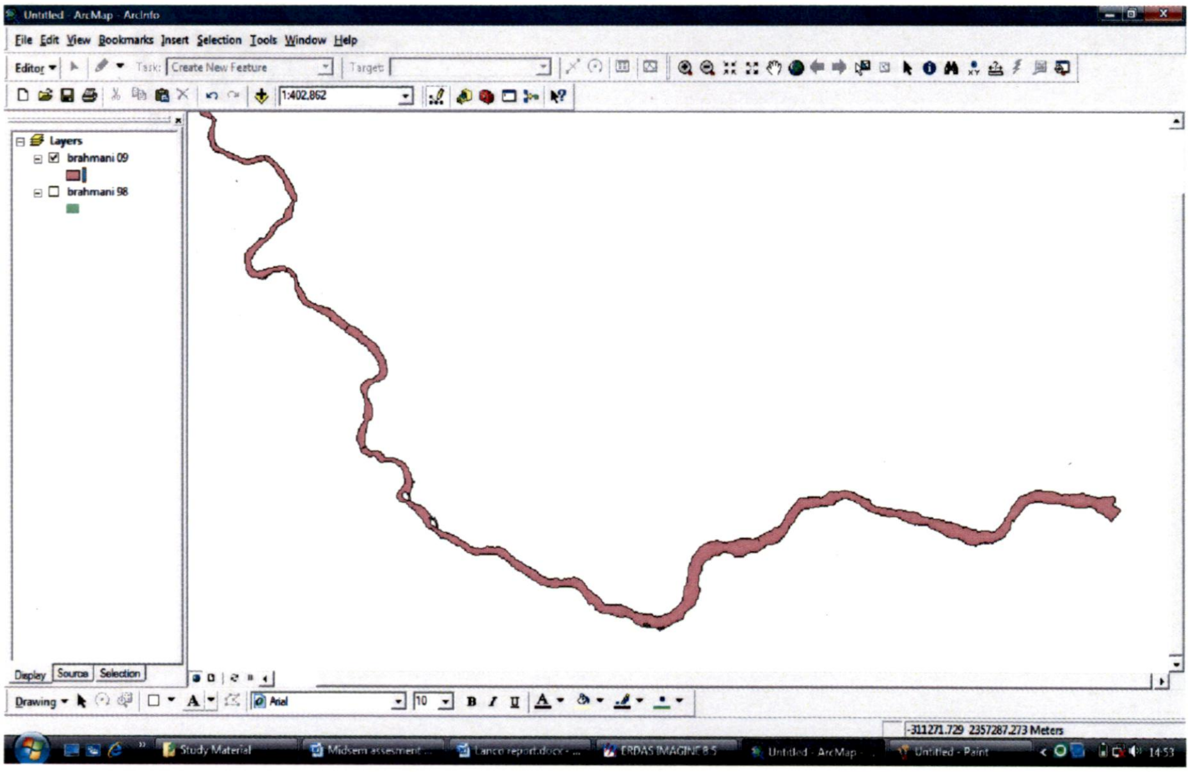


Fig 3.7 Shape file of Brahmani River for Year 2009

4.1 General

The design flood analysis was carried out to work out the design flood for the Brahmani River near Kharagprasad. The design flood and the low flow values were required for the computation of In-stream storage volume using HEC-RAS 4. The Samal barrage discharge data was available. This data can be used for the design flood and low flow calculation near Kharagprasad as neither any stream is joining to Brahmani River between this reach nor is any channel drifting. Moreover there were no other losses found between this reach. Therefore Hydrological Data available for Samal barrage had been adopted to carry out frequecny analysis for the design flood and low flow for Brahmani. The analysis was carried out for 18 years data available from year 1990 to year 2007. Following methods had been adopted for frequency analysis.

4.2 Flow Duration Curve

This method was used for the calculation of low flow for the computation of In-stream storage during low flow.

The 10 daily flow of Samal barrage was arranged in descending order to carry out the frequency analysis. The analysis carried out is shown in Table 4.1 Based on the calculations; the flow duration curve is plotted as given in Fig 4.1.

Table 4.1.: Frequency Analysis for Low Flow Calculation

Year	Month	Ten daily	Stream flow, cumec	Rank	Exceedence Probability(%)
2001	July	3	6104.55	1	0.158
1994	July	2	4692.05	2	0.315
1994	Aug	1	4662.78	3	0.473

2001	July	2	4294.00	4	0.631
2006	Aug	3	4223.64	5	0.789
1994	Aug	2	3990.50	6	0.946
1994	July	1	3758.08	7	1.104
1991	Sept	1	3645.82	8	1.262
1991	Aug	2	3609.39	9	1.420
1994	July	3	3588.15	10	1.577
1994	Aug	3	3261.64	11	1.735
1994	Sept	2	3207.86	12	1.893
1999	Aug	1	3187.90	13	2.050
1999	Sept	2	3084.40	14	2.208
2001	Sept	1	2800.30	15	2.366
1991	Aug	3	2559.40	16	2.524
1994	Sept	1	2530.60	17	2.681
1998	Sept	2	2480.20	18	2.839
2001	Aug	1	2453.10	19	2.997
2001	Aug	3	2434.18	20	3.155
1997	Aug	2	2292.20	21	3.312
2003	Sept	1	2181.20	22	3.470
1997	Aug	1	2095.80	23	3.628
2006	Aug	1	2080.80	24	3.785
1997	Sept	2	2033.00	25	3.943

1996	Aug	3	2019.91	26	4.101
1996	Aug	1	2010.00	27	4.259
1999	Sept	3	1928.30	28	4.416
2003	Oct	3	1884.09	29	4.574
2005	July	1	1812.57	30	4.732
1999	Sept	1	1763.20	31	4.890
1990	Aug	1	1751.40	32	5.047
2003	Oct	2	1636.20	33	5.205
2005	July	3	1631.91	34	5.363
1996	Aug	2	1625.00	35	5.521
1997	Sept	1	1613.60	36	5.678
2003	Sept	2	1594.70	37	5.836
1999	Aug	2	1591.90	38	5.994
1993	Sept	3	1586.23	39	6.151
1990	July	3	1585.71	40	6.309
2004	Aug	3	1566.09	41	6.467
1999	Oct	2	1560.10	42	6.625
1996	Jun	3	1546.50	43	6.782
1993	Aug	1	1509.16	44	6.940
1993	Oct	1	1496.22	45	7.098
2003	Oct	1	1481.30	46	7.256
2000	July	3	1471.91	47	7.413

1993	Sept	1	1448.16	48	7.571
1996	July	3	1437.55	49	7.729
1995	Sept	2	1404.10	50	7.886
1995	Sept	3	1387.80	51	8.044
1990	Oct	2	1359.60	52	8.202
2005	July	2	1359.02	53	8.360
2001	July	1	1344.10	54	8.517
2007	July	2	1321.10	55	8.675
2006	Sept	1	1311.10	56	8.833
2004	Aug	2	1308.50	57	8.991
1993	Sept	2	1294.08	58	9.148
1991	July	3	1281.10	59	9.306
1997	July	3	1273.27	60	9.464
2005	Aug	1	1247.00	61	9.621
1999	Oct	1	1236.70	62	9.779
1992	July	3	1236.18	63	9.937
1992	Aug	2	1230.96	64	10.095
1993	July	2	1224.73	65	10.252
1990	Sept	2	1217.47	66	10.410
2002	Sept	1	1182.90	67	10.568
1991	Sept	2	1166.64	68	10.726
1993	Aug	2	1164.09	69	10.883

2003	Aug	3	1159.18	70	11.041
2001	Aug	2	1157.60	71	11.199
1992	Sept	1	1152.67	72	11.356
1993	Aug	3	1139.56	73	11.514
1995	Aug	1	1120.20	74	11.672
1990	Oct	1	1113.13	75	11.830
1992	Aug	3	1105.03	76	11.987
1990	Sept	1	1061.20	77	12.145
1997	Aug	3	1056.64	78	12.303
1996	Sept	1	1052.60	79	12.461
1994	Sept	3	1030.02	80	12.618
1995	Aug	2	1025.40	81	12.776
2002	Sept	2	1025.20	82	12.934
1992	Sept	2	1019.29	83	13.091
1992	Aug	1	1016.62	84	13.249
1999	Oct	3	986.36	85	13.407
2007	July	3	968.55	86	13.565
1990	Aug	3	960.48	87	13.722
1990	July	2	954.66	88	13.880
1997	Sept	3	937.60	89	14.038
2000	Aug	3	935.82	90	14.196
1995	Sept	1	928.80	91	14.353

1995	July	3	923.45	92	14.511
1992	Sept	3	923.01	93	14.669
1993	Oct	2	915.03	94	14.826
1992	Oct	2	899.94	95	14.984
2004	Sept	3	892.10	96	15.142
1992	Oct	1	878.92	97	15.300
2003	Aug	2	872.60	98	15.457
2006	Sept	3	865.90	99	15.615
1991	Sept	3	831.28	100	15.773
1995	Aug	3	830.73	101	15.931
2007	July	1	829.00	102	16.088
2002	Aug	2	823.20	103	16.246
1996	July	1	821.70	104	16.404
2001	Sept	2	818.60	105	16.562
2000	Sept	1	812.00	106	16.719
1997	July	1	799.40	107	16.877
1999	July	3	795.18	108	17.035
1992	Oct	3	791.87	109	17.192
2001	Sept	3	783.40	110	17.350
1994	Jun	3	773.45	111	17.508
2002	Sept	3	771.10	112	17.666
1998	Oct	3	767.45	113	17.823

2004	Sept	1	766.60	114	17.981
1993	Oct	3	759.88	115	18.139
2003	Sept	3	759.60	116	18.297
2000	July	2	759.40	117	18.454
1990	Aug	2	759.18	118	18.612
2003	Jun	3	739.30	119	18.770
2003	July	3	736.27	120	18.927
2002	Aug	3	734.36	121	19.085
1992	Nov	1	726.35	122	19.243
2000	Sept	3	722.00	123	19.401
2000	Aug	2	716.60	124	19.558
1995	Oct	1	716.50	125	19.716
2006	Sept	2	714.30	126	19.874
2005	Aug	3	710.45	127	20.032
1997	Oct	1	704.80	128	20.189
1990	Nov	1	698.85	129	20.347
1990	Oct	3	684.63	130	20.505
1994	Oct	1	682.53	131	20.662
1997	July	2	681.80	132	20.820
1993	July	3	677.47	133	20.978
1991	Aug	1	672.74	134	21.136
2003	Aug	1	672.10	135	21.293

1993	July	1	661.56	136	21.451
1990	Sept	3	645.59	137	21.609
2005	Aug	2	645.50	138	21.767
2000	Sept	2	642.50	139	21.924
1996	Sept	2	641.80	140	22.082
1990	Dec	2	640.05	141	22.240
1998	Nov	1	634.10	142	22.397
2003	Nov	1	631.20	143	22.555
2006	July	3	630.82	144	22.713
2005	Sept	1	623.80	145	22.871
1991	Jan	1	614.34	146	23.028
2000	Aug	1	607.20	147	23.186
2007	Jun	3	600.05	148	23.344
2002	July	1	600.00	149	23.502
2004	Aug	1	599.70	150	23.659
2004	Sept	2	598.50	151	23.817
1991	Oct	2	597.94	152	23.975
1991	Oct	1	593.22	153	24.132
1999	Nov	1	580.40	154	24.290
1998	Feb	1	579.70	155	24.448
1998	Mar	1	579.70	155	24.448
1990	Dec	3	565.88	157	24.763

2006	Oct	2	563.60	158	24.921
2003	July	1	561.60	159	25.079
2003	July	2	553.00	160	25.237
1991	Oct	3	550.70	161	25.394
2006	Oct	1	548.20	162	25.552
1996	Sept	3	547.50	163	25.710
1991	Nov	1	547.14	164	25.868
1991	Jan	2	527.21	165	26.025
1998	July	2	525.70	166	26.183
2000	Oct	2	517.80	167	26.341
2006	Aug	2	514.85	168	26.498
1998	July	3	511.09	169	26.656
2003	Nov	2	511.00	170	26.814
1993	Jun	3	509.80	171	26.972
1990	Jun	1	500.79	172	27.129
1994	Oct	3	496.45	173	27.287
1999	July	2	496.00	174	27.445
1997	Oct	2	495.40	175	27.603
2004	Oct	1	493.70	176	27.760
1998	Feb	3	492.13	177	27.918
1998	Mar	3	492.13	177	27.918
1991	Jun	3	489.55	179	28.233

1990	Nov	2	480.36	180	28.391
2005	Sept	2	475.70	181	28.549
2002	Jun	3	475.60	182	28.707
1990	Nov	3	474.71	183	28.864
1991	Jan	3	466.84	184	29.022
1990	May	3	464.74	185	29.180
1998	July	1	459.80	186	29.338
1998	Aug	3	457.82	187	29.495
2003	Nov	3	457.55	188	29.653
2000	Oct	1	456.70	189	29.811
1996	July	2	456.00	190	29.968
1999	Aug	3	449.64	191	30.126
1999	July	1	447.30	192	30.284
1991	Nov	2	446.61	193	30.442
1992	Jan	2	443.25	194	30.599
2005	Oct	3	441.41	195	30.757
1990	Dec	1	436.38	196	30.915
2002	Oct	1	435.80	197	31.073
2003	Apr	3	432.90	198	31.230
1993	Nov	1	427.23	199	31.388
2005	Sept	3	427.20	200	31.546
1990	Jun	2	424.09	201	31.703

2007	Apr	2	418.60	202	31.861
1998	Nov	2	416.60	203	32.019
1995	July	2	411.60	204	32.177
2003	Dec	2	410.60	205	32.334
2006	Apr	1	409.90	206	32.492
1995	Oct	2	404.30	207	32.650
2005	Oct	2	403.10	208	32.808
1998	Aug	1	402.50	209	32.965
2003	May	2	397.25	210	33.123
1998	Aug	2	395.00	211	33.281
2002	Apr	1	394.40	212	33.438
2007	Jun	2	391.95	213	33.596
1998	Sept	3	389.90	214	33.754
1998	May	3	389.55	215	33.912
2006	Jun	1	389.25	216	34.069
2002	Mar	3	386.91	217	34.227
2002	Mar	2	382.80	218	34.385
2000	Jun	3	381.50	219	34.543
2003	May	1	378.15	220	34.700
1994	Jan	1	374.06	221	34.858
1992	Jun	3	371.36	222	35.016
2001	Oct	3	364.64	223	35.174

2002	July	2	364.60	224	35.331
2004	July	2	364.50	225	35.489
1994	Nov	1	361.40	226	35.647
1992	Jan	3	360.78	227	35.804
1990	Jun	3	358.13	228	35.962
1998	Feb	2	354.10	229	36.120
1998	Mar	2	354.10	229	36.120
1994	Nov	2	353.50	231	36.435
2001	Oct	2	350.60	232	36.593
2003	Dec	1	350.60	232	36.593
2001	Oct	1	349.50	234	36.909
2003	Dec	3	347.64	235	37.066
1994	Oct	2	344.88	236	37.224
2004	Nov	1	338.90	237	37.382
2000	Mar	3	335.64	238	37.539
1993	Dec	1	334.97	239	37.697
2004	July	1	334.70	240	37.855
1996	Apr	2	333.30	241	38.013
1992	July	2	332.01	242	38.170
1992	Dec	2	331.43	243	38.328
1997	Oct	3	325.45	244	38.486
2005	Oct	1	325.40	245	38.644

1993	Dec	2	324.97	246	38.801
2005	Jun	3	323.15	247	38.959
1993	Nov	2	318.51	248	39.117
1994	Dec	3	318.45	249	39.274
2001	Jun	3	317.85	250	39.432
2005	Mar	3	317.00	251	39.590
1998	Apr	1	314.40	252	39.748
1998	Oct	2	313.60	253	39.905
1998	Sept	1	312.20	254	40.063
1998	Jun	1	311.20	255	40.221
1997	Jun	3	310.55	256	40.379
1992	Dec	3	309.20	257	40.536
1995	Nov	1	308.95	258	40.694
1997	Nov	2	307.90	259	40.852
2004	Jan	1	306.90	260	41.009
1994	Jun	2	306.29	261	41.167
1998	Apr	2	306.20	262	41.325
2003	Jun	2	305.40	263	41.483
1991	Dec	2	304.71	264	41.640
1993	Dec	3	304.44	265	41.798
1998	Nov	3	303.20	266	41.956
1995	May	2	302.20	267	42.114

2004	Oct	2	300.90	268	42.271
2000	May	1	298.50	269	42.429
1997	Nov	1	297.50	270	42.587
1991	Dec	3	296.08	271	42.744
1990	May	2	295.42	272	42.902
2001	Jun	2	295.10	273	43.060
1991	Nov	3	294.81	274	43.218
1996	Oct	1	291.60	275	43.375
1993	Nov	3	291.11	276	43.533
1991	Dec	1	289.06	277	43.691
1990	Jan	3	285.65	278	43.849
1998	Jun	3	282.40	279	44.006
2005	Mar	2	281.40	280	44.164
1992	Nov	2	279.69	281	44.322
1992	Nov	3	279.31	282	44.479
2002	Oct	3	277.91	283	44.637
2000	May	3	277.55	284	44.795
1990	July	1	277.17	285	44.953
1990	May	1	276.51	286	45.110
1998	Jun	2	275.80	287	45.268
1995	Feb	1	275.70	288	45.426
1995	Jan	2	275.50	289	45.584

1990	Feb	1	273.83	290	45.741
1993	Jan	1	273.74	291	45.899
2004	Oct	3	271.82	292	46.057
2001	Nov	3	269.70	293	46.215
1990	Apr	3	268.86	294	46.372
1999	Jun	3	268.70	295	46.530
1995	Jan	1	268.60	296	46.688
1994	Mar	2	267.98	297	46.845
2001	Nov	1	267.50	298	47.003
1998	May	2	266.50	299	47.161
1992	Dec	1	266.28	300	47.319
1990	Jan	2	266.24	301	47.476
1995	Mar	1	266.10	302	47.634
2001	Nov	2	265.90	303	47.792
2003	Jun	1	265.85	304	47.950
1994	Feb	3	265.73	305	48.107
1990	Apr	2	263.53	306	48.265
1998	May	1	262.60	307	48.423
2003	Mar	2	262.50	308	48.580
1992	Jan	1	260.90	309	48.738
1994	Jan	2	260.12	310	48.896
2006	May	3	259.73	311	49.054

2000	Apr	3	257.50	312	49.211
2001	Dec	2	256.40	313	49.369
1991	Jun	2	256.15	314	49.527
1990	Apr	1	255.26	315	49.685
1993	Jan	2	253.94	316	49.842
1994	Nov	3	253.90	317	50.000
1995	Oct	3	253.24	318	50.158
2005	Jan	1	252.30	319	50.315
2002	Mar	1	252.10	320	50.473
2006	Jan	1	251.80	321	50.631
1994	Dec	2	249.60	322	50.789
2004	May	2	249.30	323	50.946
2003	Apr	2	248.95	324	51.104
2006	July	2	248.95	324	51.104
2005	Jan	2	247.40	326	51.420
1998	Oct	1	246.60	327	51.577
1994	Feb	1	246.60	328	51.735
2006	Jun	2	246.30	329	51.893
2004	May	3	245.55	330	52.050
1994	Mar	1	244.74	331	52.208
1995	Mar	3	243.55	332	52.366
1995	July	1	242.20	333	52.524

