I-D MATHEMATICAL MODEL STUDIES FOR PREDICTION OF LONG TERM BED LEVEL CHANGES IN KOSI RIVER REACH FROM BARRAGE TO 47 Km. DOWNSTREAM

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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NOVEMBER, 2009

CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation entitled, ***"I-D Mathematical model studies for prediction of long term bed level changes in kosi river reach from barrage to 47 km. downstream"***, in partial fulfillment of the requirement for award of the degree of ***Master of Technology in Water Resources Development,*** submitted in Water Resource Development and Management Department, Indian Institute of Technology, Roorkee, is an authentic record of my own work, carried out during the period from Oct 2008 till the date of submission under the supervision of **Dr. Nayan Sharma**, Professor, **Dr. Ashish Pandey** Assist Professor WRD&M Department, Indian Institute of Technology, Roorkee and **Shri. D N Deshmukh** Chief Research Officer **Shri. P V Awate**, Senior Research Officer, CWPRS, Pune.

I have not submitted the matter embodied in this dissertation for the award of any other degree or diploma.

Place : Roorkee
Dated : 16th Nov 2009

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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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CERTIFICATE

This is to certify that, **Mr. SANJAY A BURELE, Trainee Officer, Enrolment No. 076015** has successfully completed the dissertation work for partial fulfillment of the requirement for the award of the degree of Master of Water Resource Development and Management, specialization in Water Resources Development. His Viva-voce Examination has been held on 16th November 2009 with respect to his dissertation work entitled ***"I-D Mathematical model studies for prediction of long term bed level changes in kosi river reach from barrage to 47 km. downstream"***

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ABSTRACT

River Kosi is the third largest Himalayan river, rises in Tibet and after traversing a distance of 720 km. in Nepal and India, joins river Ganga at Kursela. Three main tributaries in the Himalaya viz. Sun Kosi, Arun Kosi and Tamur join together at Tribeni in Nepal and river below Tribeni is Known as Kosi. River Kosi below Tribeni flows through a deep gorge over a length of about 10 km. until it debouches into the plain at Chatra. River below Chatra built up its plain and flows through several channels spread over a width varying from 6km. to 16 km. Kosi river near Bhimnagar, which is 42 km. below Chatra, enter North Bihar in India. Total distance from Bhimnagar to Kursela where river Kosi falls into river Ganga is about 260 km. The important tributaries those join Kosi river in this reach are Trijuga, Balan, Kamala and Bagmati.

The history of the river reveals that, between years 1731 and 1954, the river shifted from east to west for a distance of 112 km. After shifting, it leaves destruction and devastation in its wake, ruining towns and villages, covering agricultural land with sand, turning wide depressions into marshy land ultimately making countryside un-inhabitable and unhygienic. In order to provide relief to the areas affected by Kosi river in North Bihar and Nepal, Kosi project was undertaken in the year 1955. the project comprised:- 1) Construction of flood embankment on both the bank confine flood spread. 2) Construction of barrage near Bhimnagar in Nepal to cater for an annual irrigation of 1.05 m.ha. and power generation of 20,000 kw. Construction of flood embankments was completed in the year 1959 and barrage was commissioned in the year 1963.

Because of construction of embankment, the river flows between it and the sediment/silt start depositing, due to that there is savior attack on embankment. To avoid attack batteries of spur was constructed along the embankments. In this thesis rate of aggradation/degradation was carried out. The rate of aggradation in Kosi river within the flood embankments was worked out using HEC-RAS-4 version.10 daily average inflows was established on the basis of daily discharge data available for the year 1948 to 1966 and recent data from 2001 to 2003. Using sediment concentration and discharge data water and sediment flow relationship was established. This relationship was used for getting the sediment inflow at the upstream boundary. For downstream boundary condition, gauge discharge data available was used. Using these boundary conditions the bed levels were predicted for successive years.

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Chapter 1

INTRODUCTION

1.10 Kosi River

The Kosi river originates in Tibet at an elevation of 5500 meters above MSL by the side of foot hills of Mount Everest and traverses through Nepal and India for a distance of about 720 km. before joining the river Ganga near Kursela. It has 3 major tributaries Sun Kosi, Arun and Tamur, which unite at Tribeni **(Figure1.1& 1.22)**. The river upstream of Tribeni and for about 11km. downstream flows through deep gorge in Himalayas until it enters Gangetic plain at Chatra. The river below chatra builds up its plain and flows through several channels spread over a width varying from 6 to 16 kms. The rivers Trijuga, Balan, Kamala and Bagmati joins river Kosi after entering the plain.

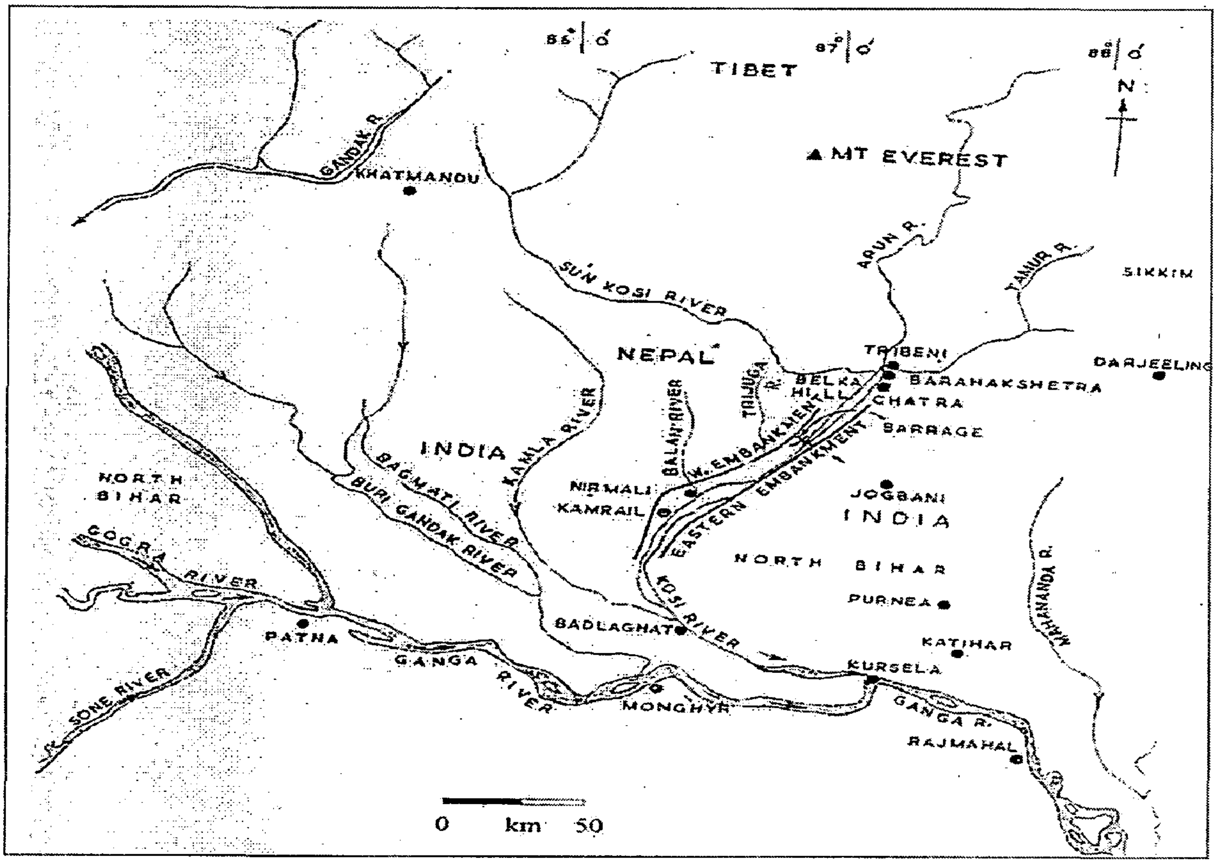


Figure-1.1: Confluence of Kosi river

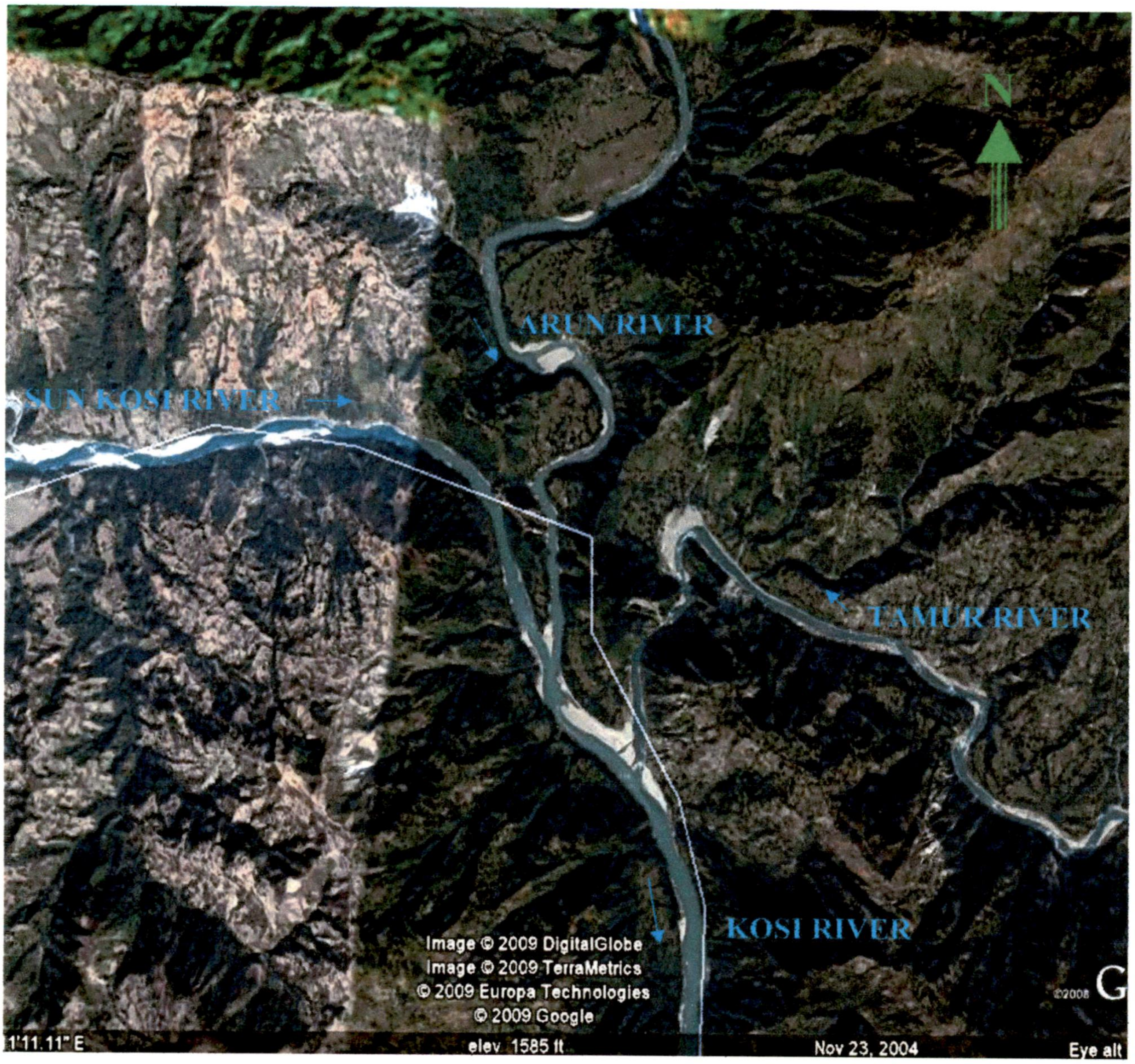


Figure-1.2: The three tributaries joining and becoming Kosi river

1.20 Hydrology of Kosi River

The total catchments area of Kosi river basin up to Chatra is about 58600 sq.kms, and it is divided into Sun Kosi, Arun and Tamur in the portio of 32%, 58% and 10% respectively. The average rainfall in these Catchments varies from 1500 mm to 1250 mm and further decreases to 250 mm in the plain. The average annual runoff measured at Barakshetra is about 53000 million cum (5 million Ham). 81% of this runoff is contributed during June to October. The annual maximum Discharge varies from 5665cum (2 lakh cusecs) to 25910 cumecs (9.15 lakh cusecs)

1.30 Sediment load of Kosi River

The average annual sediment load of Kosi at Tribeni are as follows-

Sediment sizes in mm	Sediment load in Ha-m
Coarse 0.6 to 0.2mm	19000 Ha-m
Medium 0.2 to 0.075	28000 Ha-m
Below 0.075mm	6042 Ha-m

Out of total sediment load, the sediment loads contributed by the river joining are as follows

Percentage of sediment load	River name
50%	Sun kosi
25%	Arun
25%	Tamur

Reasons for heavy Sediment load in Kosi River may be due to following points

1. Uplift and gradual building of Himalayas - The River is very old while the mountain through which it passes is very young. The process of uplift and gradual building of Himalayas are continued for very long period. During the process of uplift, folding and faulting, the river flowed with increased gradient causing erosion of riverbed and banks all along its course.
2. Landslides due to steep valley slopes and relatively soft rock.
3. Seismic activity resulting into loosening and disintegration of shattered rock.

After entering into gangetic plain which has relatively gradual slope this sediment is deposited on plains. In comparison of sediment concentration at Chatra and the sediment concentration at Kursela i.e. at the confluence with Ganga is about 22%. The river starts widening immediately downstream of Chatra (**Figure-1.3**).

Figure-1.3: Showing braided portion of river starting from Belka hills

The braiding process is however seen from Belka hills on downstream, where interlacing channels are spread over width of about 5 to 6 km. The river bed slope in different reaches as per previous 1966 data were as follows:

Chatra	1/570
Hanuman Nagar	1/2400
Baptiyahi	1/5200
Kursela	1/18000

From 15 to 20 km. downstream of Hanuman nagar the river spread into several channels occupying width as high as 15 kms. Specially due to flattening of slope and deposition of sediment. In the process of delta building the Kosi river has been shifted from east to west over a wide area from Mahanadi river on east to Balan river on west. The survey of 1731 (**Figure-1.4**) reveals that the river was flowing west of

Purnia while at present it is flowing along Nirmali. The river is shifted approximately a distance 112 km. in about 230 years. (**Figure-1.4**). After shifting, it leaves. destruction and devastation in its wake, ruining towns and villages, covering agricultural land with sand, turning wide depressions into marshy land ultimately making countryside un-inhabitable and unhygienic. About 7700 sq,km. land on Bihar and 1300 sq.km. in Nepal has turned into wasteland due to sand depositions during process of shifting.(**Figure-1.4**)

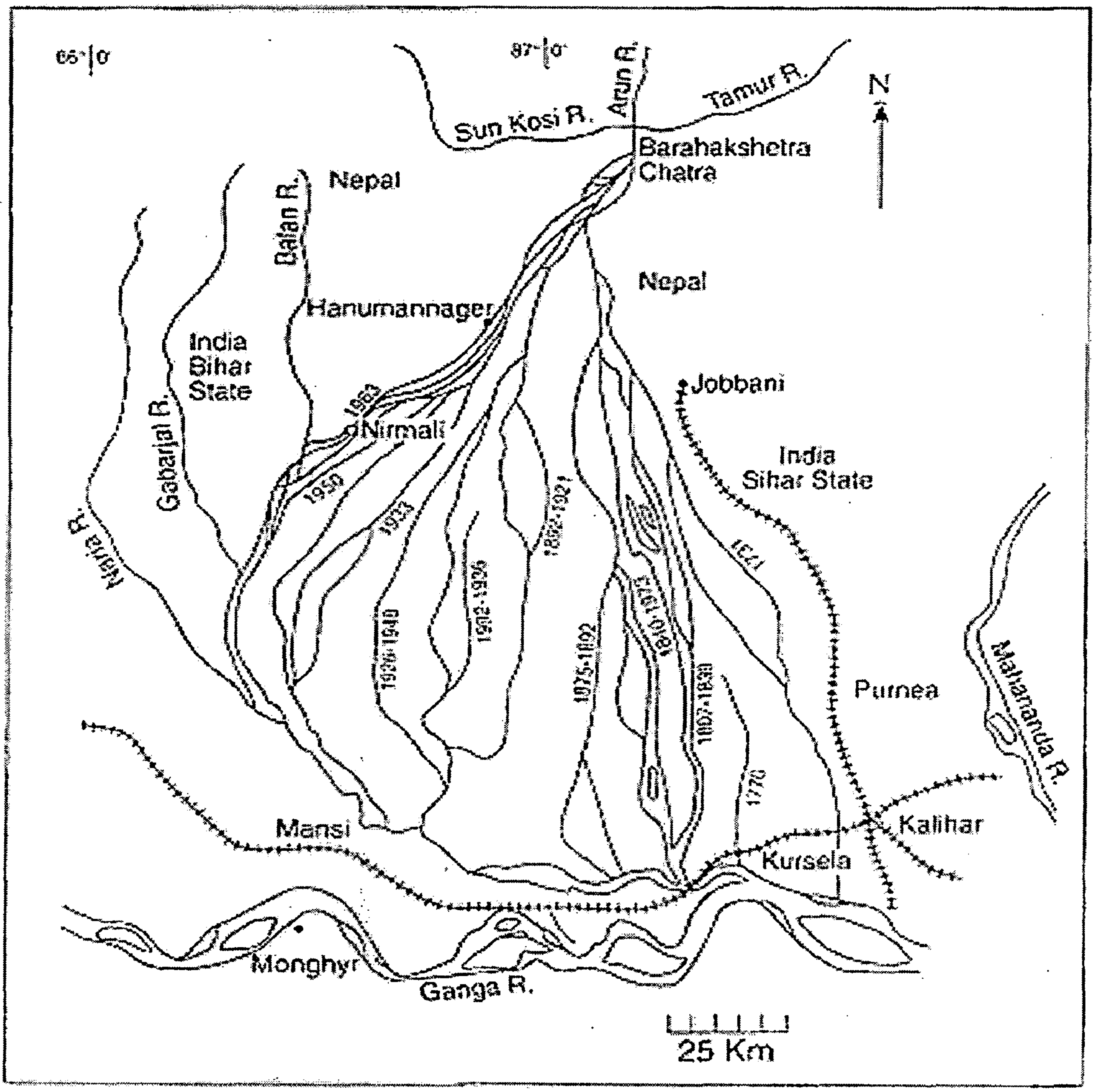


Figure-1.4: Showing shifting of Kosi river

1.40 Kosi Barrage

In order to provide relief to the north Bihar and Nepal, Kosi project was envisaged. This consisted construction of levees on both banks to confine flood spill and construction of barrage near Hanumannagar (**Photo-1.1&1.2**). The work of of construction of about 268 km. levees on either bank was completed by 1959 and the river was diverted through barrage in 1963. Since then in the reach from 40 kms., upstream of barrage to about 100 kms. downstream, river is flowing through the confined reach from 5 to 15 kms. Since the levees/embankments cannot prevent tendency of shifting river course, the river has been attacking levees at different locations during the process of channel shifting within the confined reach (**Figure-1.5**). Number of spurs was constructed to protect the embankment by keeping main river flow away from the bank. Hydraulic model studies of this river reach were carried out at CWPRS to design Kosi Barrage and flood embankments with number of spurs. In addition to this, aggradations of the river bed has been notified in the leveed reach. This resulted into increase in flood levels at different location. Since construction of Barrage and flood embankments the migration tendency of river has been arrested. However, during some of high floods river has breached Eastern and Western embankments at some locations resulting into heavy inundation.