1996	Oct	2	242.20	333	52.524
1994	Dec	1	241.50	335	52.839
1995	Apr	1	240.80	336	52.997
1991	Feb	1	238.91	337	53.155
2000	May	2	237.80	338	53.312
2005	May	1	237.30	339	53.470
1995	Feb	3	236.88	340	53.628
1994	Apr	1	236.23	341	53.785
1990	Jan	1	233.29	342	53.943
2005	Dec	2	233.10	343	54.101
2006	Nov	1	232.55	344	54.259
1996	May	1	231.40	345	54.416
2005	Feb	3	231.13	346	54.574
1999	Dec	2	228.20	347	54.732
2005	Jan	3	228.00	348	54.890
1994	Mar	3	227.30	349	55.047
1991	May	2	227.17	350	55.205
1999	Nov	2	227.10	351	55.363
1996	May	2	225.60	352	55.521
1999	Feb	1	224.50	353	55.678
1991	July	2	224.46	354	55.836
2001	Jun	1	224.40	355	55.994

1999	Jun	2	223.75	356	56.151
2002	Feb	1	222.50	357	56.309
1994	Apr	2	220.15	358	56.467
1995	Mar	2	219.80	359	56.625
1995	Nov	2	219.30	360	56.782
1991	Feb	2	218.02	361	56.940
1997	Jun	2	217.90	362	57.098
2004	July	3	217.36	363	57.256
2004	Nov	2	216.50	364	57.413
1992	Feb	3	215.77	365	57.571
1999	Jan	1	215.20	366	57.729
1995	Jan	3	214.09	367	57.886
2004	Jan	2	211.90	368	58.044
2007	Apr	1	211.60	369	58.202
2004	Jun	1	211.30	370	58.360
1995	Nov	3	210.90	371	58.517
2006	Feb	3	210.31	372	58.675
1997	Nov	3	208.70	373	58.833
2005	May	2	208.50	374	58.991
2005	Dec	1	208.00	375	59.148
2005	Nov	2	206.55	376	59.306
2005	Mar	1	204.70	377	59.464

2002	Feb	3	204.00	378	59.621
1992	Feb	1	203.35	379	59.779
2003	Mar	3	202.32	380	59.937
1998	Apr	3	202.30	381	60.095
1997	Dec	2	202.20	382	60.252
1998	Jan	1	201.10	383	60.410
1997	Dec	1	200.80	384	60.568
2004	Nov	3	200.60	385	60.726
1999	May	3	198.82	386	60.883
1997	Mar	2	197.00	387	61.041
2006	Nov	2	195.20	388	61.199
1997	May	3	194.91	389	61.356
1994	Feb	2	194.58	390	61.514
1996	Apr	1	193.00	391	61.672
2000	Mar	2	192.50	392	61.830
1999	Jan	2	192.10	393	61.987
2000	July	1	191.70	394	62.145
2000	Oct	3	191.68	395	62.303
1996	Jun	2	191.50	396	62.461
2002	Apr	2	191.50	396	62.461
2005	Apr	2	189.80	398	62.776
1999	Nov	3	189.60	399	62.934

1991	Jun	1	189.21	400	63.091
2004	Dec	1	188.30	401	63.249
1992	Mar	1	188.15	402	63.407
1995	Jun	1	187.40	403	63.565
1999	Jan	3	187.18	404	63.722
2005	Apr	1	186.70	405	63.880
2000	Jun	1	186.20	406	64.038
2005	Nov	1	185.90	407	64.196
2002	Feb	2	185.60	408	64.353
1997	Jan	2	185.50	409	64.511
1999	May	1	185.40	410	64.669
1993	Jan	3	184.87	411	64.826
1996	Mar	3	184.45	412	64.984
2004	May	1	182.50	413	65.142
2003	Apr	1	182.00	414	65.300
2002	Jan	3	181.82	415	65.457
2002	Nov	2	181.40	416	65.615
1999	Dec	1	180.80	417	65.773
1997	Dec	3	180.00	418	65.931
2002	Jan	2	179.60	419	66.088
2004	Feb	2	179.45	420	66.246
2000	Nov	1	179.10	421	66.404

2000	Jun	2	179.00	422	66.562
1998	Dec	2	178.70	423	66.719
1997	Jan	3	178.55	424	66.877
2005	Feb	2	178.25	425	67.035
2005	Dec	3	178.23	426	67.192
2004	Feb	1	177.20	427	67.350
2004	Mar	2	177.15	428	67.508
1997	Feb	1	176.70	429	67.666
2002	Nov	3	176.60	430	67.823
2001	Dec	1	176.30	431	67.981
1998	Dec	1	175.90	432	68.139
2006	Nov	3	175.85	433	68.297
2002	Apr	3	175.70	434	68.454
1995	May	3	175.36	435	68.612
1996	May	3	173.45	436	68.770
2002	Jan	1	173.00	437	68.927
1999	May	2	172.90	438	69.085
2002	Nov	1	172.80	439	69.243
2005	Apr	3	172.10	440	69.401
1999	Apr	3	172.00	441	69.558
2005	May	3	172.00	441	69.558
2002	July	3	170.95	443	69.874

2002	Aug	1	169.50	444	70.032
1997	Jan	1	169.40	445	70.189
2002	Jun	2	168.95	446	70.347
2000	Apr	2	168.90	447	70.505
2004	Jan	3	168.41	448	70.662
2000	Feb	1	167.20	449	70.820
2004	Jun	2	166.95	450	70.978
1995	Apr	3	166.30	451	71.136
2002	May	1	166.10	452	71.293
1996	Jan	1	164.80	453	71.451
2006	Jan	2	164.05	454	71.609
1996	Apr	3	163.90	455	71.767
2002	May	2	163.75	456	71.924
2005	Jun	1	163.50	457	72.082
1999	Dec	3	163.00	458	72.240
2005	Feb	1	162.20	459	72.397
2001	Dec	3	161.82	460	72.555
1991	May	1	161.39	461	72.713
1999	Jun	1	160.25	462	72.871
2007	Jan	2	159.90	463	73.028
1991	Feb	3	159.74	464	73.186
2004	Dec	2	159.60	465	73.344

2003	May	3	159.59	466	73.502
1997	Apr	1	159.20	467	73.659
1997	Jun	1	158.10	468	73.817
2003	Mar	1	157.40	469	73.975
1997	Mar	3	156.18	470	74.132
2005	Nov	3	155.50	471	74.290
2000	Apr	1	155.20	472	74.448
2007	Mar	3	154.36	473	74.606
1995	Feb	2	154.20	474	74.763
2006	Feb	2	153.65	475	74.921
1996	Dec	2	152.60	476	75.079
1995	Dec	2	150.80	477	75.237
2000	Jan	2	150.40	478	75.394
2006	Jan	3	149.45	479	75.552
2000	Jan	1	148.30	480	75.710
1991	May	3	147.41	481	75.868
1996	Oct	3	147.36	482	76.025
2006	Oct	3	147.09	483	76.183
1995	Apr	2	147.00	484	76.341
2006	Dec	1	146.35	485	76.498
1996	Feb	2	145.60	486	76.656
2000	Mar	1	145.50	487	76.814

2007	Jun	1	144.75	488	76.972
2004	Apr	3	144.70	489	77.129
2004	Mar	1	144.65	490	77.287
2007	May	2	144.65	490	77.287
1998	Jan	3	143.91	492	77.603
2002	Dec	1	143.90	493	77.760
1998	Dec	3	142.91	494	77.918
2003	Feb	1	142.15	495	78.076
1995	May	1	142.10	496	78.233
1991	July	1	141.92	497	78.391
2004	Apr	2	140.90	498	78.549
1996	Dec	3	140.36	499	78.707
1995	Dec	1	139.80	500	78.864
1993	Feb	1	139.69	501	79.022
2000	Feb	3	139.56	502	79.180
2002	Jun	1	139.10	503	79.338
2005	Dec	3	138.82	504	79.495
1999	Apr	1	138.40	505	79.653
1999	Mar	2	137.80	506	79.811
1999	Mar	3	137.64	507	79.968
1995	Dec	3	137.55	508	80.126
1999	Mar	1	137.50	509	80.284

2005	Jun	2	137.10	510	80.442
1999	Feb	2	137.00	511	80.599
1999	Feb	3	137.00	511	80.599
2000	Dec	3	136.09	513	80.915
2001	May	3	135.09	514	81.073
1998	Jan	2	134.80	515	81.230
1997	May	2	134.60	516	81.388
1991	Apr	3	133.67	517	81.546
1994	Jan	3	133.22	518	81.703
2007	May	1	132.95	519	81.861
2000	Jan	3	132.55	520	82.019
2006	May	1	132.40	521	82.177
2002	Dec	2	132.15	522	82.334
1996	Dec	1	130.90	523	82.492
2007	Mar	1	130.85	524	82.650
1996	Feb	1	130.60	525	82.808
1990	Feb	2	129.42	526	82.965
1996	Nov	3	129.20	527	83.123
2002	Dec	3	128.95	528	83.281
2006	Feb	1	127.70	529	83.438
2006	Dec	3	126.82	530	83.596
2006	Mar	3	126.77	531	83.754

1994	May	1	126.52	532	83.912
1996	Jan	2	125.70	533	84.069
1997	May	1	124.60	534	84.227
2000	Dec	2	124.50	535	84.385
1994	Apr	3	124.29	536	84.543
2000	Feb	2	124.20	537	84.700
1997	Apr	3	123.30	538	84.858
2006	Apr	2	122.10	539	85.016
2006	Mar	2	121.90	540	85.174
1996	Nov	1	121.00	541	85.331
1999	Apr	2	120.50	542	85.489
1997	Apr	2	120.40	543	85.647
1996	Nov	2	120.10	544	85.804
2006	May	2	119.30	545	85.962
2000	Nov	2	119.10	546	86.120
2007	Mar	2	118.50	547	86.278
2002	May	3	117.82	548	86.435
1996	Mar	1	117.70	549	86.593
1996	Mar	2	117.50	550	86.751
2001	Jan	1	117.45	551	86.909
2001	Mar	1	117.45	551	86.909
1991	Apr	2	117.41	553	87.224

1996	Feb	3	117.22	554	87.382
2004	Feb	3	114.94	555	87.539
2006	Dec	2	114.70	556	87.697
2001	Feb	1	111.65	557	87.855
1991	Mar	1	110.89	558	88.013
1991	Mar	2	109.34	559	88.170
1991	Mar	3	109.34	559	88.170
1991	Apr	1	109.34	559	88.170
2003	Feb	3	108.81	562	88.644
1992	Mar	3	108.54	563	88.801
1995	Jun	3	108.30	564	88.959
2004	Jun	3	108.00	565	89.117
2006	Apr	3	107.80	566	89.274
1996	Jan	3	106.82	567	89.432
2002	Oct	2	105.55	568	89.590
1997	Mar	1	105.50	569	89.748
1994	Jun	1	104.06	570	89.905
2006	July	1	104.00	571	90.063
2007	Apr	3	103.30	572	90.221
2003	Jan	1	102.50	573	90.379
2003	Feb	2	102.25	574	90.536
2003	Jan	2	101.70	575	90.694

1992	Feb	2	99.27	576	90.852
1997	Feb	3	99.25	577	91.009
1995	Jun	2	98.40	578	91.167
1992	Mar	2	97.14	579	91.325
2001	Apr	1	95.75	580	91.483
1996	Jun	1	95.60	581	91.640
2007	Jan	3	95.50	582	91.798
1992	Apr	1	94.88	583	91.956
1997	Feb	2	94.30	584	92.114
2001	Jan	3	93.91	585	92.271
2001	Mar	3	93.91	585	92.271
2007	Jan	1	93.90	587	92.587
2007	Feb	3	91.75	588	92.744
1992	Jun	2	90.72	589	92.902
2007	Feb	2	90.20	590	93.060
2000	Nov	3	89.80	591	93.218
1992	Apr	2	89.22	592	93.375
2007	Feb	1	88.45	593	93.533
2006	Mar	1	87.95	594	93.691
2001	Jan	2	86.65	595	93.849
2001	Mar	2	86.65	595	93.849
2004	Apr	1	84.50	597	94.164

1993	Feb	2	83.68	598	94.322
2003	Jan	3	83.45	599	94.479
2001	May	2	81.35	600	94.637
1993	Jun	2	80.86	601	94.795
2007	May	3	80.68	602	94.953
2000	Dec	1	80.65	603	95.110
2001	Apr	2	80.00	604	95.268
2006	Jun	3	79.50	605	95.426
1993	Feb	3	78.87	606	95.584
1993	Mar	1	78.87	606	95.584
1993	Mar	2	77.78	608	95.899
2001	Feb	2	77.20	609	96.057
2001	May	1	77.05	610	96.215
2001	Apr	3	76.75	611	96.372
1990	Mar	3	71.87	612	96.530
2001	Feb	3	70.44	613	96.688
1990	Mar	2	69.84	614	96.845
2004	Mar	3	69.82	615	97.003
1992	July	1	69.75	616	97.161
1992	Apr	3	55.52	617	97.319
1992	May	1	54.14	618	97.476
1992	May	3	53.72	619	97.634

1992	Jun	1	51.54	620	97.792
1992	May	2	50.33	621	97.950
1993	Mar	3	43.45	622	98.107
1990	Mar	1	41.62	623	98.265
1990	Feb	3	39.12	624	98.423
1994	May	2	33.29	625	98.580
1993	May	3	29.95	626	98.738
1993	Apr	1	29.11	627	98.896
1993	May	2	28.13	628	99.054
1993	Apr	3	27.76	629	99.211
1993	Apr	2	27.55	630	99.369
1993	Jun	1	25.91	631	99.527
1993	May	1	25.88	632	99.685
1994	May	3	25.39	633	99.842
			Number of record (n)	**633**	

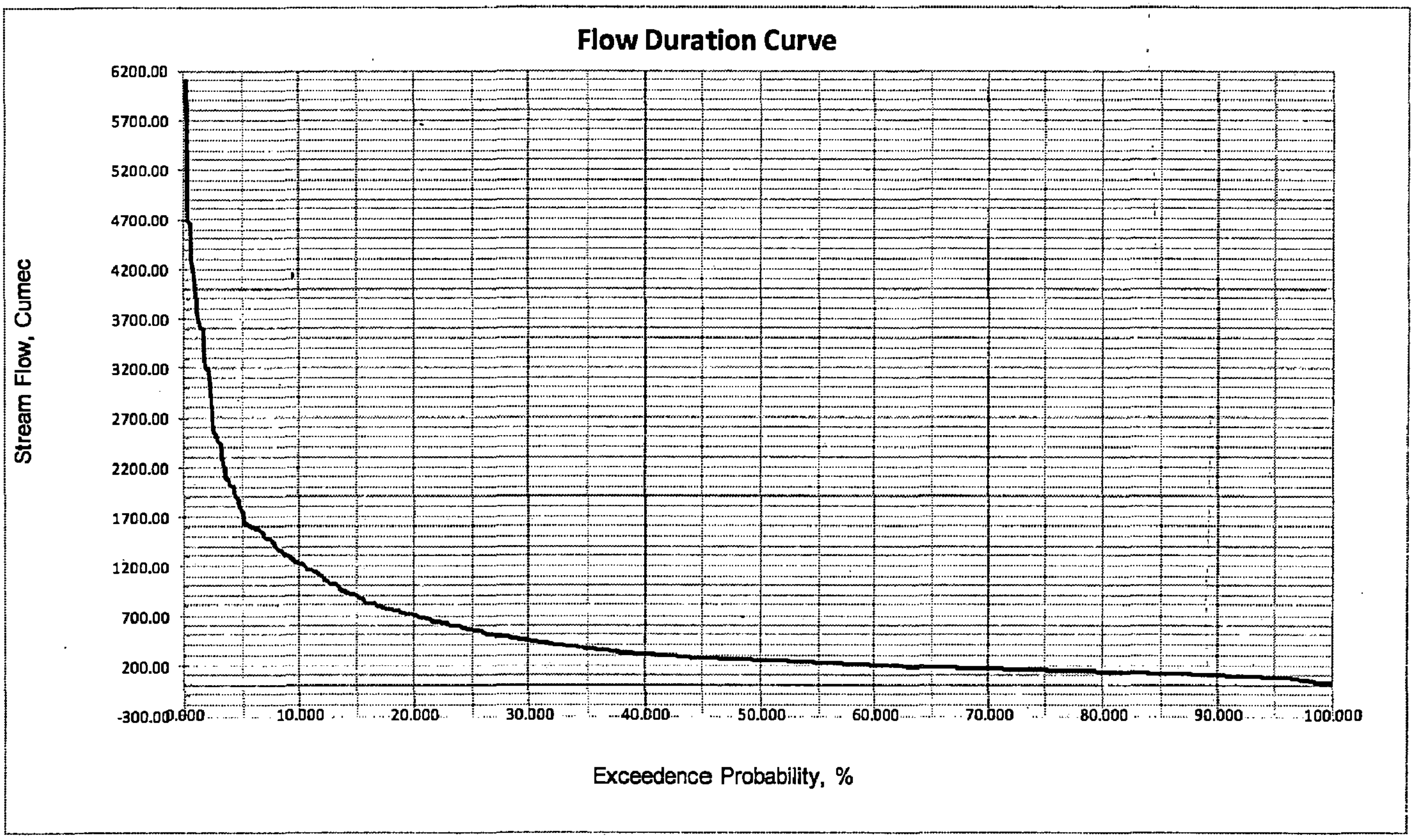


Fig. 4.1 Flow Duration Curve for 10 daily Inflow in Samal Barrage

4.3 Log Normal Distribution Method

The procedure to calculate design discharge by using Log Normal distribution method is as Follows:

Table 4.2: Maximum Yearly Flow and Antilog

Year	Maximum yearly discharge, Q, cumec	z=log Q
1990	1751.40	3.24
1991	3645.82	3.56

1992	1236.18	3.09
1993	1586.23	3.20
1994	4692.05	3.67
1995	1404.10	3.15
1996	2019.91	3.31
1997	2292.20	3.36
1998	2480.20	3.39
1999	3187.90	3.50
2000	1471.91	3.17
2001	6104.55	3.79
2002	1182.90	3.07
2003	2181.20	3.34
2004	1566.09	3.19
2005	1812.57	3.26
2006	4223.64	3.63
2007	1321.10	3.12

1. The mean value of variate Z, worked out from the Table 4.2 is

$$\overline{Z} = \frac{\sum Z}{N} = 3.34 \text{ where N= No of years.}$$

2. The standard deviation of variate Z is

$$\sigma_Z = \sqrt{\frac{\sum (Z-\overline{Z})^2}{(N-1)}} = 0.213417$$

3. Logarithm of Maximum discharge Q for return period T,

$$Z_T = \overline{Z} + K_Z \times \sigma_z$$

4. The values of K_Z for different return period taken from the table 4.3 for Cs=0 and the corresponding values of Z_T are calculated and shown in the Table 4.4.