Photo-1.1

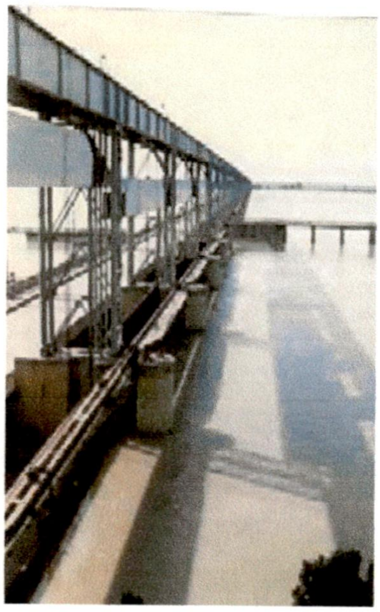


Photo-1.2

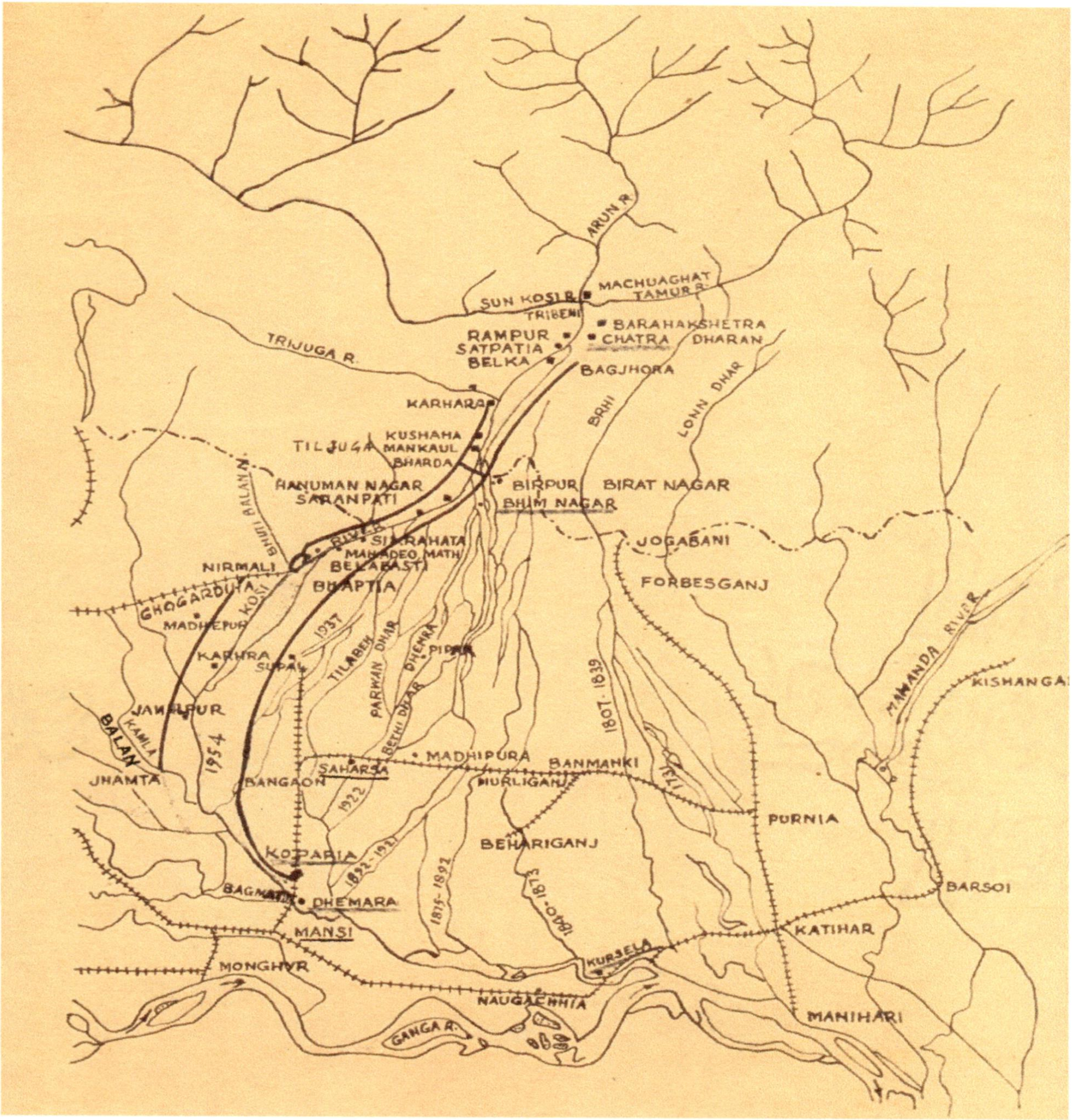


Figure-1.5 Showing Eastern & Western Embankments on river Kosi

1.50 High Floods in Kosi River

Kosi River has experienced high floods of 24300cum/s (6.58 lakh cusecs) and 25910 cum/s (9.15 lakh cusecs) in the year 1956 and 1968 respectively. Yearly maximum flood discharge of Kosi River for the period from 1964 to 2008 are given **vide table 1.1**

Table – 1.1

Year	Date	Discharge at Barrage in cusecs
1964	04.08.64	281946
1965	10.08.65	239309
1966	25.08.66	391042
1967	09.07.67	316094
1968	05.10.68	788200
1969	28.07.69	315020
1970	15.07.70	450400
1971	12.06.71	418100
1972	29.07.72	337361
1973	13.10.73	401935
1974	05.08.74	387818
1975	28.07.75	325384
1976	23.08.76	291183
1977	26.08.77	270610
1978	28.07.78	332483
1979	24.07.79	406813
1980	20.07.80	282500
1981	22.08.81	253828
1982	19.08.82	197219
1983	05.07.83	279157
1984	17.09.84	501787
1985	05.09.85	323844
1986	02.08.86	274158
1987	11.08.87	523771
1988	26.08.88	400190
1989	19.09.89	472413
1990	12.08.90	393475
1991	16.08.91	352009
1992	24.07.92	284729
1993	15.08.93	311482
1994	19.08.94	243212
1995	30.08.95	238314
1996	20.07.96	331229
1997	18.08.97	284868

1998	31.08.98	311629
1999	03.07.99	380358
2000	02.08.00	316917
2001	23.08.01	354771
2002	23.07.02	386910
2003	10.02.03	389970
2004	11.07.04	398669
2005	07.08.05	335316
2006	23.07.06	191948
2007	05.09.07	335298
2008	07.08.08	225227

The following have been the main problem encountered subsequent after the completion of Kosi Barrage, Embankment/Flood protection and its Canal system.

1) Breaching of Embankment
2) Aggradations in the Embankment reach.
3) Excessive sediment deposit in both Eastern and Western canals.
4) Water logging in the adjoint area of river and canal due to aggradation of river.

In this thesis problem has been focused only on first two mentioned above.

It is seen from last many years there is a under attack of flood every year on the Eastern and Western Embankment, So movable bed model with rigid embankment has been constructed in CW&PRS, Pune. After recession of flood every year survey data is reproduced in model with changed cross section and, On the basis of model studies various protective measure are suggested on the advice of high level committee by the CW&PRS, Pune.

1.60 Scope of Study:-

The dissertation topic which is approved by Director Mrs. V M Bendre is "1D Mathematical model studies for prediction of long term bed level changes in Kosi river reach from barrage to 47 km. downstream", the rate of aggradation within the flood embankments would be worked out using HEC-RAS-4 version.

10 daily average inflows will be established on the basis of daily discharge data available for the year 1948 to 1966 and recent data from 2001 to 2008. Using sediment concentration and discharge data water and sediment flow relationship will be established. This relationship will be used for getting the sediment inflow at the upstream boundary. For downstream boundary condition gauge discharge data available will be used. Using these boundary conditions the bed levels were predicted for successive years.

For proving studies and for manning coefficient fixation the Cross Section for the year May 2002 and the maximum discharge recorded at Kosi barrage for same year is used as follows

1) For maximum discharge of 10660 cum/s at Kosi Barrage in the year 2002, the corresponding water levels recorded at Dagmara (C/S No. 22) and Bhaptiahi (C/S No. 32) were 64.30m and 60.20m respectively will be used.

2) Similarly a discharge Q = 2000cum/s, who's the water level recorded in May 2002 was used.

Chapter 2

REVIEW OF LITERATURE

2.1 GENERAL

In order to provide relief to the areas affected by the river in North Bihar and Nepal, the Kosi project was undertaken. The project consisted of a) the construction of flood embankments on both the banks, to confine spills that formerly spread far beyond, b) the construction of a barrage, near Bhimnagar, which could raise the water level in the pond by about 2.44m and provide an annual irrigation of 1.05 million ha. and power generation of about 20,000 kw. The work of constructing 240 km of flood-embankment was started in 1955 and was completed by 1959 and the river was diverted through the barrage in 1963. Spurs also known as groyons or dykes were constructed along the embankments, where required from time to time. The river regime of expected to be affected significantly by both these measures. In order to ensure safety of the embankments and the barrage and to give them, as a long as a possible, intimate knowledge of the river behavior and the ability to forsee possible river changes is imperative. This would be possible only with proper understanding of the causes of shifting courses and of the processes associated with the delta building activities.

The embankment were constructed in 1959, the barrage in 1963 and the eastern canal in 1964. The following has been the main point that have been encountered subsequently to the completion of most of the above phases

1) Breaching of sections of the embankments leading to huge cost of maintenance.
2) Aggradation in the embankment reach.
3) Excessive sediment deposition in eastern Kosi canal
4) Water logging in the command area of eastern Kosi canal.
5) Underutilization of the irrigation potential created.

The importance of lateral shifting and aggradation of Kosi studies was carried by large number of scientists.

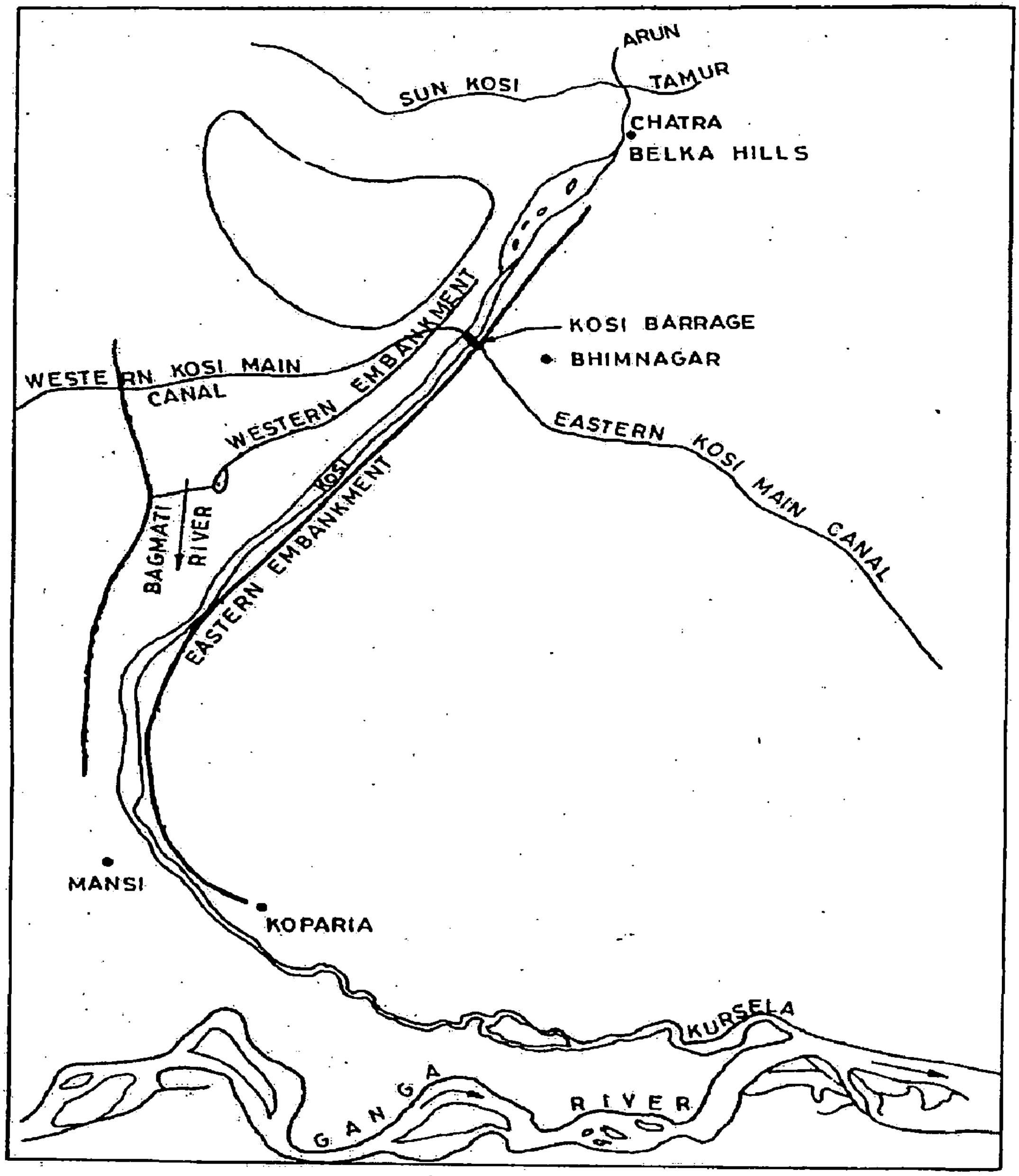


Figure – 2.1: Showing Chatra to Mansi Reach

2.2 Literature Review

Dr. Garde and others (Ref. # 1)