Table 4.3: K_z for Use in Log Normal and Log-Pearson Type III Distribution

Coefficient of skew, C_s	**Recurrence interval T is years**						
	2	**10**	**25**	**50**	**100**	**200**	**1000**
3.0	-0.396	1.180	2.278	3.152	40.51	4.970	7.250
2.5	-0.360	1.250	2.262	3.048	3.845	4.652	6.600
2.2	-0.330	1.284	2.240	2.970	3.705	4.444	6.200
2.0	-0.307	1.302	2.219	2.912	3.605	4.298	5.910
1.8	-0.282	1.318	2.193	2.848	3.499	4.147	5.660
1.6	-0.254	1.329	2.163	2.780	3.388	3.990	5.390
1.4	-0.225	1.337	2.128	2.706	3.271	3.828	5.110
1.2	-0.195	1.340	2.087	2.626	3.149	3.661	4.820
1.0	-0.164	1.340	2.043	2.542	3.022	3.489	4.540
0.9	-0.148	1.339	2.018	2.498	2.957	3.401	4.395
0.8	-0.132	1.336	1.998	2.453	2.891	3.312	4.250
0.7	-0.116	1.333	1.967	2.407	2.824	3.223	4.105

0.6	-0.099	1.328	1.939	2.359	2.755	3.132	3.960
0.5	-0.083	1.323	1.910	2.311	2.686	3.041	3.815
0.4	-0.066	1.317	1.880	2.261	2.615	2.949	3.670
0.3	-0.050	1.309	1.849	2.211	2.544	2.856	3.525
0.2	-0.033	1.301	1.818	2.159	2.472	2.763	3.380
0.1	-0.017	1.292	1.785	2.107	2.400	2.670	3.235
0.0	**0.000**	**1.282**	**1.751**	**2.054**	**2.326**	**2.576**	**3.090**
-0.1	0.017	1.270	1.716	2.000	2.252	2.482	2.950
-0.2	0.033	1.258	1.680	1.945	2.178	2.388	2.810
-0.3	0.050	1.245	1.643	1.890	2.104	2.294	2.675
-0.4	0.066	1.231	1.606	1.834	2.029	2.201	2.540
-0.5	0.083	1.216	1.567	1.777	1.955	2.108	2.400
-0.6	0.099	1.200	1.528	1.720	1.880	2.016	2.275
-0.7	0.116	1.183	1.488	1.663	1.806	1.926	2.150
-0.8	0.132	1.166	1.448	1.606	1.733	1.837	2.035
-0.9	0.148	1.147	1.407	1.549	1.660	1.749	1.910
-1.0	0.164	1.128	1.366	1.492	1.588	1.664	1.880
-1.4	0.225	1.041	1.198	1.270	1.318	1.351	1.465
-1.8	0.282	0.945	1.035	1.069	1.087	1.097	1.130
-2.2	0.330	0.844	0.888	0.900	0.905	0.907	0.910
-3.2	0.396	0.660	0.666	0.666	0.667	0.667	0.668

5. The design flood value Q for the various return periods are also calculated in result table where $Q = anti\log Z_T$.

Table 4.4: Calculation for Log Normal Distribution Method

T, years	Kz	Kz x z	$\overline{Z}$	Z_T	Q, cumec
2	0.000	0	3.34	3.34	2166.837856
10	1.282	0.273600594	3.34	3.61	4068.431502
25	1.751	0.373693167	3.34	3.71	5122.943679
50	2.054	0.438358518	3.34	3.77	5945.452755
100	2.326	0.496407942	3.34	3.83	6795.702471
200	2.576	0.549762192	3.34	3.89	7684.022119
1000	3.090	0.65945853	3.34	4.00	9892.019057

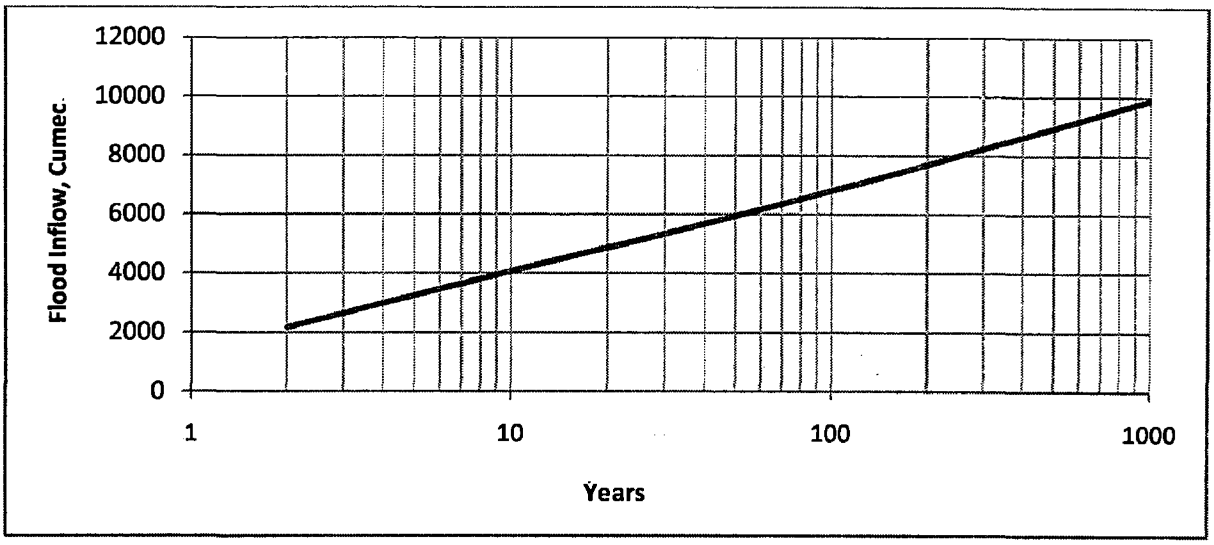


Fig. 4.2 Log Normal Distribution Method for 10 Daily Inflow in Samal Barrage

4.4 Log-Pearson Type III Distribution Method

Table 4.5: Maximum Yearly Flow and Antilog

Year	Maximum yearly discharge, Q, cumec	z=log Q
1990	1751.40	3.24
1991	3645.82	3.56
1992	1236.18	3.09
1993	1586.23	3.20
1994	4692.05	3.67
1995	1404.10	3.15
1996	2019.91	3.31
1997	2292.20	3.36
1998	2480.20	3.39
1999	3187.90	3.50
2000	1471.91	3.17
2001	6104.55	3.79
2002	1182.90	3.07
2003	2181.20	3.34
2004	1566.09	3.19
2005	1812.57	3.26
2006	4223.64	3.63
2007	1321.10	3.12

The calculation procedure by using Log Pearson Type III distribution method is as follows:

1. The mean value of variate Z, worked out from the Table 4.5 is

$$\bar{Z} = \frac{\sum Z}{N} = 3.34 \text{ where N= No of years.}$$

2. The standard deviation of variate Z is

$$\sigma_Z = \sqrt{\frac{\sum (Z-\bar{Z})^2}{(N-1)}} = 0.213417$$

3. The skewness coefficient for Z, $C_S = \dfrac{N\sum (Z-\bar{Z})^3}{(N-1)(N-2)(\sigma_Z)^3}$ =0.033813

4. Logarithm of Maximum discharge Q for return period T,

$$Z_T = \bar{Z} + K_Z \times \sigma_z$$

5. The values of K_Z for different return period taken from the Table 4.3 and the corresponding values of Z_T are calculated and shown in the Table 4.6.
6. The design flood value Q for the various return periods are also calculated in result Table 4.6 where $Q = anti\log Z_T$.

Table 4.6: Calculation for Log Pearson Type III distribution

T, years	K_Z	$K_Z \times \sigma_Z$	$\bar{Z} + K_Z \times \sigma_Z$	**Q=Antilog Z, cumec**
2	0.005748	0.001226766	3.334599648	2160.725752
10	1.285381	0.274322376	3.610148791	4075.198716
25	1.762496	0.376146912	3.711973326	5151.970007

50	2.071921	0.442183391	3.778009806	5998.046187
100	2.351022	0.501748266	3.83757468	6879.782047
200	2.607784	0.5565458	3.892372215	7804.987552
1000	3.139029	0.6699225	4.005748914	10133.25366

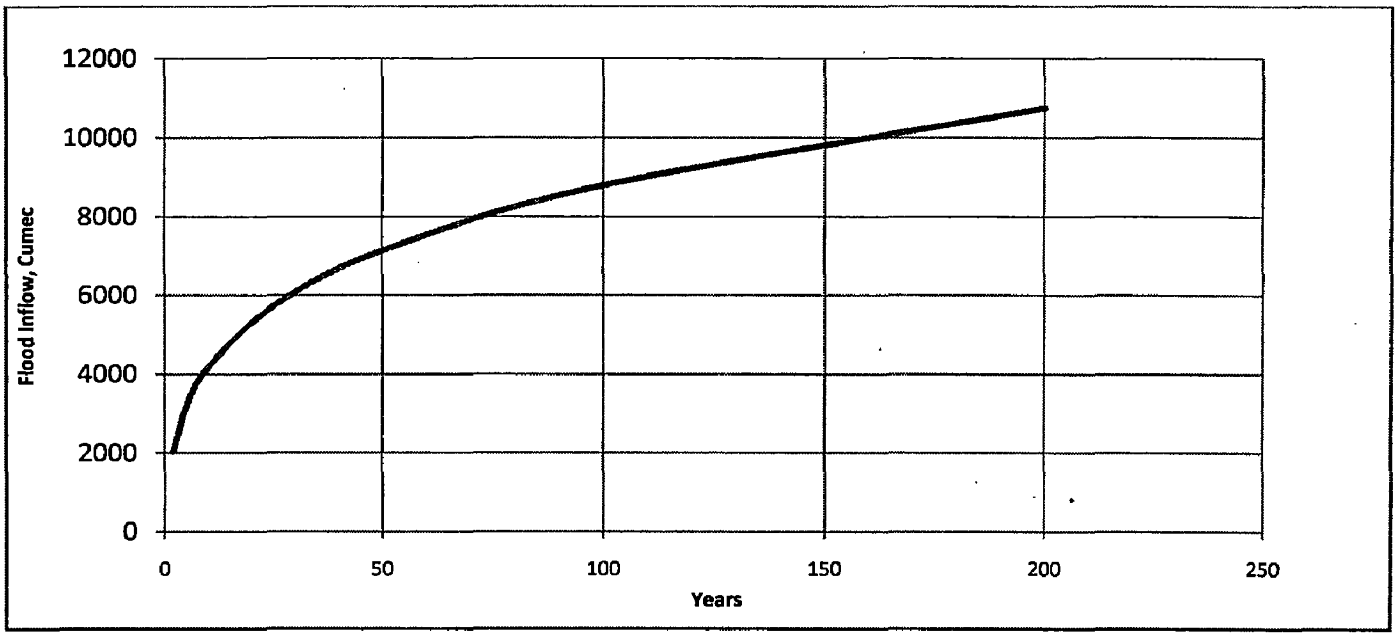


Fig. 4.3 Log Pearson III Distribution Method for 10 Daily Inflow in Samal Barrage

5.1 DESIGN OF PIANO KEY WEIR

5.1.1 General

This technology helps to reduce the reservoir sedimentation. It will also help in minimising the sub-mergence effect. About 52cm of submergence depth reduction can be achieved if Piano Key Weir is adopted instead of conventional barrage. The three dimensional view of generalized Piano Key Weir shape is shown in Fig. 5.1.

In-steam water storage will be created without submergence in the river channel cross section to meet low flow demands of the industry, drinking water and navigation. Besides, the system will not have any operation and maintenance costs.

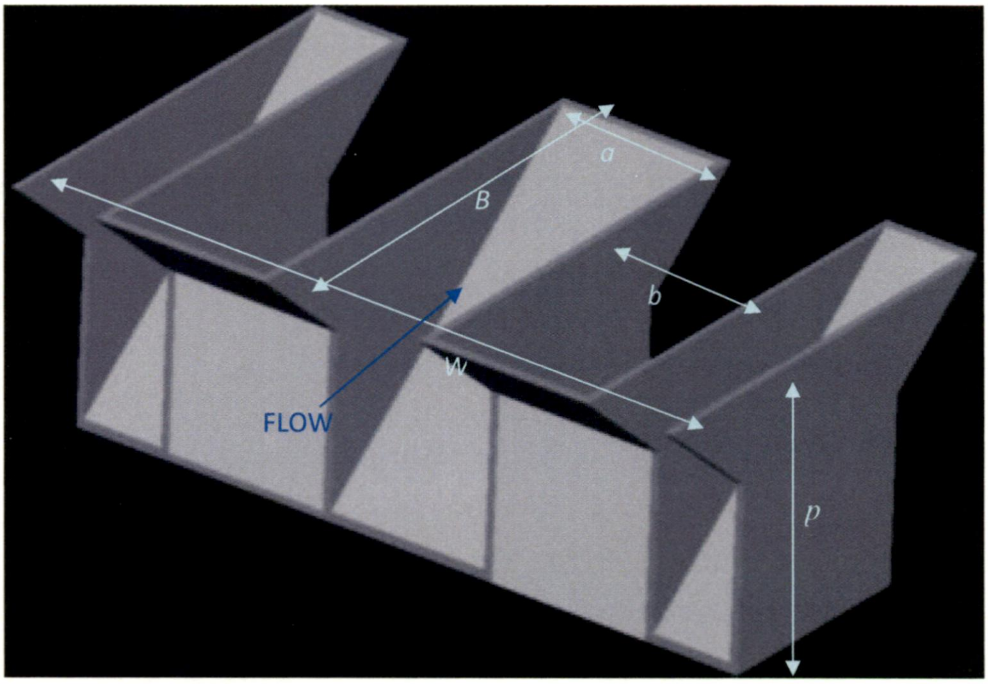


Fig. 5.1 Three dimensional view to generalize Piano Key Weir shape

a = Width of inlet cell

B = Length of elements

b = Width of outlet cell

L = Perimeter of Piano Key Weir crest

p = Crest height of Piano Key Weir

W = Width of channel

5.1.2 Design of Piano Key Weir Type Barrage

Basic Design Data (At Proposed Site near Kharagprasad Village)

River Bed Level	= 46.63 m
Lowest Water Level	= 47.13 m
High Flood Level (HFL)	= 54.01 m
Average Bank Level	= 53.89 m
Upstream Reach Length	= 20 Km
Design Discharge for High Flood	= 8678 cumec
Design Discharge for Low Flow	= 80.65 cumec
Water Way Width	= 624 m
Average Channel Slope	= 1:2000

Considering the fact that piano key weir of different shapes are to be used in the field conditions, experimental studies were conducted by Dr. Gopal Das Singhal, (Ph.d. Thesis, 2009) to identify the Piano Key weir in which the maximum discharge capacity at different L/W with p (height of weir) could be achieved. To achieve this for different flow conditions various parameters were taken in different combinations for getting optimum of the Piano Key weir for better performance. Dr. Gopal Das Singhal studied the various parameters and carried out the data analysis for the most efficient design parameters for Piano Key weir.

5.1.2.1 H for Piano Key Weir for high flood discharge

a) Considering r = 2.0 (for h/p close to unity)

$r = Q_{PK}/Q_L$

Q_{PK} = Discharge Passes over Piano Key Weir

Q_L = Discharge Passes over Linear Weir

$$Q_L = \frac{8678}{2.0} = 4339 \text{ Cumec}$$

$$Q_L = \frac{2}{3} C_d \sqrt{2g} L H^{3/2}$$

$$4339 = \frac{2}{3} \times 0.64 \times \sqrt{2 \times 9.81} \times 624 \times H^{3/2}$$

H = 2.38 m

5.1.2.2 H for low flood discharge

r = 3.4 (For lower value of h/p)

$$Q_L = \frac{80.65}{3.4} = 23.72 \text{Cumec}$$

$$Q_L = \frac{2}{3} C_d \sqrt{2g} L H^{3/2}$$

$$23.72 = \frac{2}{3} \times 0.64 \times \sqrt{2 \times 9.81} \times 624 \times H^{3/2}$$

H = 7.40 cm = 0.074 m

5.1.2.3 H for Ogee spillway for high flood discharge

$$Q_L = \frac{2}{3} C_d \sqrt{2g} L H^{3/2}$$

$$8678 = \frac{2}{3} \times 0.72 \times \sqrt{2 \times 9.81} \times 624 \times H^{3/2}$$

H = 3.50 m

5.1.2.4 Design Dimensions of Piano Key Weir

Crest Level of Piano Key Weir	= 51.67 m
Head Over the Crest on PIANO KEY Weir for High Flood Discharge	= 2.38 m
Maximum Water Level on PIANO KEY Weir	= 54.01 m
Head Over the Crest on PIANO KEY Weir for Low Flow	= 7.40cm
Minimum Water Level on PIANO KEY Weir	= 51.70 m
Afflux	= 3.90 m
Height of Piano Key Weir (p)	= 5.0 m
Width of Channel (W)	= 624 m
Perimeter of Piano Key Weir crest (L)	= 3120 m
Length of elements (B)	= 12 m
Width of outlet cell (b)	= 3 m
Width of inlet cell (a)	= 3 m
Bottom width of wall	= 6 m
L/W	= 5