The rate of aggradation of within the flood embankments was work out using HEC-6 model by **Dr. Garde and others** using field data from 1975 to 1982 for calibration. Dr. Garde and others was essentially concern with the morphology of the river and attention was focused only on the first two problems mentioned above. The cross sectional data considered for study of reach from Chatra to Mansi for the year May 1975 was used **(Figure-2.1)**

Input Data : Input data to the HEC-6 computer programme was grouped into 3 categories

1) Geometric data : a) Cross section and reach length : For the year 1975 cross section are used and the reach length is from Chatra to Mansi. (b) Manning's n value: value of n is assumed as 0.02 over the entire reach under consideration. (c) Movable bed width : a value of 20 feet was used as a scour depth of the sediment

2) Sediment data: a) Inflowing sediment load:- The inflowing sediment load is given as input in the form of a total sediment load versus water discharge relationship (b) **Gradation of stream** – bed material:- The fraction of sediment bed material contained in each grain size is required to be given as input to describe the stream – bed material gradation

The following was the main conclusions of the study

(1)The proximity of the deep channel to a levee to the extent of 4000-5000m, a current at an angle to levee from such a channel and a lateral migration of the deep channel of the order of 200m per year in any reach may pose danger to a levee in the direction of which the channel is migration.

(2) Use of Lauren –Madden transport law and a Manning's n value of 0.02 in HEC-6 is seen to satisfactorily reproduce the observed bed levels of the Kosi river.

(3)Aggradation (with respect to the levels of 1984) of the order of 8 ft (2.44m) may occur in the reach by 2005A.D.with the levees in their present positions. Such aggravation may being the water surface to within 4.5 ft (1.37m)to 7ft(2.13m) of the top of the levee in the reach between sections 63and 91.

(4) Uniform reduction of river width in the entire levered reach is not a feasible solution to the large aggradation noticed in the river, but selective reduction in width does offer a good solution.

Shri Gole and Chitale (Ref. # 2)

The processes of the shifting of the courses of the river and the delta building activities have been investigated by Shri Gole and Chitale. It has been concluded by the authors that the shifting of the courses of the Kosi river was cause by the deficient river slope, which is not sufficient to carry the excessive sediment load brought down by the river. The river in its natural course could have continued building the delta and ultimately achieved such as stable slope, probably along the line running straight south from Chatra, so that all the sediment could be carried down the river, with progressive changes in the bed material load, caused by attrition and sorting. Since the Kosi Project has hindered the processes involved in the delta building activities of the river, a study of possible repercussions of the human interference is essential. In this context, study of the effect of barrage and the embankments is imperative. **(Figure-2.2)**

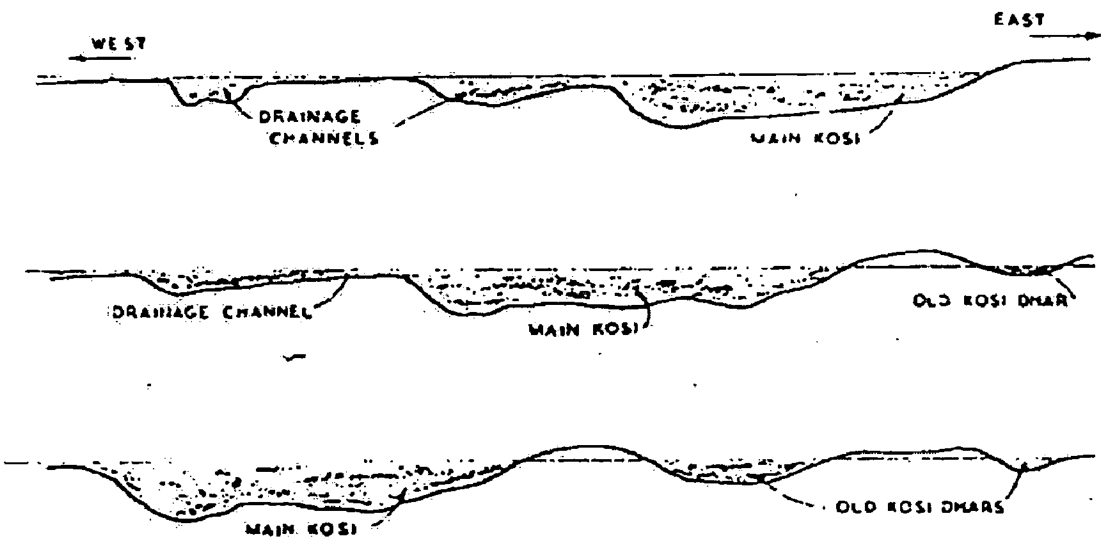


Figure – 2.2: Shifting of the courses

Shri R. Ghosh (Ref. # 3)

Cubature studies based on post flood 1963 and 1970 surveys were also made. These indicated that about 35.05 million cubic meter of sediment had deposited in the pond length of about 10 km upstream of the barrage, giving and average depth of about 0.4 m in about 8 years with a rate of rise working at about 0.05 m per annum.

Kosi project. (Ref. # 4)

The Kosi Irrigation Committee has brought out a report on the behaviour of the river as well as on the performance of the Kosi project.

I I T Delhi (Ref. # 5)

A detailed cubature study of the data collected from 1854 to 1974 was carried out by I I T Delhi to indicate reach wise tendency for aggradation and degradation.

Shri. Danju (Ref. # 6)

Danju has analysed land sat imageries of the period 1972-75 to study the shifting of the river between levees and also the meandering of the Kosi as well as the Ganga near their confluence.

2.3 DETAILS OF ONE DIMENSIONAL MATHEMATICAL MODEL

2.3.1 HEC-RAS Software

Hydrologic Engineering Centre of U.S. Army Crops of Engineers, USA has designed and developed River Analysis System (HEC-RAS, Version 4.0, March 2008).

The U S Army Crops of Engineers, River Analysis System (HEC-RAS) is software that allows to perform one- dimensional steady and unsteady flow river hydraulic calculation.

HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking, multi-user network environment. The system 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 contains three one-dimensional hydraulic components for: 1) Steady flow computation; 2) Unsteady flow simulation; and 3) movable boundary sediment transport computation. 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 the basic water surface profiles are computed.

2.2.2 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 modeling system is intended for calculating water surface profiles for steady gradually varied flow. The system can handle a single river reach, a dendritic system, or a full network of channels. The steady flow component is capable of modeling 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 evaluates by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situation where the water surface profile is rapidly varied. These situation include mixed flow regime calculations (i.e., hydraulic jumps), hydraulics of bridges and evaluating profiles at river confluences. The effect of various obstructions such as bridges, culverts, weirs, spillways and 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 for assessing the change in water surface profiles due to channel improvements and levees. Special features of steady flow component include: multiple plan analysis: multi profile computations: multi bridge and culvert opening analysis, and split flow optimization at stream junctions and lateral weirs and spillways.

Unsteady Flow Simulation:- The component of the HEC-RAS modeling system is capable of stimulating one dimensional unsteady flow through a full network of open channels. The hydraulic calculations for cross-sections, bridges, culverts, and other hydraulic structures that were developed for steady flow component were incorporated into the unsteady flow module. Additionally, the unsteady flow component has the ability to model storage areas and hydraulic connections between storage areas, as well as between stream reaches.

Sediment Transport/Movable Boundary Computations:- This component of the modeling system is intended for simulation of one-dimensional sediment transport/movable boundary calculations resulting from scour and deposition over moderate time periods (typical years, although application to single flood events will be possible). The sediment transport potential is computed by grain size fraction, thereby allowing the simulation of hydraulic sorting and armoring. Major features include the ability to model a full network of streams, channel dredging, various levee and encroachment alternatives and the use of several different equations for the computation of sediment transport. The model is designed to stimulate long term trends of scour and deposition in a stream channel that might result from modifying the frequency and duration of the water discharge and stage or modifying the channel geometry. This system can be used to evaluate deposition in reservoirs, design channel contractions required to maintain navigation depths, predict the influence of dredging on the rate of deposition, estimate maximum possible scour during large flood events and evaluate sedimentation in fixed channels,

The HEC-RAS system is capable of modeling one dimensional unsteady flow through a full network of open channels. The unsteady flow components were developed for subcritical flow regime calculations. In both the cases, steady and unsteady flow simulations various obstructions such as bridges, culverts, weirs, spillways, and other structures in flood plain were incorporated in flow module. The physical laws which governs the flow of water in a stream are

- the principle of conservation of mass (continuity)
- the principle of conservation of momentum

These laws are expressed in terms of mathematical expressions as continuity equation and momentum equation.