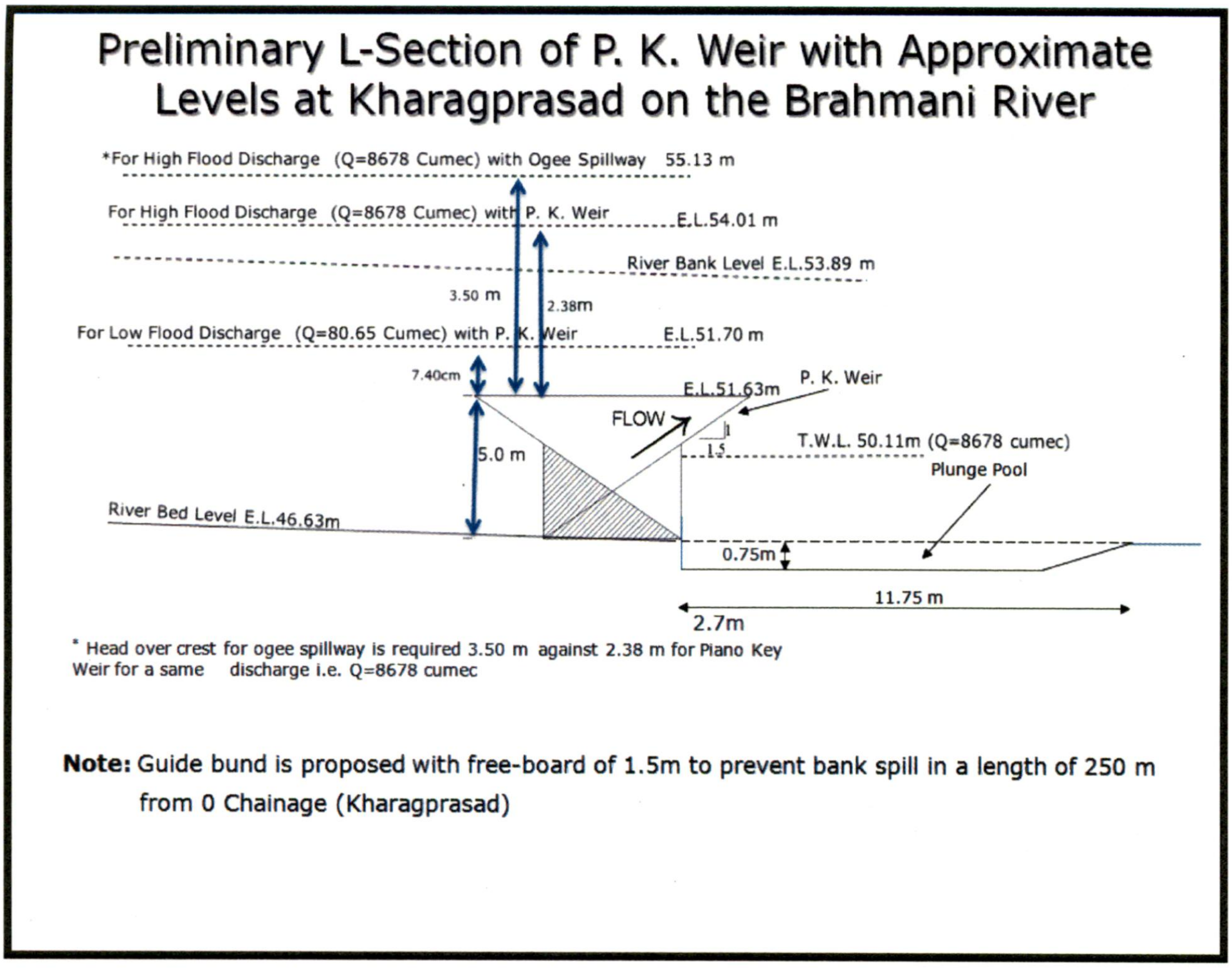


Fig. 5.2 Details of Piano Key Weir and Water Levels

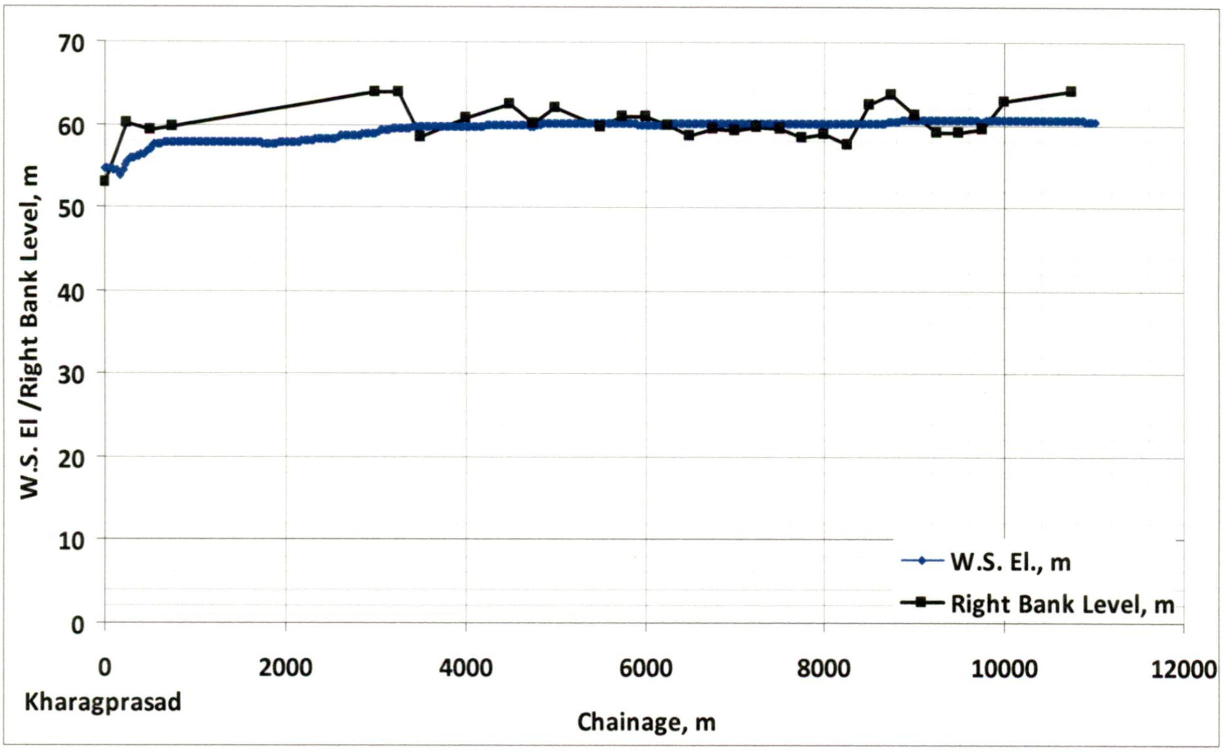


Fig. 5.3 Details of Water Surface Elevation and Right Bank Level

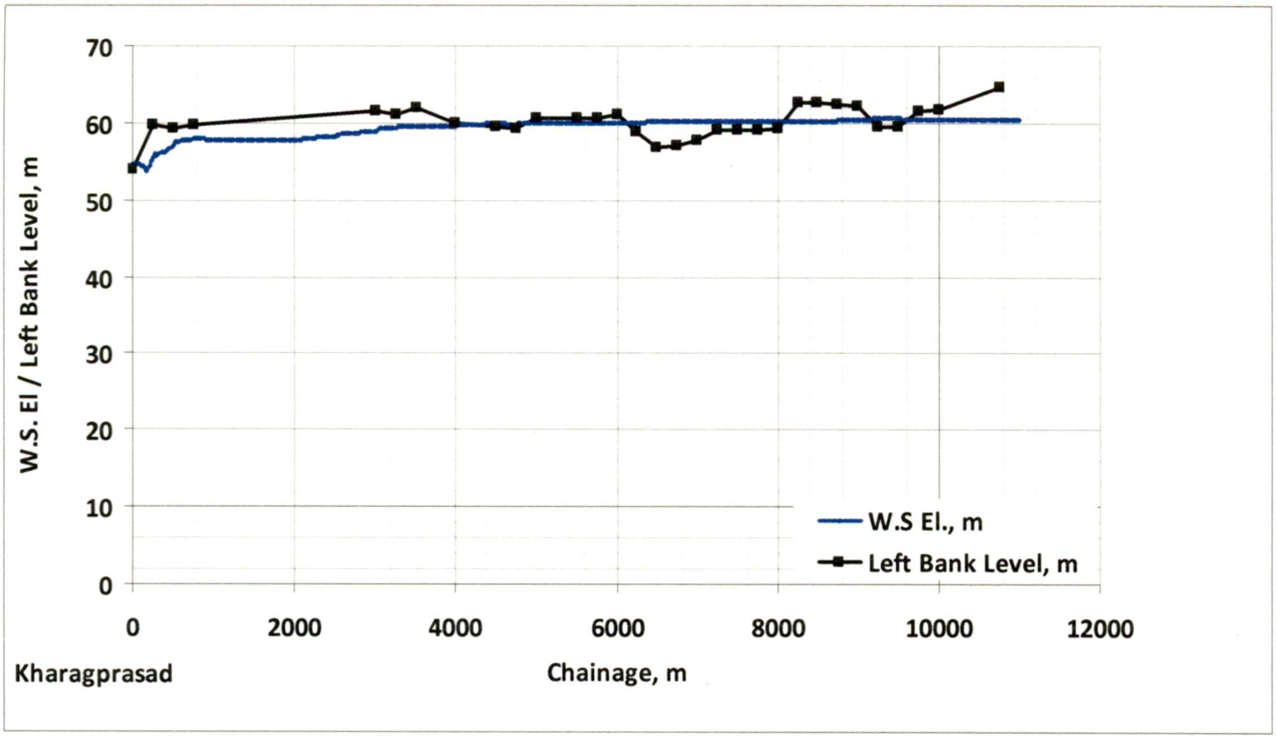


Fig. 5.4 Details of Water Surface Elevation and Left Bank Level

From Fig 5.3 and Fig 5.4, it is clear that the water surface elevation is lower than the Left bank and Right bank level at most of the points. Only in some reach it is higher than the bank levels. In this the guide bund can be provided to avoid the spilling of the banks.

For left bank, the reach from 6.2 km chainage to 8.2km chainage requires guide bund and for right bank, the reach from 6 km to 8 km requires guide bund.

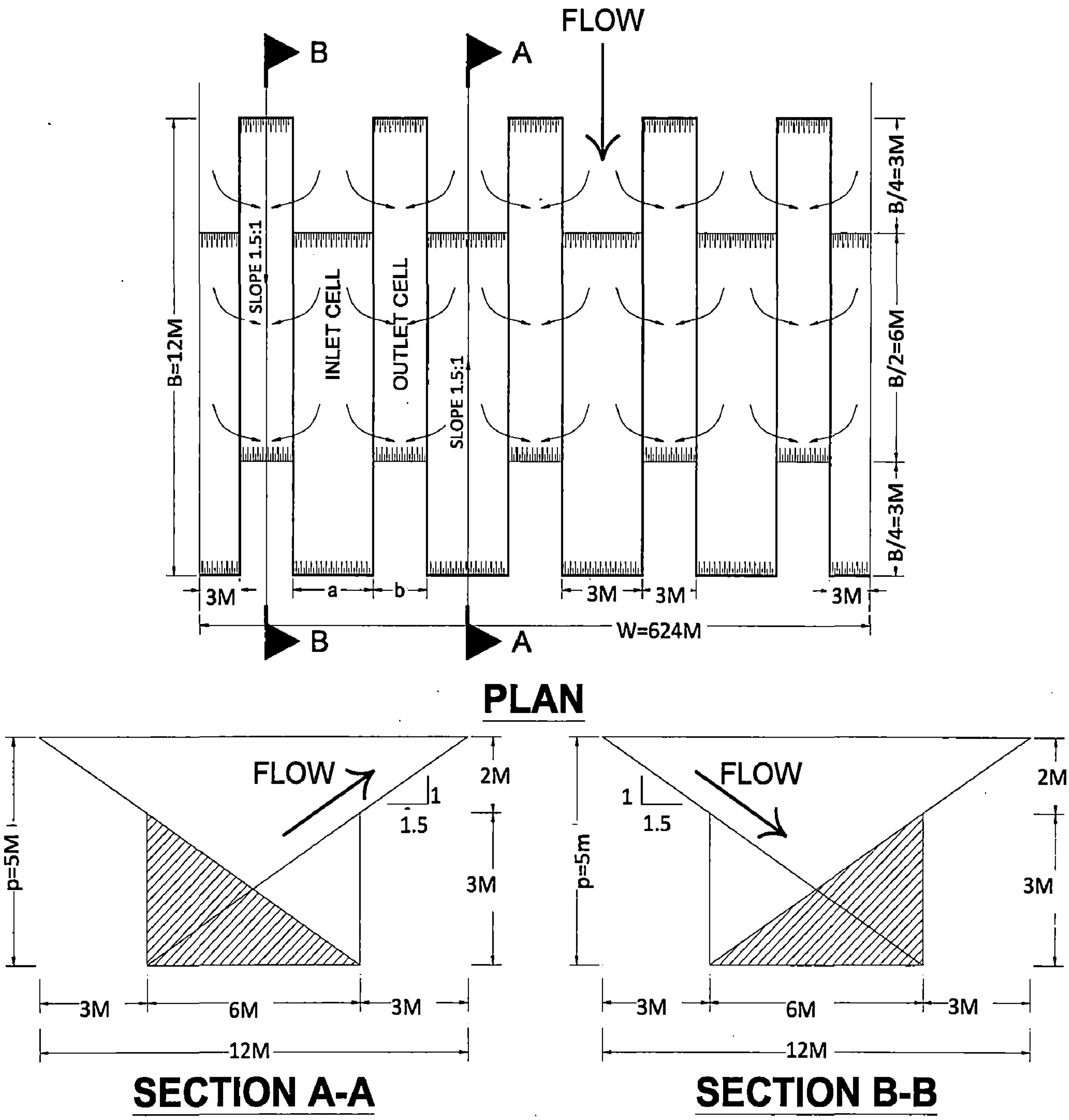


Fig. 5.5 Plan and Section of Piano Key Weir

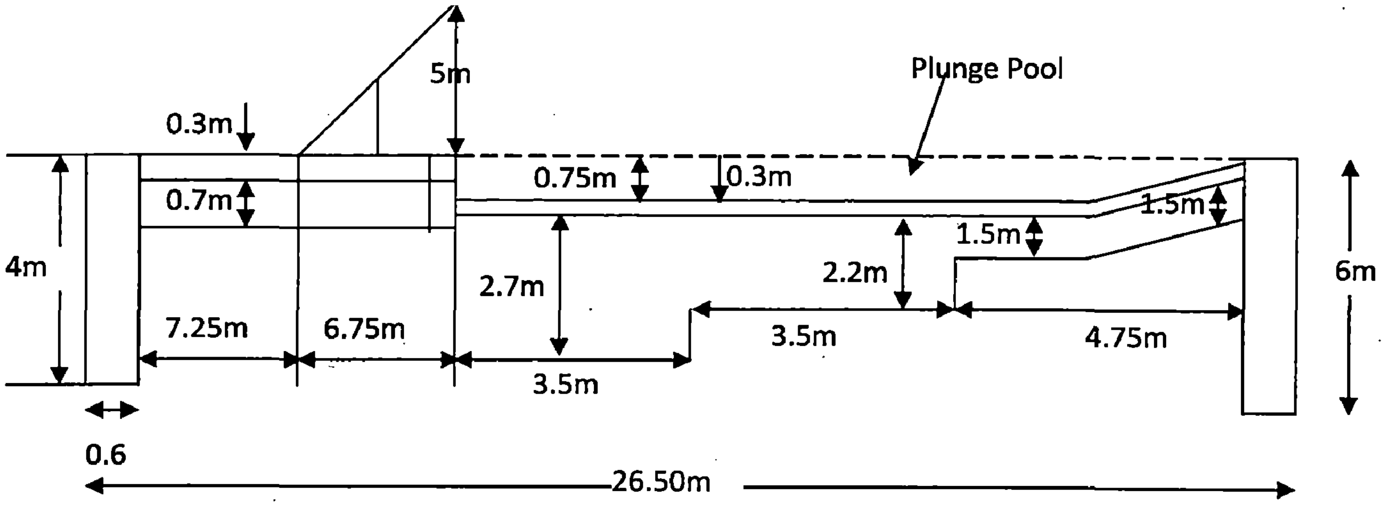


Fig. 5.6 L-Sectiion showing Plunge Pool of Piano Key Weir

Note: 1) The Figures are indicated in the above sketches Not-To-Scale

2) The total number of elements are 104, with each element of 6 M dimensions having inlent and outlet cells of 3m each

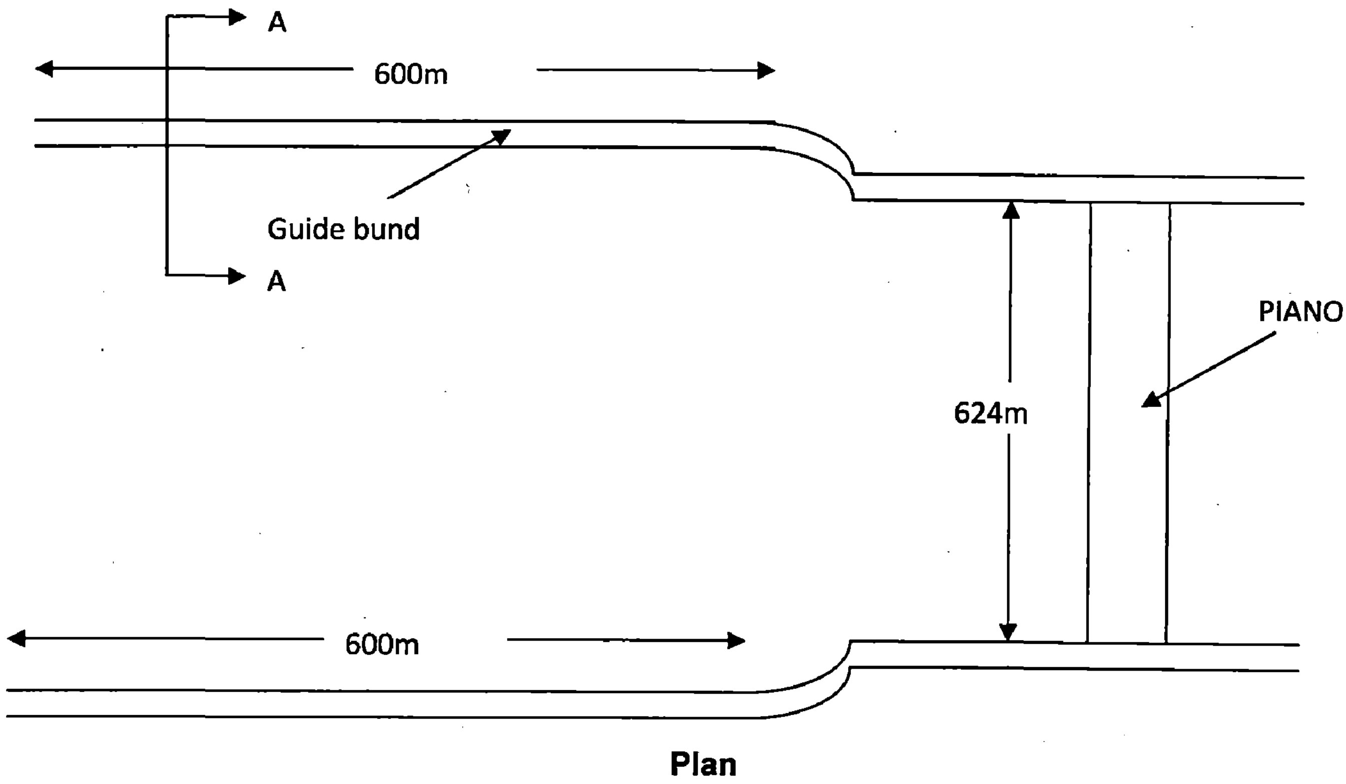


Fig. 5.7 Tentative Details Showing Guide Bund Plan

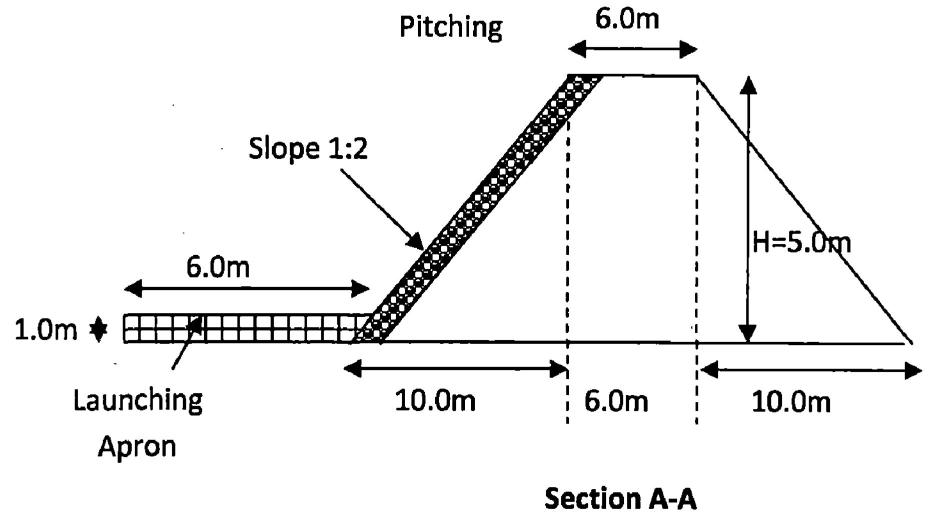


Fig. 5.8 Tentative Details Showing Guide Bund Section at Project Site

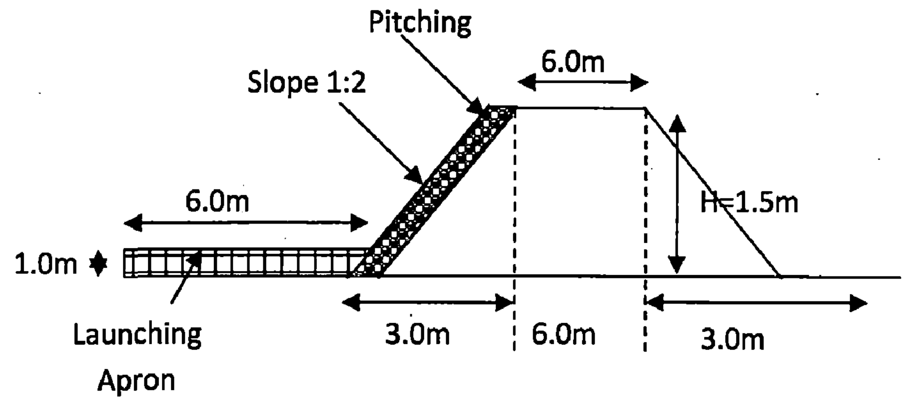


Fig. 5.9 Tentative Details Showing Embankment Section suggested on the left and right b to avoid bank spill

5.2 Evaluation of In-stream Storage Volume through Hydraulic Analysis

HEC-RAS 4, the latest mathematical model developed by US HEC, has been adopted in this study to conduct intensive computer simulation modelling runs for the purpose of evaluating the volume of In-stream storage – both for low flow as well as high flood conditions. Towards this end, the required latest river cross-sections of the Brahmani River have been surveyed from near Kharagprasad village to about 11 Km upstream and 1.5 Km downstream.