Continuity Equation:

Conservation of mass for a control volume states that the net rate of flow into the volume be equal to the rate of change of storage inside the volume. The final form of continuity equation is as follows

$$\frac{\partial Q}{\partial t}+\frac{\partial Q}{\partial x}-q_1=0$$

Where, Q = Discharge, q_1 = lateral inflow per unit length, A = total cross sectional flow area, x = distance along the channel, t = time

Momentum Equation:

Conservation of momentum for a control volume states that net rate of momentum entering the volume (momentum flux) plus the sum of all external forces acting on the volume is equal to the rate of accumulation of momentum. The final form of momentum equation is as follows and application of momentum principle is shown in *Figure 2.3.*

$$\frac{\partial Q}{\partial t}+v\frac{\partial Q}{\partial x}+gA\frac{\partial z}{\partial x}+gAS_f=0$$

where, S_f = slope of the energy grade line (friction slope)

A_i = wetted cross sectional area

v = velocity through area

$\frac{\partial z}{\partial x}$ = water surface slope

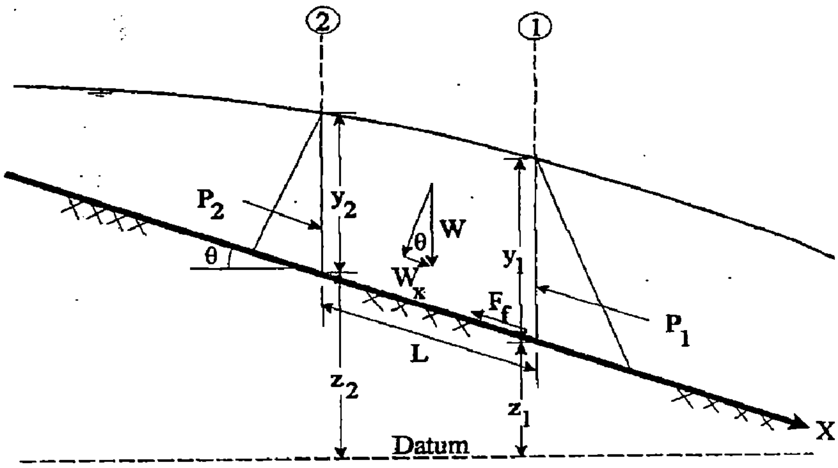


Figure 2.3 : Application of momentum principle

Numerical Solution: Implicit Finite Difference Scheme

The most successful and accepted procedure for solving one dimensional unsteady flow equation is the four-point implicit scheme, which also known as the box scheme. Under this scheme, space derivatives and functions values are evaluated at an interior point, $(n+\vartheta)\Delta t$. thus values at $(n+\vartheta)\Delta t$. enter into all terms in equations. For a reach of river, a system of simultaneous equations results. The simultaneous solution is an important aspect of this scheme because it allows information from the entire reach to influence the solution at any one point. Consequently, the time step can be significantly larger than with explicit numerical schemes. Von Neumann stability analyses shows that the implicit scheme to be unconditionally stable (theoretically) for $0.5 \leq \theta \leq 1.0$, conditionally stable for $\theta = 0.5$, and unstable for $\theta \leq 0.5$. In practice, may other factors contribute to the non stability of the solution scheme. These factors include dramatic changes in channel cross sectional properties, abrupt changes in channel slope, characteristics of the flood wave itself, and complex hydraulic structures such as levees, bridges, culverts, weirs, and spillways.

BOUNDARY CONDITIONS: The model simulates steady and unsteady water surface profile, sediment and contaminant movement in a simple or complex system of channel. For a reach of river there are N computational points or nodes which bound (N-1) finite difference cells. From these cells (2N-2) finite difference equations can be developed. There are two unknowns ΔQ and ΔZ for each node, two additional equations are needed. These equations are provided by the boundary conditions for each reach. For sub-critical flow, upstream and down stream boundary conditions are required and for supercritical flow, boundary conditions are required only at the upstream end.

Boundary conditions must be provided at all of the open ends of the river system being modeled. Unsteady flow data consists of boundary conditions (external and internal), as well as initial conditions. Upstream ends of the river system can be modeled with boundary conditions such as flow hydrograph, stage hydrograph, flow and stage hydrograph. Downstream ends of the river system can be modeled with boundary conditions such as rating curve, normal depth(Manning's equation), stage hydrograph, flow hydrograph, stage and flow hydrograph.

Boundary conditions can also be provided at internal locations within the river system. The user can specify boundary conditions at internal cross sections such as lateral inflow hydrograph, uniform lateral inflow hydrograph, groundwater interflow.

Upstream Boundary Conditions:

Upstream boundary conditions are required at the upstream end of all reaches that are not connected to other reaches or storage areas. An upstream boundary condition is applied as a flow hydrograph of discharge versus time.

2.2.5.2 Interior Boundary Conditions For Reach Connections or at Junctions:

A network is composed of a set of 'm' individual reaches. Interior boundary equations are required to specify connections between reaches. Depending on the type of reach junctions one of two equations is used. Apply flow continuity equation to reaches upstream of flow splits and downstream of flow combination (reach 1 in *Figure 2.4*) only one flow continuity boundary equation is used per junction. Apply stage continuity for all other reaches (reach 1&2 in *Figure 4.3*) Zc is computed as the stage corresponding to the flow in reach 1. Therefore stages in reaches 2& 3 will be set equal to Zc.

Downstream Boundary Conditions:

Downstream boundary conditions required at the downstream end of all other reaches which are not connected to other reaches or storages areas. There our types of boundary condition.

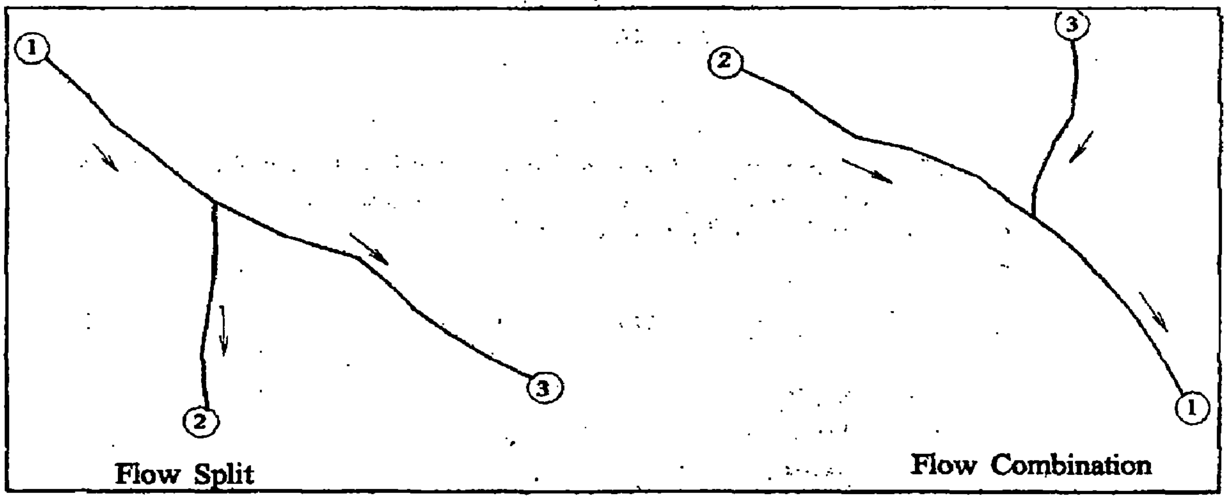


Figure- 2.4 : Typical flow split and flow combination

Stage Hydrograph:

A stage hydrograph of water surface elevations versus time may be used as the downstream boundary condition if the stream flows into the back water environment such as an estuary or bay where the water surface elevation is governed by tidal fluctuations or where it flows into a lake or reservoir of known elevation.

Flow Hydrograph:

A flow hydrograph is used as down stream condition if recorded gauge data from a g-d site is available and model is being calibrated to a specific flood event.

Single Valued Rating Curve:

The single valued rating curve is a monotonic function of stage and flow. An example of this type of curve is the steady, uniform flow rating curve. The single valued rating curve is used to accurately describes the stage-flow relationship of the free outfalls such as water falls or hydraulic control structures such as spillways, weirs or lock and dam operations. When applying this type of boundary condition to a natural stream, caution should be used which may create errors after introduction.

Normal Depth:

If the uniform flow condition exits in channel, use of Manning's equation with a user entered friction slope produces a stage considered to be normal depth. Generally uniform flow condition does not exist in natural channel or streams, this boundary condition should be used for enough downstream from study area so that it does not affect the results in the study area.

Cross Section Geometry:

Boundary geometry for the analysis of low in natural streams is specified in terms of ground surface profiles that are cross sections and the measured distances between them that is the reach lengths. Cross sections are located at the regular or irregular intervals along a stream to characterize the flow carrying capacity of the stream and its adjacent flood plain. They should extend across the entire flood plain and should be perpendicular to the anticipated flow lines.

Cross sections are required at representative locations throughout a stream reach and at locations where changes occurs in discharge, slope, shape, or roughness, at locations where levees begins or end and at bridges or at control structures such as weirs.

Roughness coefficient or Manning's roughness:

The selection of an appropriate value for Manning's roughness is very significant to the accuracy of the computed water surface profiles. The value of Manning's roughness is highly variable and depends on a numbers of factors which includes surface roughness, vegetation, channel irregularities, channel alignment, scouring and deposition, obstructions, size and shape of the channel, stage and discharge, seasonal changes, temperature, and suspended material and bed load. The Manning's roughness factor based upon the type of channel, material used and its values are readily available in HEC-RAS reference manual or any other text books

2.4 Sediment Modeling

Sediment transport modeling is notorious difficult. The data utilized to predict bed change is fundamentally uncertain and the theory employed is empirical and highly sensitive to a wide array of physical variables. However, with good data, a skilled modeler can utilize a calibrated sediment model to predict regional, long term trends that can inform planning decisions and can be used to evaluate project alternatives. HEC-RAS now includes the framework with which to perform mobile boundary, sediment transport modeling. The assumption and theory used are as follows

2.4.1 Quasi-Unsteady Flow

Before HEC-RAS can compute the sediment transport, the river hydraulics must first be determined. HEC-RAS uses a hydrodynamic simplification, a common approach used by many sediment transport models. The quasi-unsteady flow assumption approximates a continuous hydrograph with a series of discrete steady flow profiles. For each record in the flow series, flow constant over a specified time window for transport. The steady flow profiles are easier to develop than a fully unsteady model, and program execution is faster. Each discrete steady flow profile is divided, and subdivided into shorter blocks of time for sediment transport computations. HEC-RAS utilizes three different steps, each a subdivision of another. The three time steps are the Flow Duration, the Computation Increment and the Mixing Time Step.

2.4.1.1 Flow Duration

The flow duration is the coarsest time step. It represents the length of time over which flow, stage, temperature, or sediment loads are assumed constant **(figure:-5)**

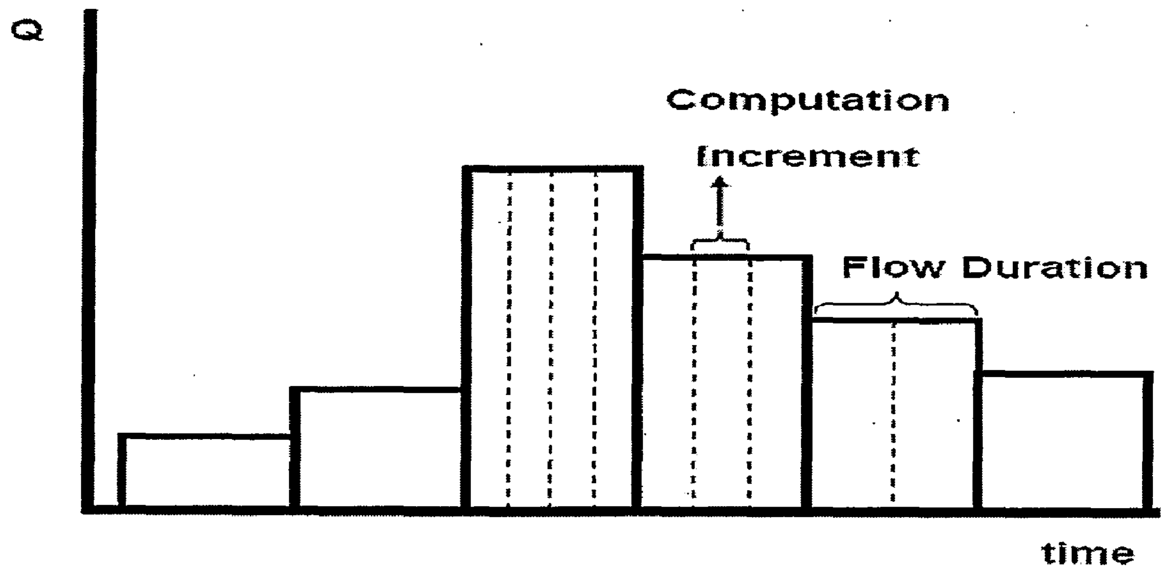


Figure :-2.5: A Quasi-Unsteady Flow Series with time step

For instance, if the flow data was collected daily, the flow duration would be twenty-four hours unless smaller time steps were interpolated. To specify a constant stage, flow, temperature, or sediment inflow, a single value can be associated with a very large duration which, if enough, will set the parameter for the whole run.

2.4.1.2 Computational Increment

The flow duration is further sub- divided into a computational increment **(figure:-5)**. Although flow remains the same over the entire flow duration, the bed geometry and hydrodynamics are updated after each computational increment. Model stability can be sensitive to this time step. When the computational increment is too long, the bed geometry is not updated frequently enough and the model results can vary.

2.4.1.3 Bed Mixing Time Step

Finally, computational increments are further subdivided into the bed mixing time step. During each mixing time step in a computation increment, bathymetry, hydraulic parameters, and transport potential for each grain size remains constant. However, the computations for sediment erosion and deposition take place during this time step and this can cause changes to the composition of the bed mixing layers. The vertical gradational profile is rearranged in response to the removal or addition of material. Since active layer gradation changes during the bed mixing time step, the sediment transport capacity changes even when the hydrodynamics- and therefore, the transport potential- remains constant.

2.4.2 Sediment Continuity

The HEC-RAS sediment routing routines solve the sediment continuity equation also known as the Exner equation

$$(1-\lambda_p)B\frac{\partial \eta}{\partial t} = -\frac{\partial Q_s}{\partial x}$$

where:

where:	B	= channel width
	η	= channel elevation
	λ_p	= active layer porosity
	t	= time
	x	= distance
	Q_s	= transported sediment load

This equation simply states that the changes of sediment volume in a control volume (e.g. aggradation or degradation) is equal to the difference between the inflowing and outflowing loads **(figure-6)**

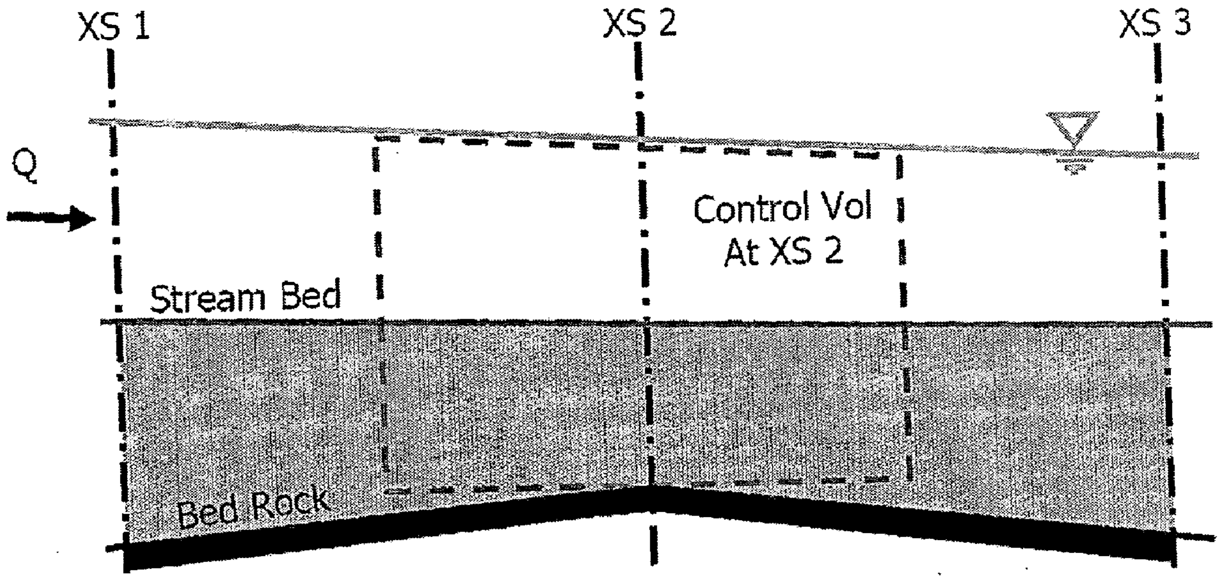


Figure :-2.6 :Schematic of the control volume used by HEC-RAS for sediment calculation

The sediment continuity equation is solved by computing a sediment transport capacity through the control volume associated with each cross section. This capacity is compared to the sediment supply entering the volume. If capacity is greater than supply there is a sediment deficit which is satisfied by eroding bed sediments. If supply exceeds capacity there is a sediment surplus causing material to deposit.

2.4.3 Computing transport capacity

The right hand side of the continuity equation is the sediment gradient across the control volume comparing the sediment inflow with the sediment outflow. Sediment inflow is simply the sediment entering the control volume from the upstream control volume (s)and any local sources (lateral sediment inflows) .The maximum amount of sediment that can leave the control volume ,however , is a function of the amount of sediment that the water can move .This is referred to as the sediment transport capacity , and it is computed for each control for each bed mixing time step .

2.4.3.1 Grain Classes HEC-RAS divides the sediment material into multiple grain classes. The range of transportable material, between 0.002 and 2048 mm, is divided into 20 grain classes or bins that contain adjacent, non-overlapping portions of the grain size spectrum. The default grain classes are based on a standard log base 2 scale where the upper bound of each class is twice the upper bound of the adjacent, smaller class. All of the particles in each grain class are represented by a single, representative grain size. HEC-RAS uses the geometric mean of the grain class (the square root of the product of the upper and lower bounds) to represent the grain size for each bin.

2.4.3.2 Sediment Transport Potential

Sediment transport potential is the measure of how much material of a particular grain class a hydrodynamic condition can transport. Transport potential is computed with one of the number of sediment transport equation available in the program. Since most of this equations were developed to be used for a single grain size, like the d50 (or at the most, two grain sizes like d_{50} and the d_{90}), the equation is applied independently to each grain class present in the system. This value, computed separately for each grain class regardless of their prevalence in the bed is called the transport potential. There are currently seven sediment transport potential functions in the HEC-RAS. There are dozens of transport functions that have been developed. Since sediment transport is sensitive to so many variables, the potentials computed by the different equations can vary by order of magnitude, depending on how the project material and hydrodynamics compare to the parameters over which the transport function should be selected that was developed for similar gradation and hydraulic parameters as found in the project of interest.

2.4.3.2.a) Acker and White

Acker and White (1973) is a total load function that was developed from flume data for relatively uniform gradations ranging from sand to the fine gravels. Hydrodynamic were selected to cover a range of bed configurations including ripples, dunes and plane bed conditions. Suspended sediment is a function of shear velocity while bedload is a function of shear stress.

2.4.3.2.b) England Handersen

England Handersen (1967) is a total load transport equation that was developed flume data. Relatively uniform sand sizes between 0.19 to 0.93 mm were used. The attraction of England Handersen is that it is not complicated function. Instead, it is relatively simple function of channel velocity, bed shear and the d50 of the material. Application should be restricted to sand systems.

2.4.3.2.c) Laursen-Copeland

Laursen (1968) is also total load function that was initially based on flume equation and later expanded by Madden to include the Arkansas river data. It is a *basic function* of *excess shear* and a *ratio* of the shear velocity to the fall velocity,. Later, Copeland (1989) generalized the equation for gravel transport so the equation could be used for graded beds. The distinctive future of Laursen is that the sediment material the function was developed for extends down into the silt range. None of other functions currently included in RAS were developed for silt sized particles. Any sediment potentials computed for silt, by the other function, would be extrapolation, compounding extrapolation errors on top of the standard uncertainty associated with computing transport capacity. Recent work at Colorado State has demonstrated that the Laursen equation out performs other transport function in the silt range.

2.4.3.2.d) Meyer-Peter Muller

the Meyer-Peter and Muller (MPM) equation (1948) was one of the earliest equation developed and is still one of the most widely used. It is a simple excess shear relationship. It is strictly a bed load equation developed from flume experiments of sand and gravel under plane bed condition. Most of the data was developed for relatively uniform gravel substrates-MPM is most successfully applied over the gravel range. It tends to under predict the transport of finer material. Recently, Wong (2003) and Parker (2007) demonstrated that this function over predicted transport by, approximately, a factor of two. This conclusion was not based on new data but on a reanalysis of MPM's original results. To improve the function , they recast the base , excess shear equation:

$$q_b^* = 8(\tau^* - \tau_c^*)^{3/2} \quad , \quad \tau_c^* = 0.047$$

as

$$q_b^* = 3.97\,(\tau^* - \tau_c^*)^{3/2} \quad , \quad \tau_c^* = 0.0495$$

Where: q^*_b is the Einstein bedload number (correlated with bedload), τ^* is the Shield's stress which is compared to, τ^*_c which is the 'critical' Shields stress.

2.4.3.2.e) Toffaleti

Like England – Hansen, Toffaleti (1968) is a load function development primarily over sand sized particles. Toffaleti is generally considered a `large river 'function however, since many of the data sets used to develop it were large, suspended load systems. The function is not heavily dependent on shear velocity or bed shear. Instead, it was formulated from regressions on temperature and an empirical exponent that describes the relationship between sediment and hydraulic characteristics.

A distinctive approach of the Toffaleti function is that it breaks the water column down into vertical zones and computes the concentration of each zone with a simple approximation of a Rouse concentration profile. Transport for each zone is computed separately. This approach is, obviously, most appropriate for transport with significant suspended load such that a vertical Rouse distribution included significant concentrations in the water column. The function has been used successfully on large systems like the Mississippi, Arkansas, and the Atchafalaya Rivers.

Additinally, the Toffaleti equation uses two different grain sizes, a d_{50} and a d_{65}, in an attempt to quantify transport dependence on the gradational deviation from the mean. This made more sense when the equation was used to compute the transport of the bulk gradational material. When it is applied to the individual grain classes, it will use the d_{50} and d_{65} for the given grain class, stretching the original intent of the d_{65} parameter a bit.