The surveyed river cross-sections have been used to construct the mathematical model. The river bed sand samples have been analysed by adopting the well-known Strickler's equation for estimation of rugosity coefficient value as 0.015 for using in the model during low flow and 0.0085 during high flow.

The mathematical model has been operated for high flood as well as low flow conditions for computation of in-stream storage volumes. The discharging capacity and head over crest for Piano Key Weir have been estimated using relevant weir formula. The corresponding afflux comes to be 2.38 m for a Piano Key Weir height of 5 m yielding h/p ratio of 0.29 for L/W ratio of 5, which is highly satisfactory from hydraulic performance of the PIANO KEY Weir.

Using the latest software HEC- RAS – 4 developed by US Hydrologic Engineering Center, Vicksburg, USA, mathematical modelling analysis has been carried out to assess the back water for high flood and low water conditions. The findings are given below and data output from HEC-Ras 4 is given in Appendix 1 and Appendix 2:

- Length of Backwater due to incident afflux with PIANO KEY Weir in position (Low Flow) =11 Km
- Volume of In-Stream Storage in Low Flow = 17.701 Million Cubic Meter (MCM)
- Water Requirement for Industry / Drinking Water Purpose = 2.1 Cumec
- If minimum flow ceases to exist, the above In-stream storage will cater for 97 days

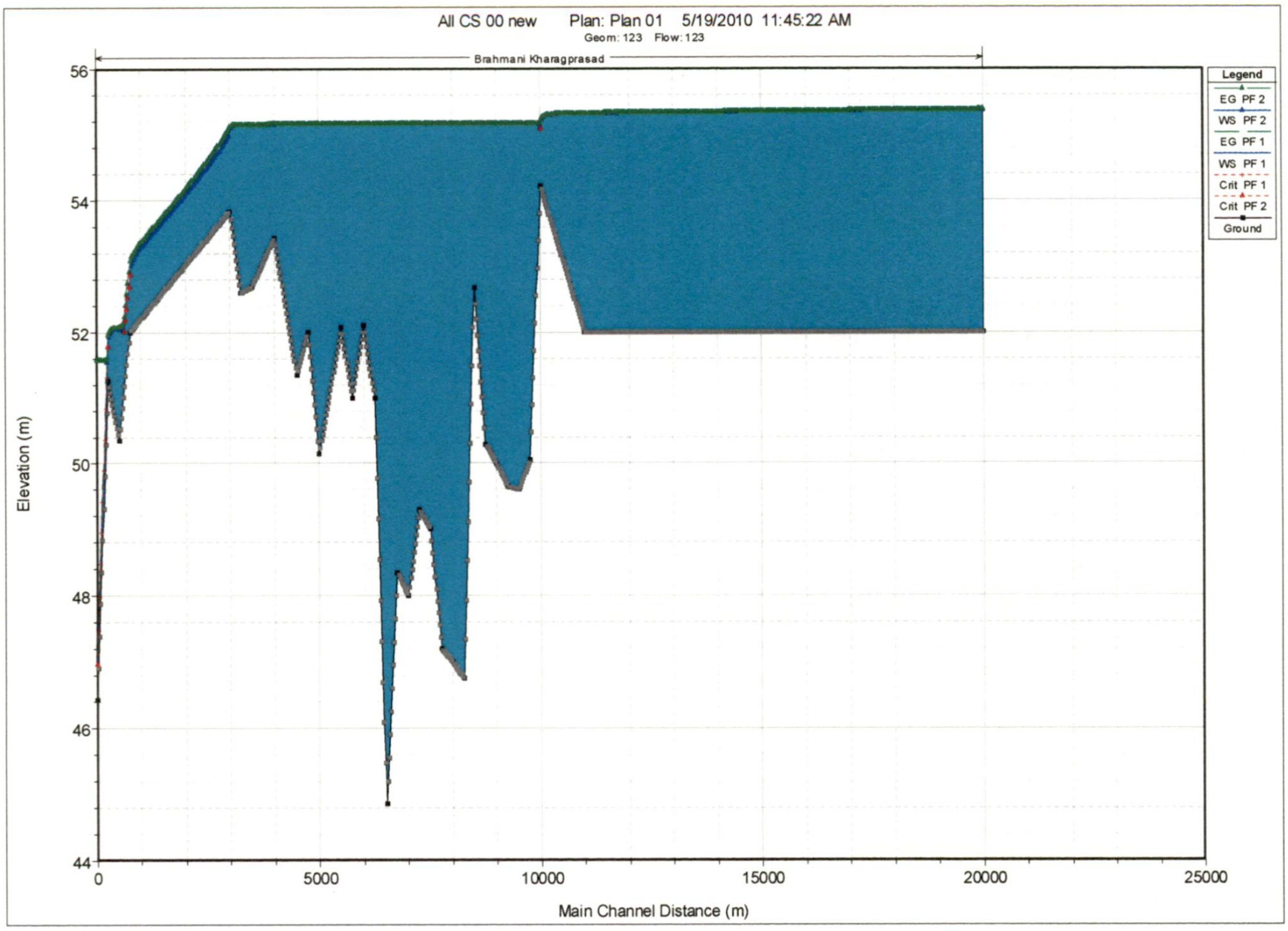


Fig. 5.10 Water surface profile of Brahmani River due to Piano Key Weir For Low Flow Condition showing In-stream storage volume

Table 5.1 Tabular Presentation of Mathematical Modelling Output Yielded by HEC-RAS 4 for Low Flow Condition (In-stream Storage Volume)

E.G. Elev (m)	55.36	**Element**	Left OB	Channel	Right OB
Vel Head (m)	0	**Wt. n-Val.**		0.015	
W.S. Elev (m)	55.36	**Reach Len. (m)**	25	25	25
Crit W.S. (m)		**Flow Area (m2)**		439.24	
E.G. Slope (m/m)	0.000005	**Area (m2)**		439.24	

Q Total (m3/s)	80.65	**Flow (m3/s)**		80.65	
Top Width (m)	329.42	**Top Width (m)**		329.42	
Vel Total (m/s)	0.18	**Avg. Vel. (m/s)**		0.18	
Max Chl Dpth (m)	3.36	**Hydr. Depth (m)**		1.33	
Conv. Total (m3/s)	35375.6	**Conv. (m3/s)**		35375.6	
Length Wtd. (m)	25	**Wetted Per. (m)**		330.8	
Min Ch El (m)	52	**Shear (N/m2)**		0.07	
Alpha	1	**Stream Power (N/m s)**		0.01	
Frctn Loss (m)	0	**Cum Volume (1000 m3)**		17701.54	
C & E Loss (m)	0	**Cum SA (1000 m2)**		8886.5	

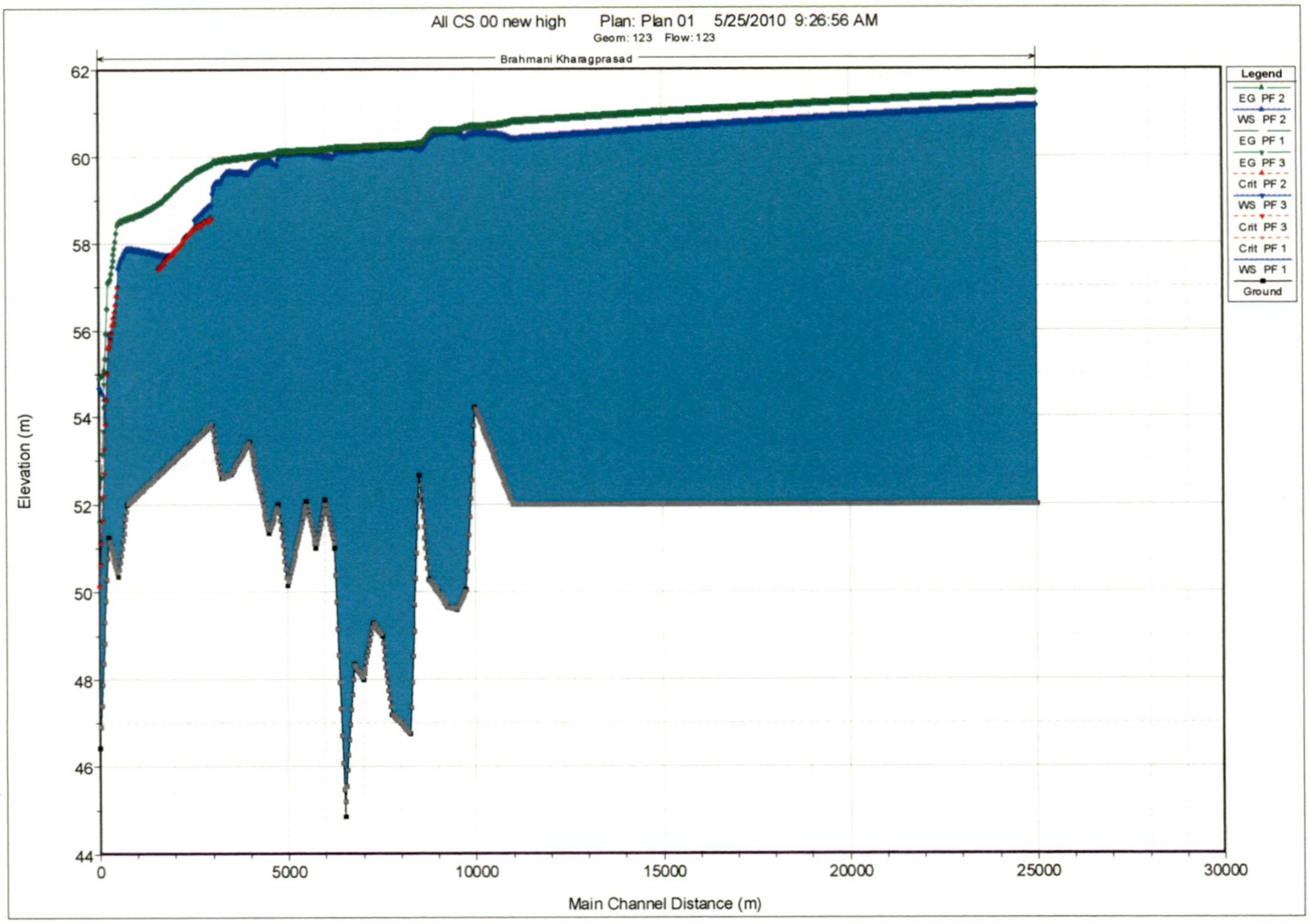


Figure 5.11 Water Surface Profile of Brahmani River Due to Piano Key Weir For High Flood Condition

Table 5.2 Tabular Presentation of Mathematical Modelling Output Yielded by HEC-RAS 4 for High Flow Condition (In-stream Storage Volume)

E.G. Elev (m)	61.45	**Element**	Left OB	Channel	Right OB
Vel Head (m)	0.32	**Wt. n-Val.**		0.009	
W.S. Elev (m)	61.13	**Reach Len. (m)**	25	25	25
Crit W.S. (m)		**Flow Area (m2)**		3464.78	
E.G. Slope (m/m)	0.000039	**Area (m2)**		3464.78	
Q Total (m3/s)	8678	**Flow (m3/s)**		8678	
Top Width (m)	540	**Top Width (m)**		540	
Vel Total (m/s)	2.5	**Avg. Vel. (m/s)**		2.5	
Max Chl Dpth (m)	9.13	**Hydr. Depth (m)**		6.42	
Conv. Total (m3/s)	1385426	**Conv. (m3/s)**		1385426	
Length Wtd. (m)	25	**Wetted Per. (m)**		552.95	
Min Ch El (m)	52	**Shear (N/m2)**		2.41	
Alpha	1	**Stream Power (N/m s)**		6.04	
Frctn Loss (m)	0	**Cum Volume (1000 m3)**		97927.6	
C & E Loss (m)	0	**Cum SA (1000 m2)**		16357.39	

CHAPTER 6
DEVELOPMENT OF COST FUNCTIONS FOR ASSESMENT OF IN-STREAM STORAGE OPTIONS

6.1 General

Within the same ideas of In-stream storage to avoid submergence and Relief and Resettlement and exercise has been carried out to investigate the scope for enhancement of In-stream storage volume by considering different heights of marginal dikes on both sides of River Brahmani.

6.2 Methodology

The following methodology has been adopted. The height of the crest level of Piano Key weir is increased in the increments of 0.5 up to 3.5 m. The corresponding increase in the cost of earthwork and the increase in In-stream storage volume are worked out. The charts are plotted to develop the relationship between the rise in crest level along with guide bund level rise and rise in cost of a Piano Key weir, rise in crest level along with guide bund level rise and rise in the storage volume created.

The latest HEC- RAS 4 software is used for estimation of volume of In-stream storage for different crest level heights varying from 5 m to 8.5 in the increments of 0.5m.

The different options examined for possible enhancement of in-stream storage are presented in the following tables.

Table 6.1: Crest level rise and corresponding In-stream storage volume

Crest Level (m)	In-stream storage days
0.5	99
1	102
1.5	106
2	127
2.5	189
3	259
3.5	295

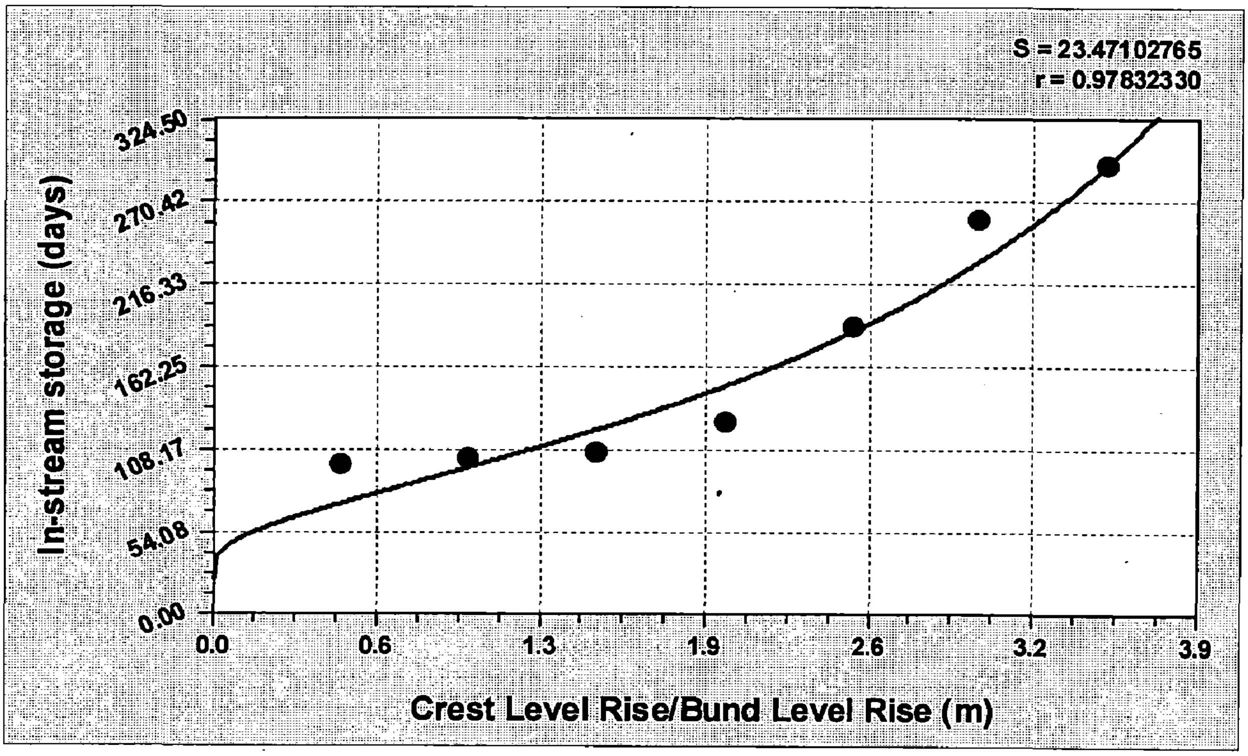


Fig. 6.1 Relationship between Crest level rise and In-stream storage volume

The Fig 6.1 is developed so that the In-stream storage for the rise in crest level of the Piano Key weir can be obtained for the range of 0.5 to 3.5 m. The equation is also developed for the above curve using CURVE EXPERT software. The value of coefficient of correlation is also shown in the Fig 6.1.

Table 6.2 Crest level and corresponding increase in cost

Crest level (m)	Increase in cost (crore)
0.5	67.48
1	69.23
1.5	71.22
2	73.48
2.5	76.02
3	78.81
3.5	81.87

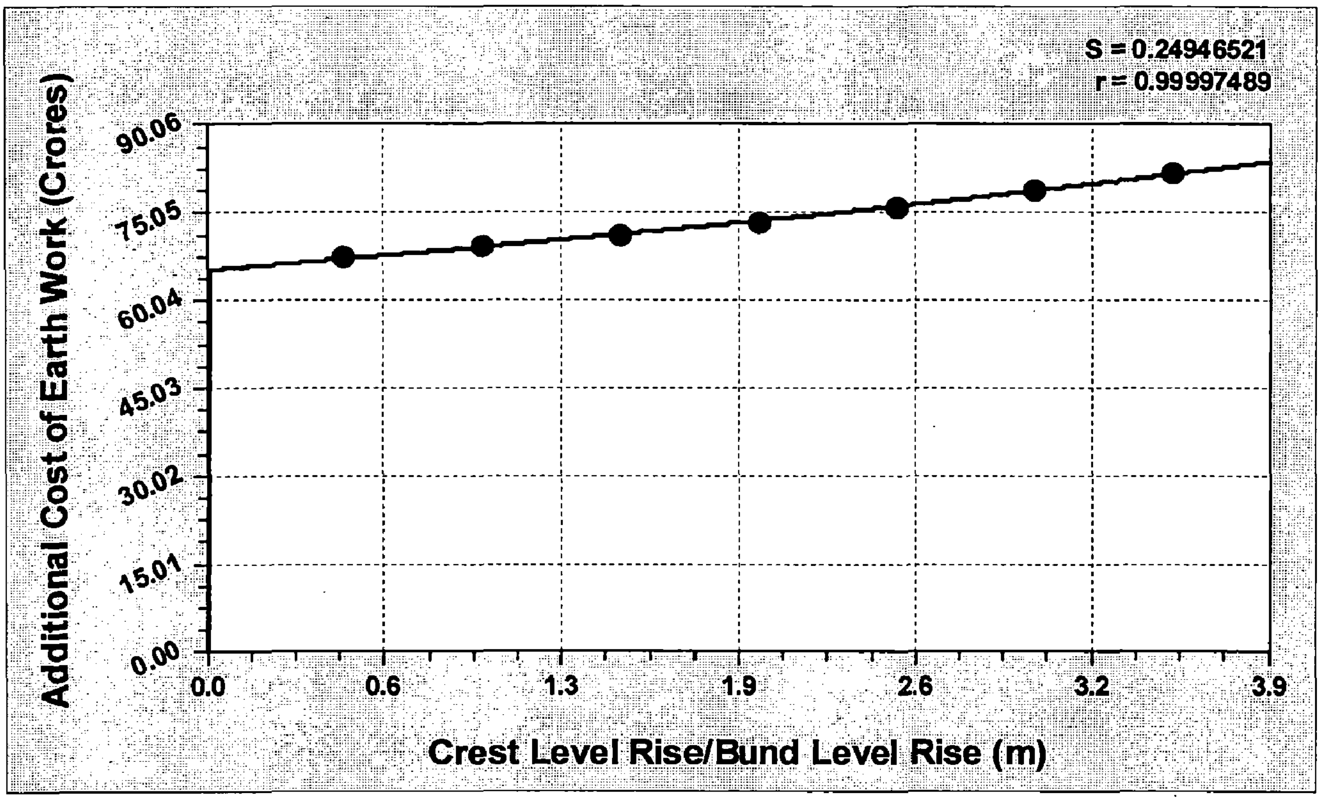


Fig 6.2 Relationship between Crest level rise and additional cost of earthwork

The Fig 6.2 is developed so that the additional cost of earthwork to rise the bund level for the rise in crest level of the Piano Key weir can be obtained for the range of 0.5 to 3.5 m. The equation is also developed for the above curve using CURVE EXPERT software. The value of coefficient of correlation is also shown in the Fig 6.2.

6.3 Bank spill Check

Due to increase in crest level, the In-stream storage volume increases with the increase in water surface elevation. The bank height may not be sufficient for the water to be stored in the stream and the water may spill the banks. Hence for each crest level rise, the graph showing the left bank level/right bank level, water surface profiles are plotted so that the bank spill information can be obtained. If the water surface elevation is more than the bank level, the water is spilled for that point. The Fig 6.3 to Fig 6.16 shows this relationship. In all the below figure, there is a very little length of bank spilling. At that point, the embankment is suggested. The bank spilling can be protected in this manner. Here the water surface profiles are plotted for the 500 years design flood of 8678 cumec.

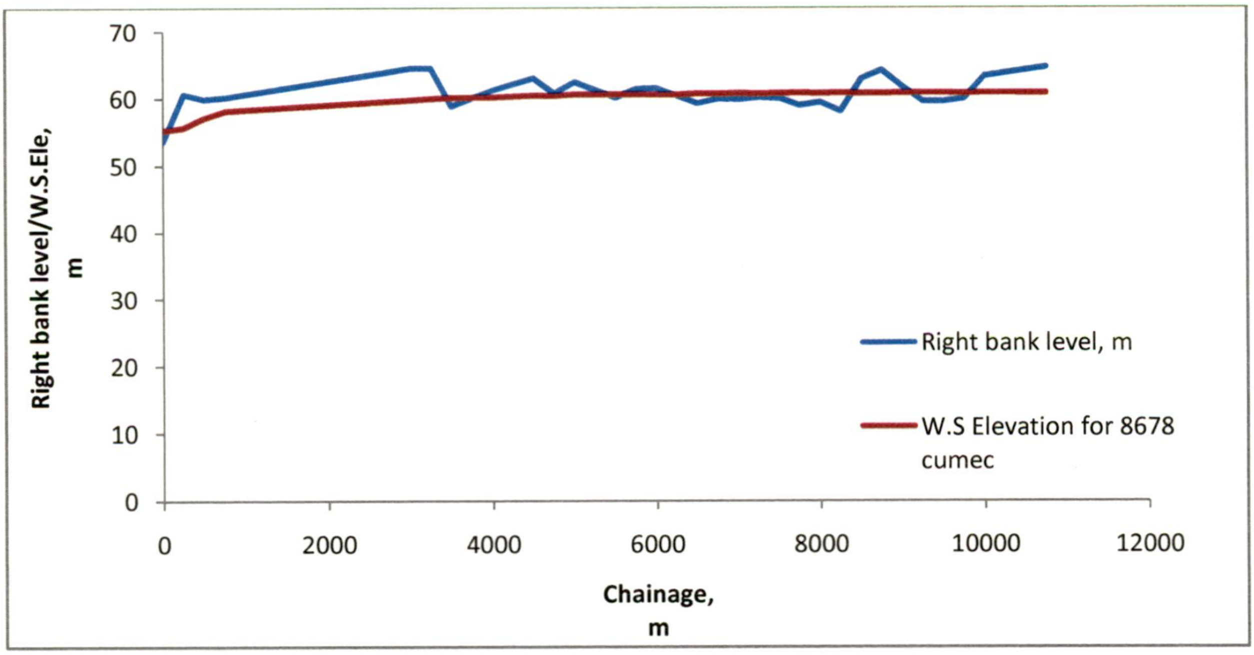


Fig. 6.3 Details of Water Surface Elevation and Right bank level for 0.5 m increase of crest level

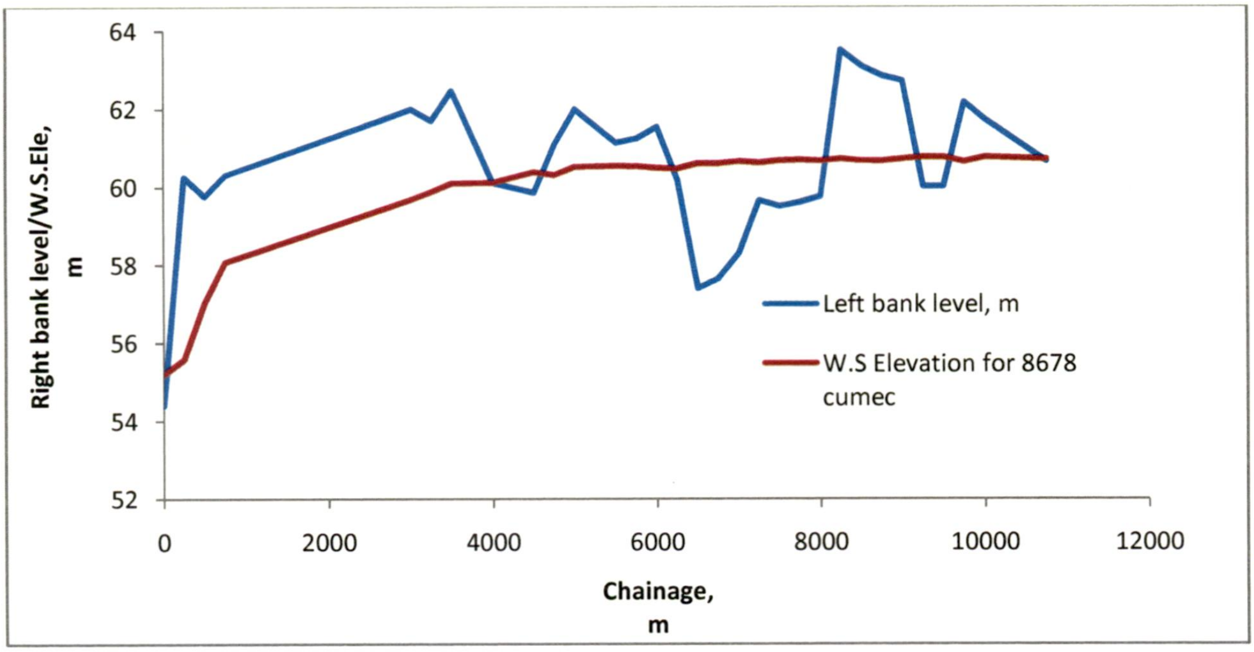


Fig. 6.4 Details of Water Surface Elevation and Left bank level for 0.5 m increase of crest level

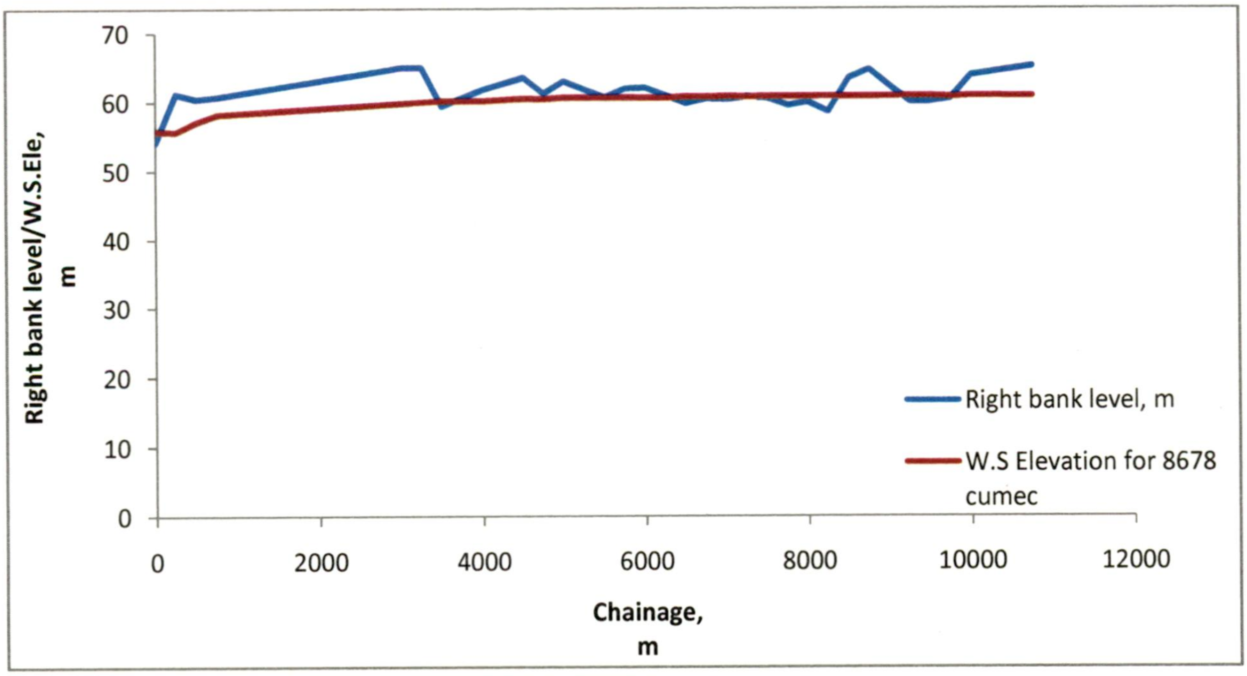


Fig. 6.5 Details of Water Surface Elevation and right bank level for 1 m increase of crest level

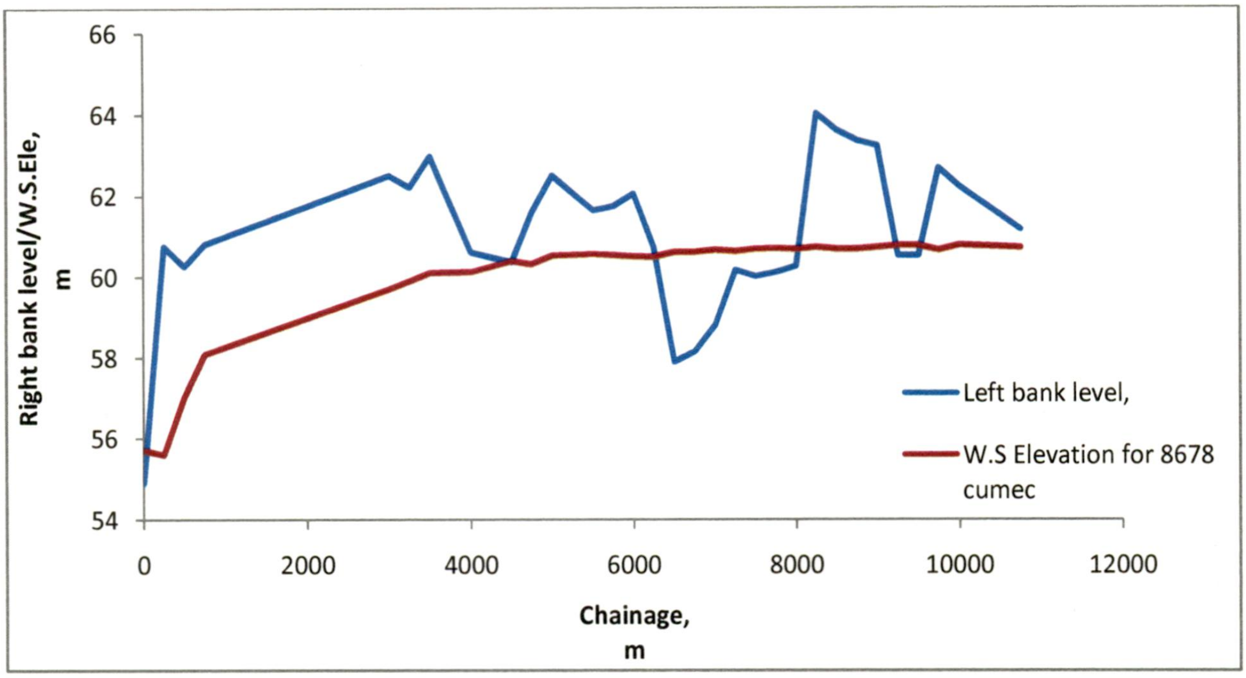


Fig. 6.6 Details of Water Surface Elevation and Left bank level for 1 m increase of crest level

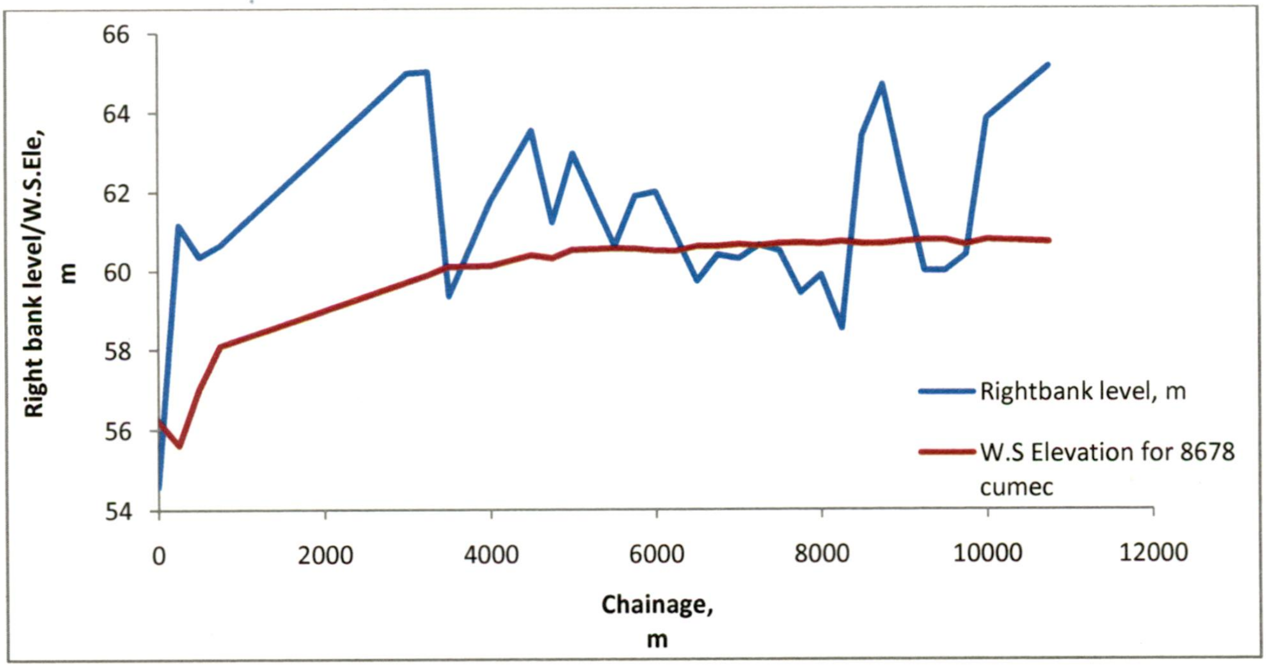


Fig. 6.7 Details of Water Surface Elevation and right bank level for 1.5 m increase of crest level

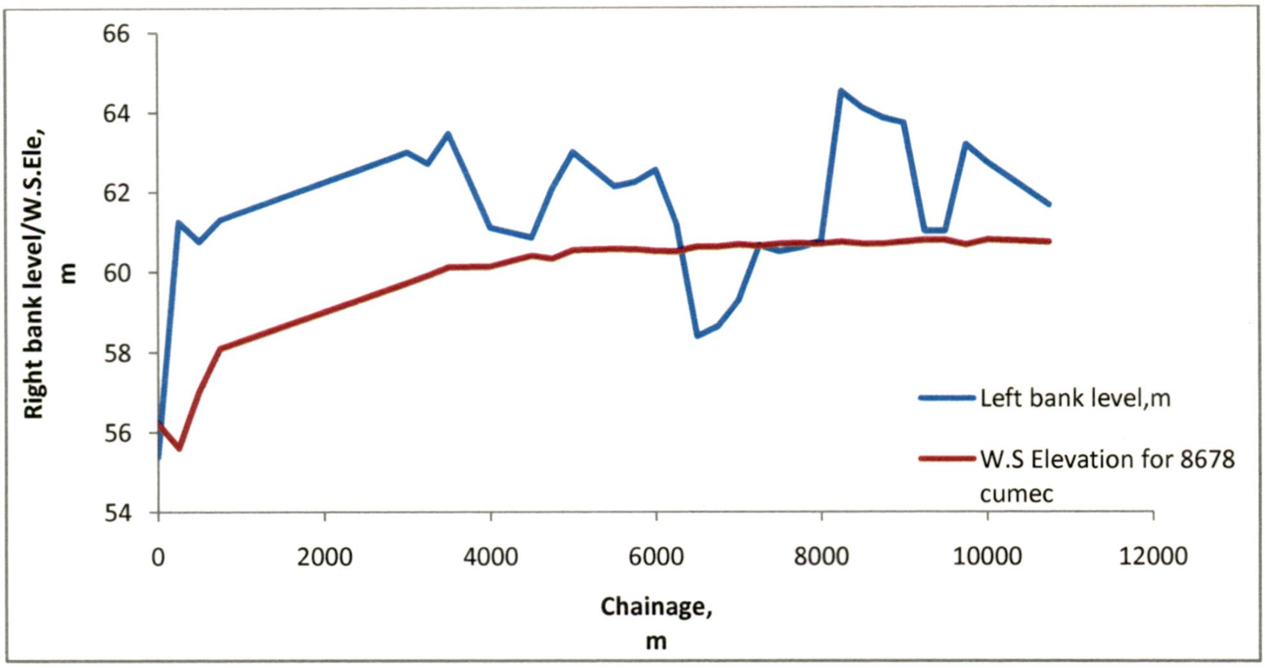


Fig. 6.8 Details of Water Surface Elevation and Left bank level for 1.5 m increase of crest level