2.4.3.2.f) Yang

Yang (1973, 1984) is a total load transport equation which bases transport on Stream Power, the product of velocity and shear stress. The function was developed and tested over a variety of flume and field data. The equation is composed of two separate relations for sand and gravel transport. The transition between sand – gravel is smoothed over in order to avoid large discontinuities. Yang tends to be very sensitive to stream velocity, and it is more sensitive to fall velocity than most.

2.4.3.2.g) Wilcock

Wilcock (2001, generalized from of the initial two fraction equation in Wilcock and Crowe, 2003) is a bed load equation designed for graded beds containing both sand and gravel. It is a surface transport method based on the theory that transport is primarily dependent on the material in direct contact with the flow. It was developed based on the surface gradations of flumes and rivers. Therefore, the bed gradations should reflect the bed surface properties. Wilcock, additionally has a hiding function that reduces the transport potential of smaller particles based on the premise that they are nestled between larger gravel clasts and do not experience the full force of the flow field (or the turbulent boundary layer).

Finally, the central theory of the Wilcock equation is that gravel transport potential increases as sand content increases. A dimensionless reference shear is computed for the substrate which is a function of the sand content of the bed surface:

$$\tau^*_{rm} = 0.021 + 0.015 \cdot e^{-20 \cdot FS}$$

τ^*_{rm} is the reference shear stress and FS is the sand content in percent. As the sand content increases: the reference shear decreases, the excess bed shear increases, and the total transport increases. The Wilcock equations is very sensitive to this sand content parameter. It tends to be most appropriate for bimodal systems and tends to diverge from the other equations for unimodal gravel or sand transport.

2.4.3.2 Transport Capacity

Once transport potential is computed for each grain class, a total, single representative transport for the actual system gradation has to be computed. Since each potential was computed with no reference to the actual abundance of the grain class (i.e. transport potential is computed as if the system was composed of 100% of that grain class), the grain class potential must be prorated based on its relative amount.

The transport capacity for each grain class is the transport potential multiplied by the percentage of that grain class in the bed. Therefore, the total transport capacity is :

$$T_c = \sum_{j=1}^{n} \beta_j T_j$$

Where T_c is Total transport capacity, n is the number of grain size classes, B_j is the percentage of the active layer composed of material in grain size class "j" and T_j is the transport potential computed for the material in grain class "j". this is based on Einstien(1950) classic assumption that the sediment discharge of a size class.

The continuity equation is applied to each grain class separately. Total capacity is not used anywhere in the program. Capacity computed is compared to supply for each grain class and surplus or deficit is determined for that grain class.

2.4.4 Continuity Limiters

The continuity equation compares the transport capacity to the inflowing load for each grain class for each time step. If the capacity exceeds the supply a deficit is computed. If the supply exceeds capacity the control volume has a surplue of the grain class. In general, surplus becomes deposition and deficit is translated into erosion. However, the difference between supply and capacity cannot be directly

Converted into a bed change because there are physical constraints on the process of deposition and erosion. HEC-RAS models these constraints with three basic limiters: a temporal deposition limiter, a temporal erosion limiter, and the sorting and armoring algorithms that provide an additional constraint of erosion

2.4.4.1 Temporal Deposition Limiter

The temporal constraint on deposition is the limiter based on the simplest and most robust theory. There is a well established theory for how fast particles can drop out of the water distance a particle has to travel fall velocity. By comparing the vertical distance a particle has to travel to reach the bed surface and the vertical distance a particle travels in a time step (fall velocity * time) , HEC_RAS will determine what percentage of the sediment surplus can actually deposit in a given control volume in a given time step. A deposition efficiency coefficient is calculated for each grain class (i)

$$C_d = \frac{V_s(i) \cdot \Delta t}{D_e(i)}$$

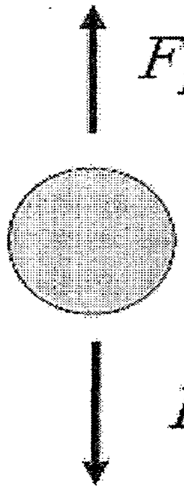
Where C_d is the deposition efficiency coefficient, V_S (i) is the fall velocity for the grain class, Δt is the time step, and D_e is the effective depth of the water column over which the grain class is transported.

The coefficient is a fraction such that if the product of the fall velocity and the time step duration is less than the effective depth, the amount of the surplus that can be deposited in the control volume is reduced proportionally. If the denominator is greater than the numerator, all of the surplus sediment is translated into deposition. To generate this parameter, two variables must be computed: fall velocity and the effective transport depth.

2.4.4.1.a) Fall Velocity

Most fall velocity theories are derived by balancing the gravitational force and the drag force on a particle falling through the water column. The free body diagram is included in **(Figure :-7)**

$$F_D = \frac{1}{2}\pi\rho c_D\left(\frac{D}{2}\right)^2 v_s^2$$



$$F_g = \frac{4}{3}\pi\rho Rg\left(\frac{D}{2}\right)^3$$

Figure :-2.7: Free Body diagram used for computing fall velocity.

Applying these equation for fall velocity is a little more complex than it original might seem. When they are balanced and solved for fall velocity, fall velocity turns out to be a function of the drag coefficient C_D which is a function of the Reynolds number which itself a function of fall velocity. This requires either some kind of approximation for the drag coefficient/Reynolds number or an iterative solution.

Rubey circumvented this dependency with an assumed property and built a simple, analytical function for fall velocity. Toffaleti developed empirical, fall velocity curves that are based on experimental data which accounted for this dependency. Van Rijn used Rubey as an initial guess and then computed a new fall velocity from experimental curves based on the Reynolds number computed from the initial guess. Finally Report 12 is an iterative solution that uses the same curves as Van Rijn but uses the computed fall velocity to compute a new Reynolds number and continues to iterate until the assumed fall velocity matched the computed within an acceptable tolerance.

Fall velocity is also dependent upon particle shape. The aspect ratio of a particle can cause both the driving and resisting force in Figure 13-3 to diverge from their simple spherical derivation. All of the equations assume a shape factor or build one into their experimental curve. Only Report 12 is flexible enough to compute fall velocity as a function of shape factor. Therefore, shape factor is exposed as a user input variable but it will only be used if the Report 12 method is selected.

2.4.4.4.b) Effective Transporting Depth

The deposition limited works by comparing how far a particle can fall in a time step versus the distance available for it to travel. The distance it can fall is computed using the selected fall velocity method. But the travel distance available depends on the concentration profile of the grain class in the flow field (i.e. sediment is not uniformly distributed in the water column).

The classic concentration profile theory was developed by Rouse (1963) and is summarized in Figure 13-4. The Rouse number z is higher for larger particles and lower for higher shear velocities. Smaller particles and higher shears result in suspended particles distributed over more of the water column. This corresponds to a larger distance the average particle has to fall in order to be deposited.

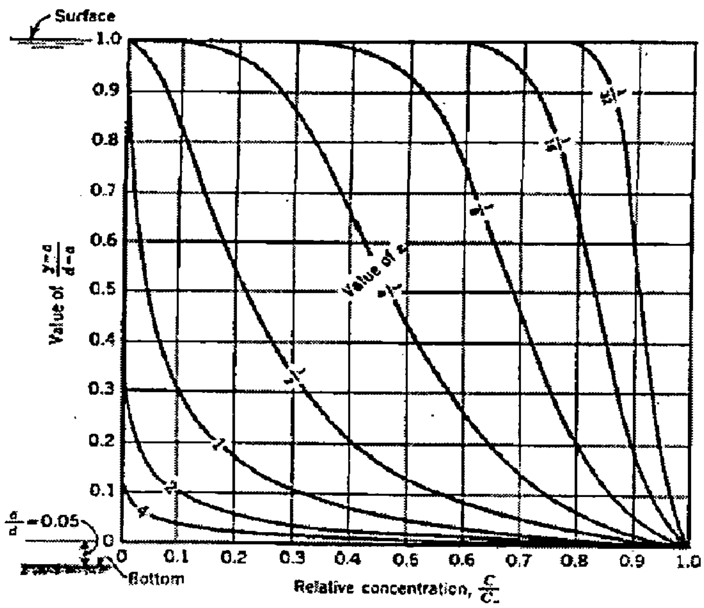


Figure :-2.8: Rouse concentration profiles

As mentioned above, Toffaleti broke the water column down into four zones and computed the transport separately for each (Figures 13-5). This can be used as discrete (if somewhat coarse) integration of the Rouse profile. HEC-RAS adopted these four zones as the effective transporting depth for different grain sizes. Grain classes including and smaller than very fine sand are evenly distributed throughout the water column. Fine sand is fully mixed over the middle, lower and bed zone which compose the lower ½.5th of the water column. All coarser particles are assumed to travel relatively close to the bed. Medium sand and coarser particles settle out of the lower zone and bed zone, a well mixed zone that is 1/11.24th of the water column thickness based on Toffaleti's regressions.

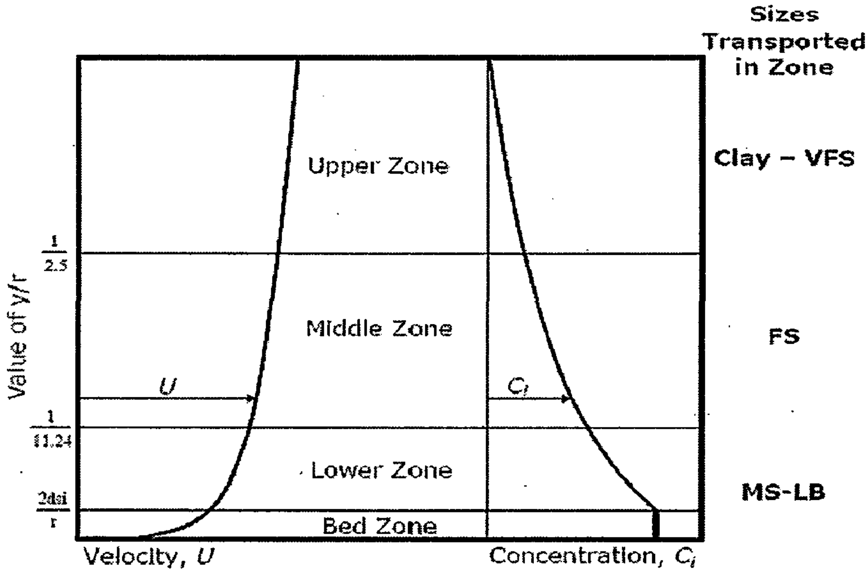


Figure :-2.9: Toffaleti's zone for computing transport (after Vononi, 1954)

The approach has limitations. Material is assumed to be evenly distributed through the zone at the beginning of each time step. This is a simplification of the concentration gradients as depicted in Figure :-9. The assumption also neglects the vertical flow distribution in a cross section. By tying the transporting depth only to grain size, the Rouse shear velocity dependence is lost. Finally, the transporting zone is fully mixed at the beginning of each time step so there is no memory of how far material settled in the previous time step. Despite the limitations, the temporal deposition limiter provides an advantage over a straight continuity approach by limiting the amount of sediment surplus that is deposited based on an approximation of a physical process.