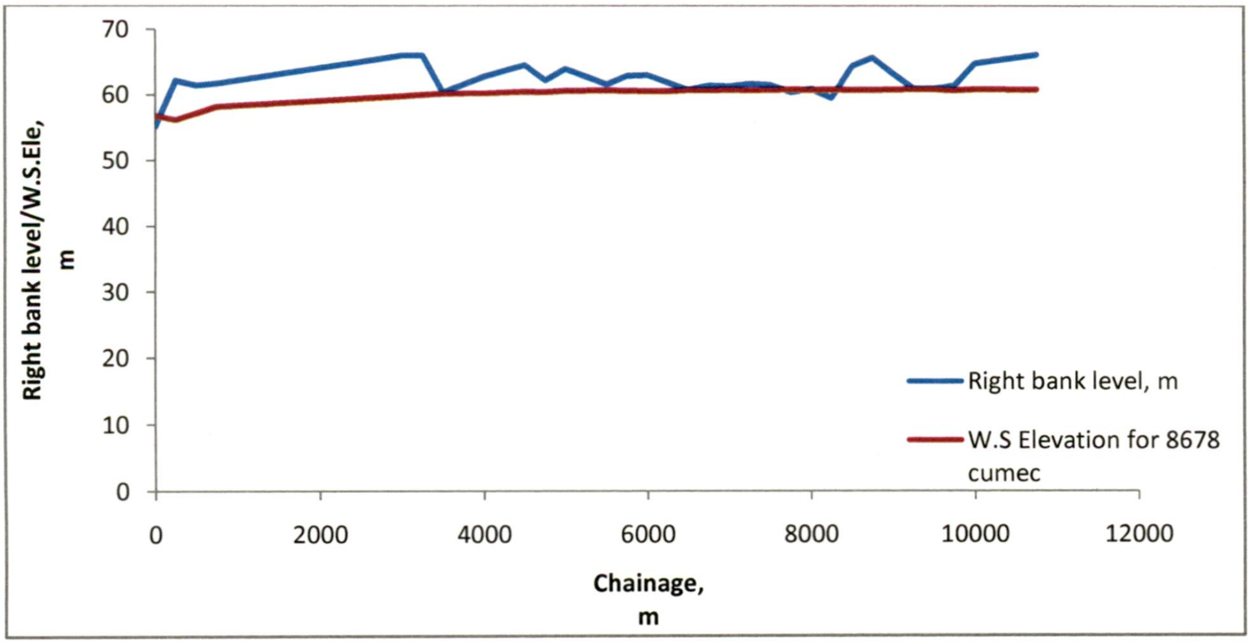


Fig. 6.9 Details of Water Surface Elevation and right bank level for 2 m increase of crest level

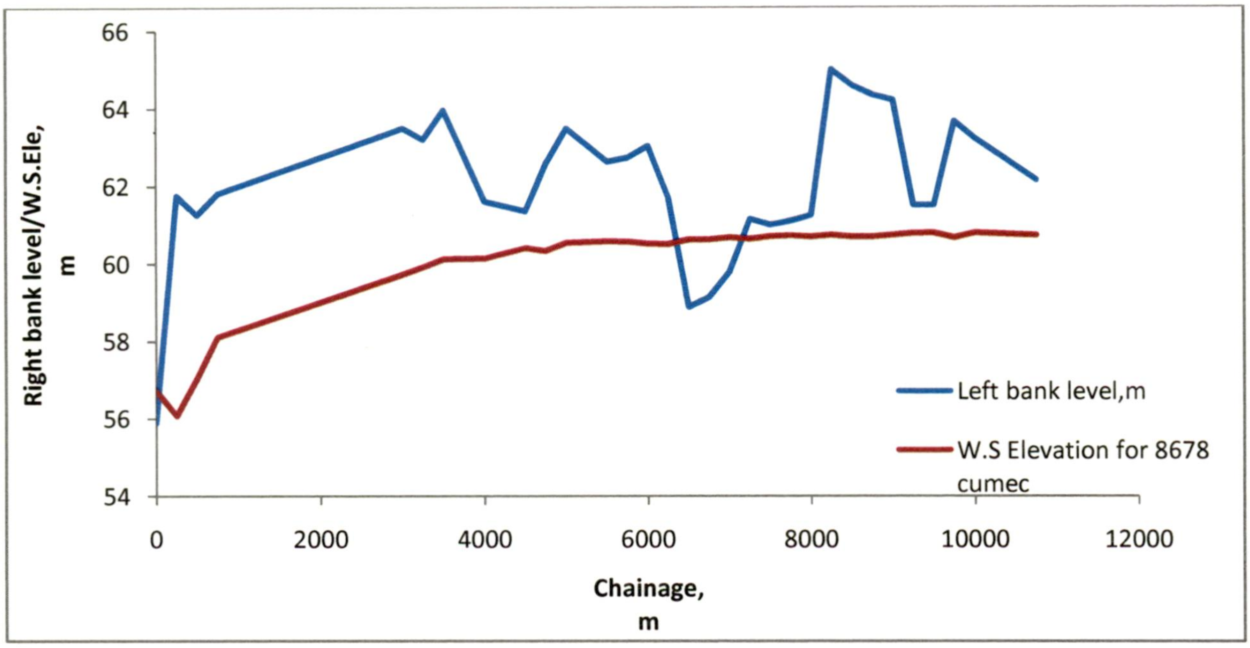


Fig. 6.10 Details of Water Surface Elevation and Left bank level for 2 m increase of crest level

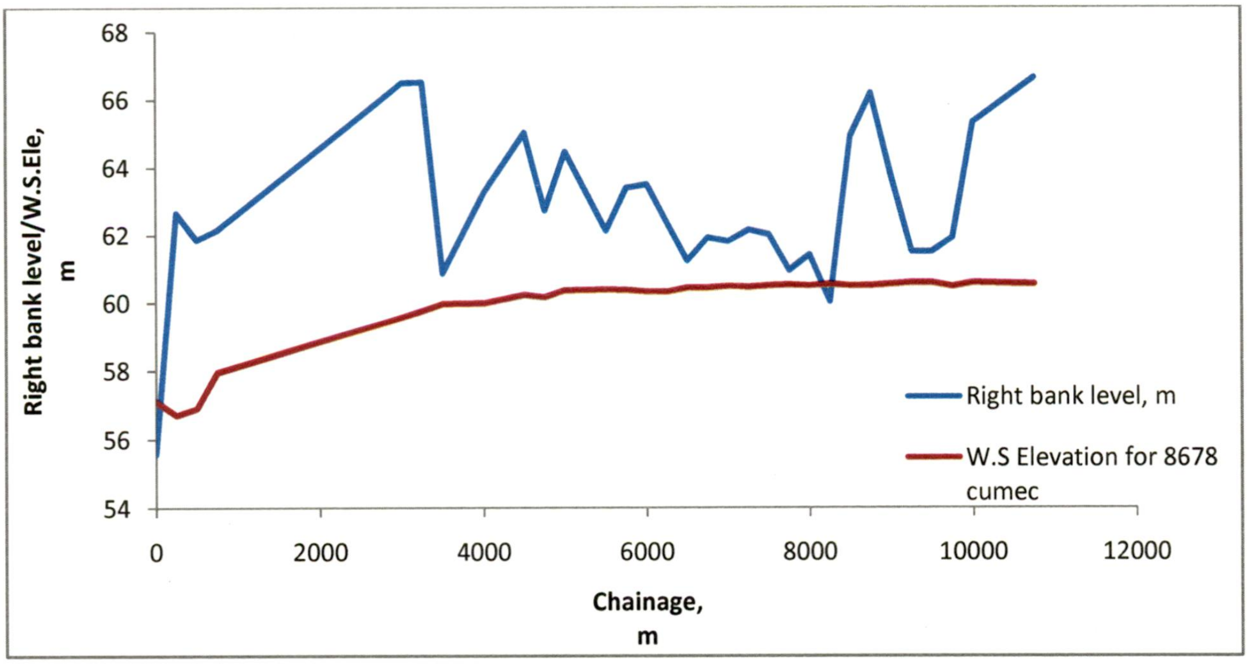


Fig. 6.11 Details of Water Surface Elevation and right bank level for 2.5 m increase of crest level

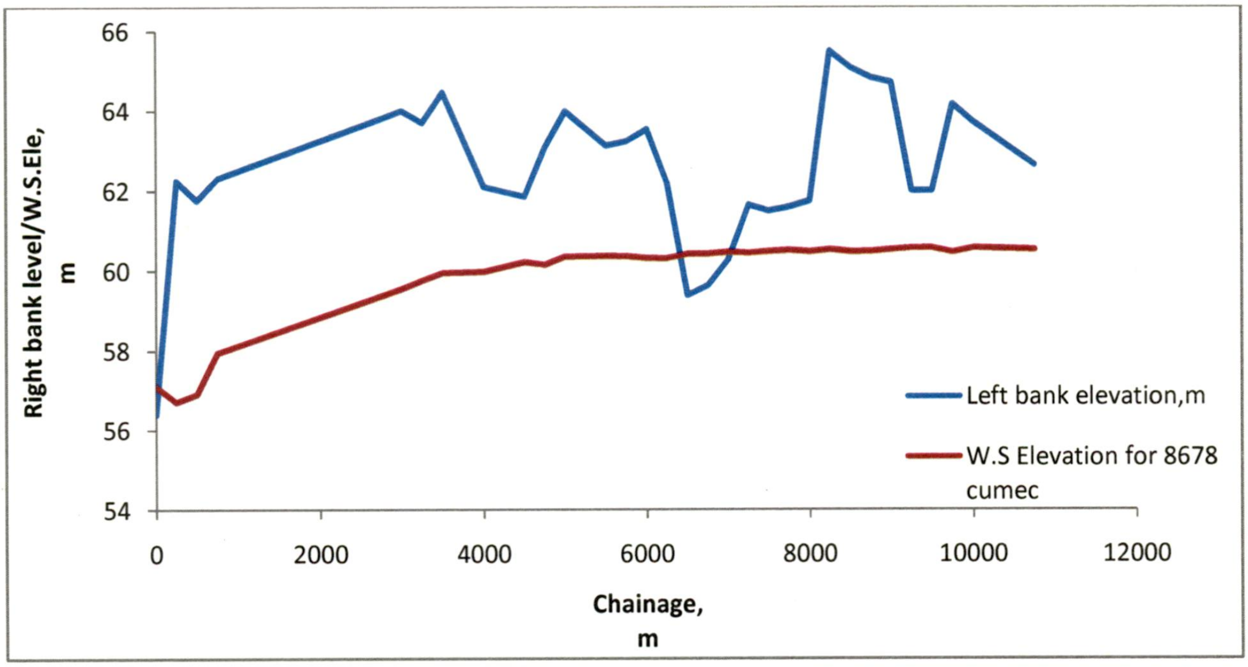


Fig. 6.12 Details of Water Surface Elevation and Left bank level for 2.5 m increase of crest level

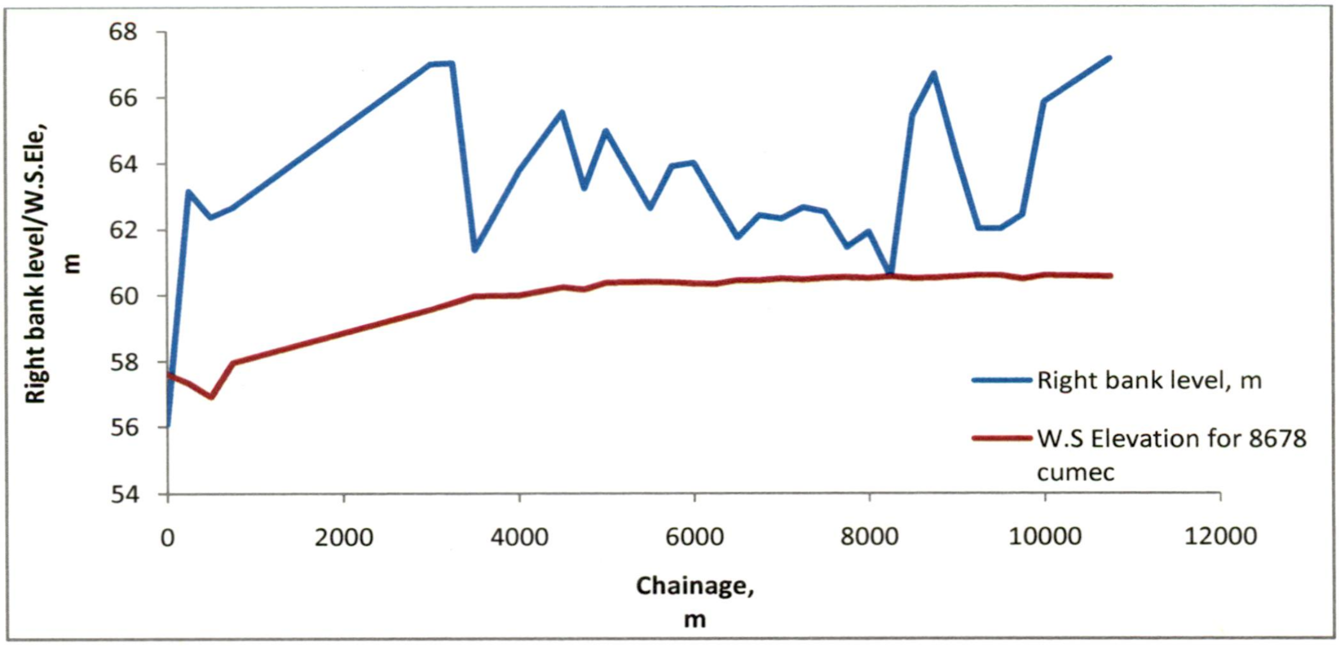


Fig. 6.13 Details of Water Surface Elevation and right bank level for 3 m increase of crest level

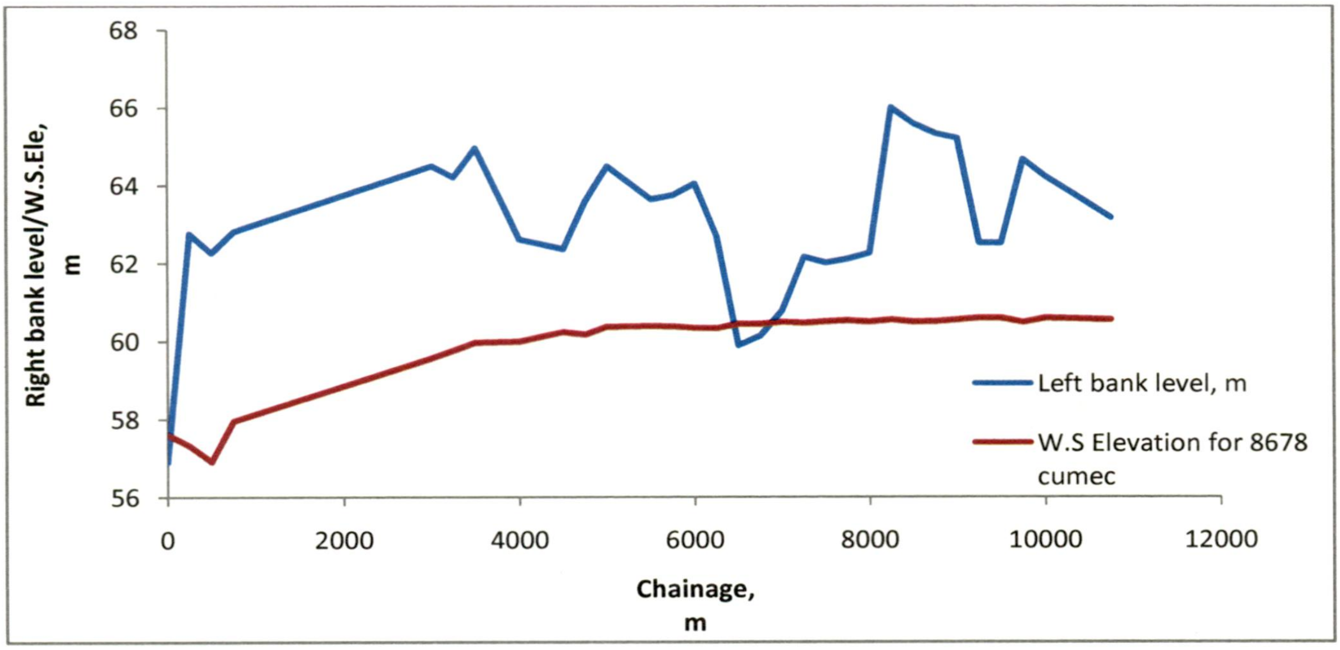


Fig. 6.14 Details of Water Surface Elevation and Left bank level for 3 m increase of crest level

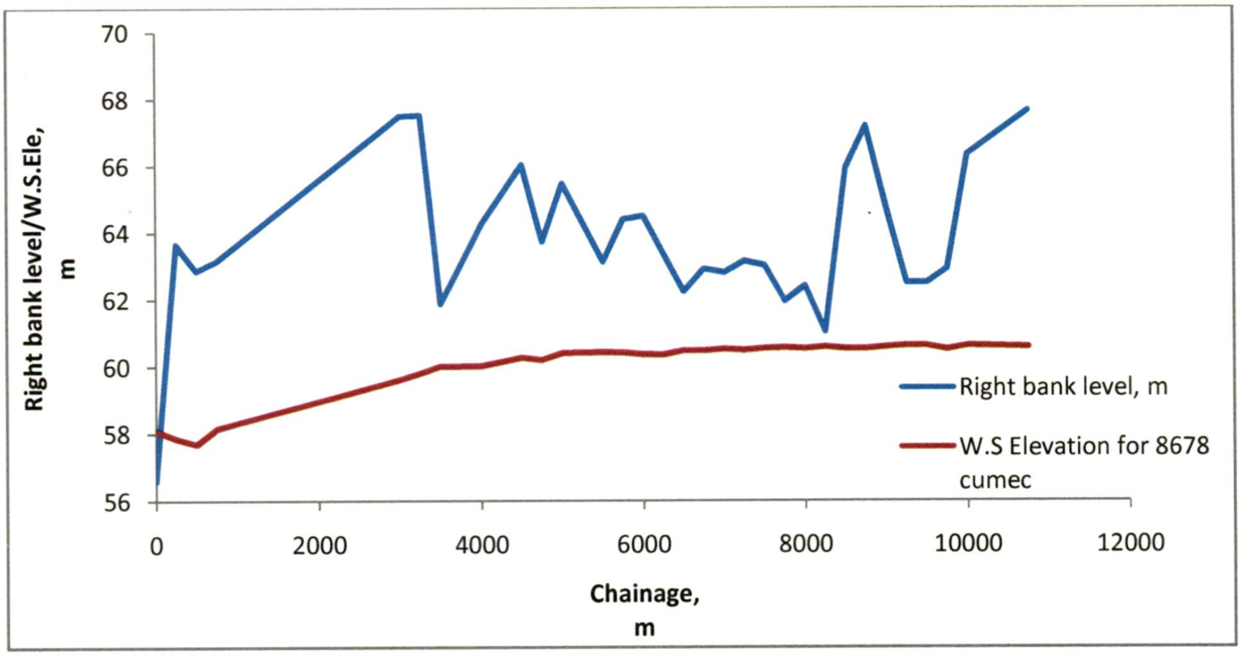


Fig. 6.15 Details of Water Surface Elevation and right bank level for 3.5 m increase of crest level

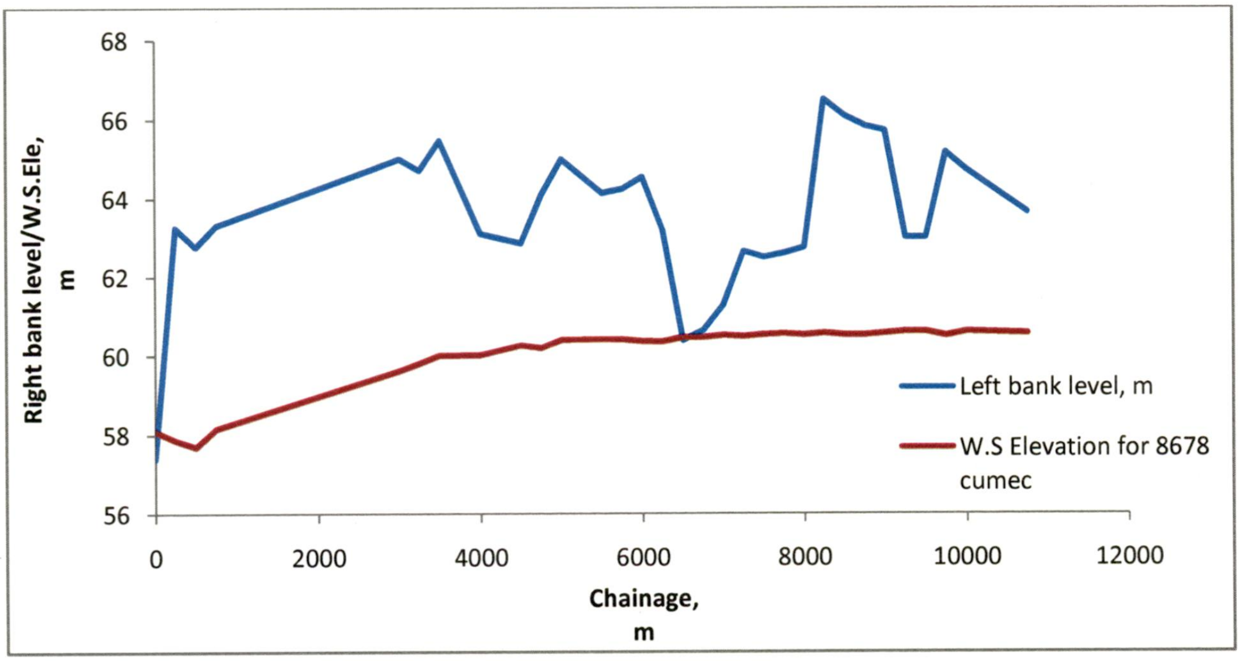


Fig. 6.16 Details of Water Surface Elevation and Left bank level for 3.5 m increase of crest level

7.1 SATELLITE IMAGERY STUDY

1) By comparing the satellite images of Brahmani River in 1998 and 2009, it can be clearly said that the river is by and large stable as there is very minor change like small amount of erosion or deposition. There was no breading of the river found. Hence there was no need of plan form studies.

2) Three barrage sites can be considered for the preliminary selection of the Barrage site near Kharagprasad for which the latitude and longitude are indicated below:

Table 7.1 Potential Barrage Sites near Kharagprasad

Barrage site	Latitude	Longitude	Approximate channel width along barrage axis (m)
1-a	$20^0 49' 34.29''$	$85^0 19' 5.57''$	450
1-b	$20^0 49' 13.12''$	$85^0 19' 43.29''$	520
1-c	$20^0 48' 32.21''$	$85^0 21' 17.78''$	624

Fig: 7.1 Potential Barrage Sites near Kharagprasad

a) **Barrage site 1-a:** It is located almost at the Kharagprasad. It is the narrowest site so it would be cheaper and easier to construct at this site. But there is a drifting channel at the upstream of this site. The drifting channel is quite stable as it has been present since the year 1998 till now. So we can consider this site as the stable and suitable one. However, it may be considered to plug the drifting channel also, but it would involve huge cost.

b) **Barrage site 1-b:** It is situated at the downstream of Kharagprasad. But it is wider than the site no 1 and therefore may cost more for the barrage.

b) **Barrage site 1-b:** It is situated at the downstream of Kharagprasad. But it is wider than the site no 1 and therefore may cost more for the barrage.

c) **Barrage site 1-c:** Located at the downstream of the Barrage site 2. It is at the downstream of the tributary channel joining from the right bank of Brahamni River. It can be advantageous as more water volume will be available for the proposed in-stream storage due to the joining of the tributary channel. But at the same time more elaborate arrangements are to be made to regulate and manage the flood.

Final Barrage Site near Kharagprasad

Out of above three sites, Site 1-c is recommended for the construction of Piano Key Weir. This site is relatively stable without any drifting channel as observed from satellite imagery study. The cross-section is relatively more prismatic. In-stream storage volume will be enhanced for this site due to wider waterway at the proposed Piano Key Weir.

3) The other four barrage sites are also identified at the downstream of Kharagprasad for the creation of In-stream storage in the cascade form. The detail of the selected sites is given in the Table 7.2.

Fig 7.2 Potential Barrage Sites near and Downstream of Kharagprasad

From an analysis of the river plan form, alignment and migration behaviour using satellite imagery data for 1998 and 2009, the above sites are found to be suitable for location of the barrages. The channel width at the sites is reasonably narrow compared to the adjoining sites. Moreover their immediate upstream channel reach is fairly well defined without any sharp sinuosity. Furthermore, the spacing of the barrage sites has been carefully chosen to provide the required space for In-stream storage provision within the backwater zone.

Table 7.2 Identification of Piano Key Weir type Barrage Sites for Creation of In-Stream Storage with Cascade Development in Brahmani River between Kharagprasad and Jenapur

Barrage site	Latitude	Longitude	Approximate chain age, along the river, (km)	Approximate river width at barrage site, (m)
1-a, near Kharagprasad	20°48'32.21"N	85°19'5.57"E	0	624
2	20°46'9.01"N	85°29'25.06"E	20.43	920
3	20°49'8.54"N	85°39'3.95"E	43.96	960
4	20°51'38.26"N	85°50'6.39"E	66.11	875
5	20°53'0.53"N	86° 0'54.92"E	88.25	1000

Five barrage sites have been identified on preliminary basis for creation of in-stream storage by adopting cascade development approach on the Brahmani River between Kharagprasad and Jenapur as shown in Figure 7.2. The location details of the five barrage sites are shown in Table 7.2.

7.2 DESIGN FLOOD ANALYSIS

7.2.1 Flow Duration Curve Method

Exceedence probability is calculated by M/ (n+1), where M is the rank and n is total number of record. The results obtained are as follows:-

- **50%** dependable flow is **253.9** cumec
- **75%** dependable flow is **152.6**cumec
- **90%** dependable flow is **104.0**cumec
- **95%** dependable flow is **80.65**cumec

The **95%** dependable flow is **80.65 Cumec** which is considered for the computation of in-stream storage during low flow.

7.2.2 Log Normal Distribution Method

Table 7.3 Results for Log Normal Distribution Method

T, years	Q, cumec
2	2166.837856
10	4068.431502
25	5122.943679
50	5945.452755
100	6795.702471
200	7684.022119
1000	9892.019057

The value for discharge with **500 years** of return period is interpolated from the Table 7.2 and is **8512.02 cumec.**

7.2.3 Log Pearson III distribution Method

Table 7.4 Results for Log Pearson III Distribution Method

No. of Years	Flood Inflow, Cumec
2	2160.725752
10	4075.198716
25	5151.970007
50	5998.046187
100	6879.782047
200	7804.987552
1000	10133.25366

The discharge value for **500 years** return period is interpolated from the values obtained which comes to be **8678.08 cumec.**

The Design flood value for 500 years return period is taken for the calculation of in-stream volume and PK weir design as recommended in IS 6966 (part 1): 1989, "Hydraulic Design of Barrages and Weirs-Guidelines". Out of the two methods used, the Log-Pearson Type III method value is adopted for the design as it is more authentic and the value coming is higher also.

Therefore the **8678 cumec** discharge value is taken for the design of PK weir near Kharagprasad.

7.3 INSTREAM STORAGE CREATION

7.3.1 Design of a Piano Key weir

The hydraulic design of the Piano Key weir near Kharagprasad site was done. The Piano Key weir is designed of 5 m height. It is designed for river width of 624m. The length of one element for it is 12m. The width of inlet and outlet cell is designed as 3 m. The slope of the ramp is kept as 1.5 to 1 for the safe passage of the sediments. It could be seen from the design configuration that the In-streams storage created with the Proposed Piano Key weir.

7.3.2 Mathematical Modelling of Brahmani River for a stretch 17.25 km stretch

The mathematical modelling of the Brahmani River for 17.25 km reach was carried out to work out the In-stream storage volume during the low flow and high flow condition. The results obtained are as follows:

- Length of Backwater due to incident afflux with P. K. Weir in position (Low Flow) =11 Km
- Volume of In-Stream Storage in Low Flow = 17.701 Million Cubic Meter (MCM)
- Volume of In-Stream Storage in High Flow=97.93 Million Cubic Meter(MCM)
- Water Requirement for Industry / Drinking Water Purpose = 2.1 Cumec
- If minimum flow ceases to exist, the above In-stream storage will cater for 97 days

The output tables for HEC-RAS analysis are presented in Appendix A and Appendix B.

7.4 COST FUNCTION DEVELOPMENT

The analysis was carried out to get the relationship between

- Incremental crest level rise of Piano Key weir and the Additional cost of earthwork.
- Incremental crest level rise of Piano Key weir and the storage volume created In-stream storage.

The equations developed as a result of the analysis performed are as follows.

1. $$y = \frac{a+bx}{1+cx+dx^2}$$

Where y = Additional cost of earthwork, crore

x = Rise in crest level/Rise in bund level, m

r= 0.999 Which shows a very good relationship between x and y

a = 3.32E-08

b = 8.21E08

c = 12591966

d= -722958

2. Equation for crest level rise from 0.5 m to 3.5 m,

$$y = \frac{1}{a + bx^c}$$

Where, y = Amount of In-stream storage volume, days

x = Rise in crest level/Rise in bund level, m

a = 0.0430

b = -0.0327

c = 0.1543

r = 0.978 This shows a very good relationship between the parameters x and y.

The data of Brahmani River have been adopted for the purpose of investigating the feasibility of application of the new concept of In-stream storage as a case study in the Brahmani River. On the basis of present study, the following observations have been emerged.

1. There is a good potentiality to create In-stream storage for various water uses in the study reach by using Piano Key weir. The configuration of Piano Key weir is more conducive than the conventional weirs for storage of significant amount.

2. Adapting the water demand in the present study, the cost functions have been developed for the variations in crest level along with bank level. Using the cost functions, the storage created and the additional cost of earthwork can be obtained for the given rise in the crest level along with a bank level.

3. To assess the stability of flow channel, satellite based analysis was conducted using remote sensing data covering the data of the year 1998 and year 2009. It could be concluded from the study that the river waterway is stable and thus suitable for creation of In-stream storage.

4. From design analysis of Piano Key weir, it could be seen that crest level is suitable to create In-stream storage of 17.701 Mm^3 to meet a requirement of 2.1 cumec for 97 days without undergoing submergence effect. To analyse the creation of In-stream storage volume, HEC-RAS model has been calibrated for the study reach for a length of 11 km which provided the water surface profiles for the 500 year designed flood of 8678 cumec and 95% dependable flow of 80.65.

5. Using the remote sensing data, it has been found that cascade development of In-stream storage site is possible between Samal barrage and Jenapur in the

Brahmani River by deploying the Piano Key weir concept without submergence and minimum stream bed changes.

Present study has indicated the prospects for cost effective application of Piano Key weir to create In-stream storage with inherent hydraulic efficiency.

8.2 SCOPE FOR FUTURE STUDY

- The present study for creation of In-stream storage was carried out using 52 no of cross-sections at 250 m interval. The cross-sections were available for the length of 12.75 km. The cross-sections at the smaller interval can be considered for the study to get more accurate results. The cross-sections for the larger river length can be surveyed and used in the study the river in more realistic manner.
- In the study, only 1 PK weir was designed near Kharagprasad. At the downstream of Kharagprasad also there can be found 4 or 5 barrage sites and the water potential of Brahmani River can be completely harnessed in this reach. By creation of such cascade, the large quantity of water can be stored.
- The DEM data for the study area should be used to get the better idea of water spilling and to get the flood inundation maps.

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APPENDIX A:

Reach	River Station	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Kharagprasad	1	PF 2	8678	46.43	54.67	50.11	54.9	0.000023	2.13	4068.63	540	0.25
Kharagprasad	8	PF 2	8678	51.25	55.57	55.57	57.1	0.000495	5.47	1586.86	520.8	1
Kharagprasad	9	PF 2	8678	50.35	57	57	58.42	0.000514	5.29	1639.89	579.26	1
Kharagprasad	10	PF 2	8678	52	57.87		58.56	0.00016	3.69	2350.22	594.42	0.59
Kharagprasad	11	PF 2	8678	53.83	58.89	58.53	59.81	0.000382	4.26	2039.2	803.32	0.85
Kharagprasad	12	PF 2	8678	52.59	59.39		59.89	0.000091	3.13	2771.17	585	0.46
Kharagprasad	13	PF 2	8678	52.69	59.63		59.93	0.000049	2.41	3594.96	705	0.34
Kharagprasad	14	PF 2	8678	53.43	59.6		59.98	0.000059	2.75	3157.56	585	0.38
Kharagprasad	15	PF 2	8678	51.35	59.89		60.02	0.000019	1.6	5408.96	960	0.22
Kharagprasad	16	PF 2	8678	52	59.78		60.08	0.000056	2.43	3572.22	760.31	0.36
Kharagprasad	17	PF 2	8678	50.15	60.04		60.11	0.000013	1.23	7077.87	1410	0.17
Kharagprasad	18	PF 2	8678	52.07	60.06		60.12	0.000007	1.07	8109.11	1290	0.14
Kharagprasad	19	PF 2	8678	51	60.04		60.13	0.000009	1.3	6666.47	930	0.16
Kharagprasad	20	PF 2	8678	52.11	59.98		60.16	0.000019	1.87	4641.88	660	0.23

Kharagprasad	21	PF 2	8678	51	59.96		60.18	0.00003	2.1	4137.54	705	0.28
Kharagprasad	22	PF 2	8678	44.86	60.1		60.2	0.000006	1.37	6328.72	630	0.14
Kharagprasad	23	PF 2	8678	48.35	60.1		60.2	0.000009	1.44	6015.2	705	0.16
Kharagprasad	24	PF 2	8678	48	60.15		60.21	0.000004	1.03	8431.58	840	0.1
Kharagprasad	25	PF 2	8678	49.3	60.12		60.23	0.000008	1.46	5963.29	660	0.15
Kharagprasad	26	PF 2	8678	49	60.18		60.23	0.000004	1.02	8468.02	975	0.11
Kharagprasad	27	PF 2	8678	47.2	60.19		60.24	0.000003	0.91	9586.2	1050	0.1
Kharagprasad	28	PF 2	8678	47	60.16		60.25	0.000008	1.31	6618.14	840	0.15
Kharagprasad	29	PF 2	8678	46.75	60.22		60.26	0.000003	0.89	9699.8	1168.91	0.1
Kharagprasad	30	PF 2	8678	52.67	60.15		60.29	0.000016	1.61	5395.36	825	0.2
Kharagprasad	31	PF 2	8678	50.29	60.31		60.45	0.001779	1.66	5218.08	840	0.21
Kharagprasad	32	PF 2	8678	50	60.5		60.59	0.000011	1.36	6358.18	945	0.17
Kharagprasad	33	PF 2	8678	49.65	60.53		60.6	0.000006	1.11	7843.36	1035	0.13
Kharagprasad	34	PF 2	8678	49.6	60.53		60.6	0.000006	1.1	7872.13	1035	0.13
Kharagprasad	35	PF 2	8678	50.05	60.4		60.66	0.000031	2.28	3805.8	570	0.28
Kharagprasad	36	PF 2	8678	54.22	60.51		60.68	0.000034	1.83	4751.58	1090.28	0.28
Kharagprasad	37	PF 2	8678	52	60.38		60.79	0.000059	2.84	3056.99	540	0.38
Kharagprasad	38	PF 2	8678	52	60.9		61.25	0.000044	2.6	3339.27	540	0.33

Kharagprasad	39	PF 2	8678	52	61.13		61.45	0.000039	2.5	3464.78	540	0.32

APPENDIX B

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Kharagprasad	1	PF 2	80.65	46.43	51.56	46.96	51.56	0	0.03	2391.97	540	0.01
Kharagprasad	8	PF 2	80.65	51.25	51.76	51.76	51.95	0.003154	1.9	42.54	117.79	1.01
Kharagprasad	9	PF 2	80.65	50.35	52.03		52.07	0.000152	0.93	86.79	70.61	0.27
Kharagprasad	10	PF 2	80.65	52	52.85	52.85	53.07	0.002968	2.08	38.72	89.16	1.01
Kharagprasad	11	PF 2	80.65	53.83	54.98		55.1	0.001148	1.59	50.65	84.41	0.66
Kharagprasad	12	PF 2	80.65	52.59	55.14		55.14	0.000002	0.14	570.69	352.71	0.04
Kharagprasad	13	PF 2	80.65	52.69	55.14		55.14	0.000006	0.15	529.43	604.5	0.05
Kharagprasad	14	PF 2	80.65	53.43	55.14		55.15	0.000004	0.14	579.25	557.73	0.04
Kharagprasad	15	PF 2	80.65	51.35	55.15		55.15	0	0.06	1257.01	735.45	0.02
Kharagprasad	16	PF 2	80.65	52	55.15		55.15	0.000002	0.11	762.53	534.74	0.03
Kharagprasad	17	PF 2	80.65	50.15	55.15		55.15	0	0.04	1948.05	877.3	0.01
Kharagprasad	18	PF 2	80.65	52.07	55.15		55.15	0	0.04	2030.91	1103.56	0.01
Kharagprasad	19	PF 2	80.65	51	55.15		55.15	0	0.04	2119.22	909.57	0.01
Kharagprasad	20	PF 2	80.65	52.11	55.15		55.15	0	0.06	1461.23	642.73	0.01
Kharagprasad	21	PF 2	80.65	51	55.15		55.15	0.000001	0.09	938	416.51	0.02
Kharagprasad	22	PF 2	80.65	44.86	55.15		55.15	0	0.03	3211.98	622.43	0
Kharagprasad	23	PF 2	80.65	48.35	55.15		55.15	0	0.03	2740.99	580.86	0
Kharagprasad	24	PF 2	80.65	48	55.15		55.15	0	0.02	4264.92	800.08	0
Kharagprasad	25	PF 2	80.65	49.3	55.15		55.15	0	0.03	2819.06	589.74	0
Kharagprasad	26	PF 2	80.65	49	55.15		55.15	0	0.02	3634.17	915.75	0
Kharagprasad	27	PF 2	80.65	47.2	55.15		55.15	0	0.02	4313.87	1026.86	0
Kharagprasad	28	PF 2	80.65	47	55.15		55.15	0	0.03	2575.96	706.96	0.01
Kharagprasad	29	PF 2	80.65	46.75	55.15		55.15	0	0.02	3938.14	1089.03	0
Kharagprasad	30	PF 2	80.65	52.67	55.15		55.15	0.000001	0.06	1265.14	825	0.02
Kharagprasad	31	PF 2	80.65	50.29	55.15		55.15	0.000001	0.09	908.04	775.39	0.03
Kharagprasad	32	PF 2	80.65	50	55.15		55.15	0	0.06	1417.92	764.63	0.01

APPENDIX B

aragprasad	33	PF 2	80.65	49.65	55.15		55.15	0	0.04	2285.69	1003.45	0.01
aragprasad	34	PF 2	80.65	49.6	55.15		55.15	0	0.03	2312.44	1003.45	0.01
aragprasad	35	PF 2	80.65	50.05	55.15		55.15	0.000001	0.09	855.69	470.69	0.02
aragprasad	36	PF 2	80.65	54.22	55.09	55.08	55.2	0.003676	1.51	53.53	235.17	1.01
aragprasad	37	PF 2	80.65	52	55.31		55.31	0.000006	0.19	423.11	324.34	0.05
aragprasad	38	PF 2	80.65	52	55.36		55.36	0.000005	0.18	439.24	329.42	0.05