2.4.4.2 Erosion Temoral Limiter

Similar to deposition, erosion is also a temporally dependent process. An unlimited amount of material cannot be eroded in a time step. Therefore, a temporal limiter needs to be applied to the computed continuity deficit. Unfortunately, the physical process that drive the temporal nature of erosion are not as well understood as those that limit deposition. The equation used are more empirical and generally less accurate.

The current theory implemented in HEC-RAS is based on the 'Characteristic Flow Length' principle. The governing assumption, based on undocumented flume experiments, is that a flow field requires thirty times the water depth to fully entrain a continuity deficit. The equation for entrainment coefficient is:

$$C_e = 1.368 - e^{-\left(\frac{L}{30D}\right)}$$

Where C_e is the entrainment coefficient, D is flow depth and L is length of control volume. The resulting entrainment coefficient for depth ratios between zero and forty are plotted in Figure 10. The computed sediment deficit is multiplied by this entrainment coefficient to calculate how much of it translate into erosion.

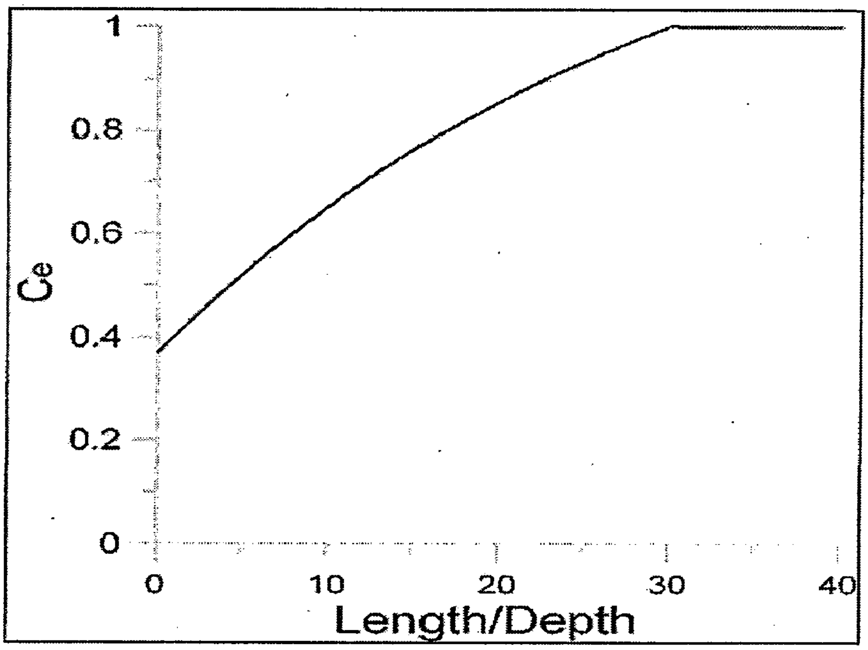


Figure :-2.10: The calculated entrainment coefficient for arrange of control volume length to depth ratios

If the length exceeds the flow depth by thirty times or more the entrainment coefficient goes to one and all of the deficit is eroded from cross section. In the lower limit, as the length approach the depth, the second term of the C_e

equation goes to 1 leaving a minimum entrainment coefficient of 0.368. therefore, the program will always allow at least 36.8% of the deficit to erode.

2.4.4.3 Sorting and Armoring

Erosion can also be supply limited. In many well graded rivers, the full bed gradation is covered by a layer of coarse material called armour layer. This layer can be formed by static armouring or the differential transport of finer materials. Particularly downstream of dam, most of flows mobilize fine particles, while the coarse material is static and collects on the surface shielding the deeper material from transport. The armour layer can also be formed by mobile to achieve equilibrium transport of a graded material(Parker,2008).

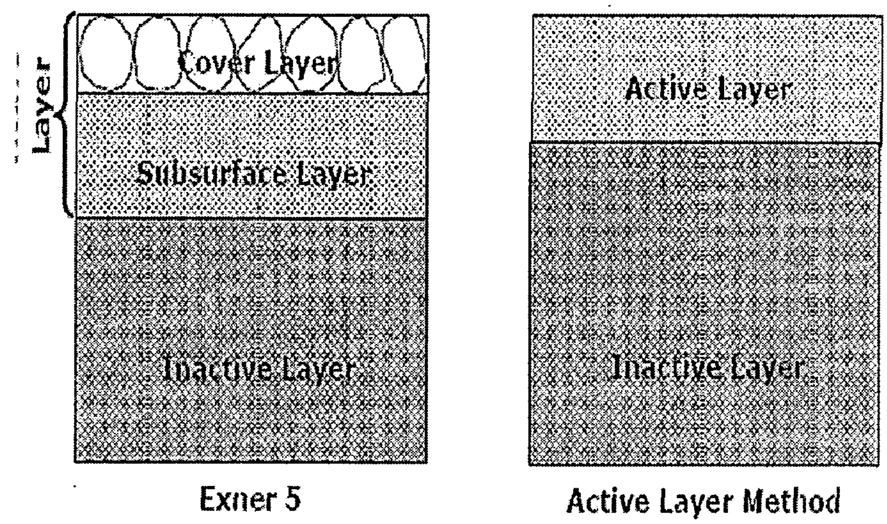


Figure :-2.11: Schematic of the mixing layers in HEC-RAS sorting and armoring methods

2.4.4.3.a) Exner 5

Exner 5, a three layerbed mixing algorithim (figure 11) was designed to account for influences of atatic armoring. This algorithim was developed by Tony Thomas (Thomas 1982) and is the default method in HEC-6 and HEC-6T. It subdivides the active layer into a cover layer and a subsurface layer. Deposition and erosion take place in the cover layer. It should be noted (once again) that the sediment capacity computation is based on the combined cover

...e layer(i.e the active layer). The concept ... Al Harrison's thesis at ... the photograph shown in figure 12 and ... where he studied under Einstein.

...yer coarsens (eg. Erosion of fines),the sediment capacity of the ...s will be reduced because the finer material will constitute a ...entage of the active layer. Additionally, if the stratification weight of the active layer, drops bellows 2d (twice the depth of one grain), ... and armoring rules are invocked to reduce the influence the cover ...d have on transport capacity. These rules test the thickness of the ...r and when it reaches 50% of 1d the cover layer is completely mixed ... subsurface layer. The value 50% comes from Al Harrison's thesis ...e found that equilibrium sediment transport was affected when 40% ... surface was covered in his flume experiments. The new cover lay... intantly (figure 12)

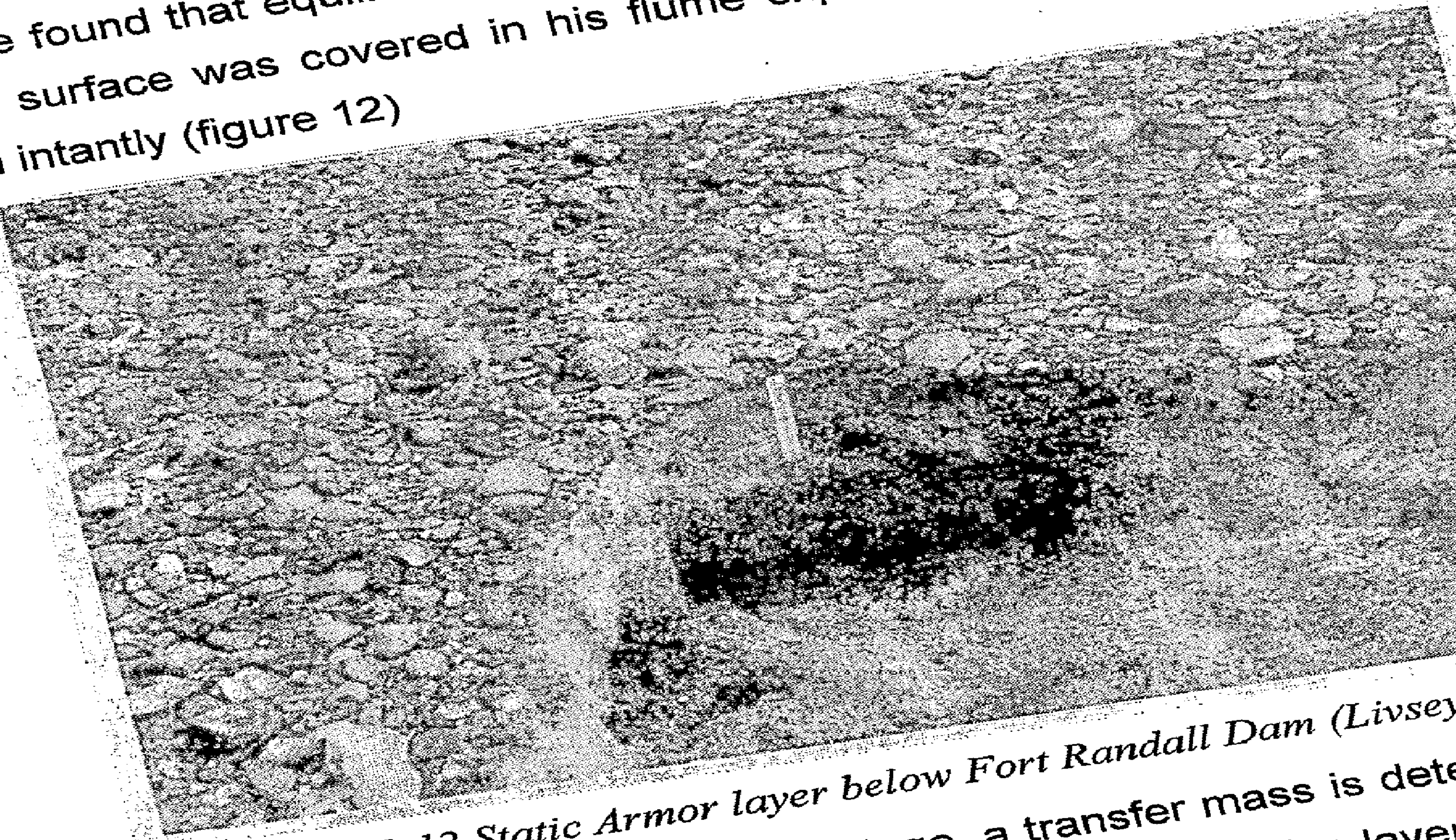


Figure :-2.12 Static Armor layer below Fort Randall Dam (Livsey...

At the beginning of the bed mixing stage, a transfer mass is dete... is the amount of material that will be taken from the inactive laye... sub layer. The initial transfer mass is determined by computing ... transport capacity and converting that rate, the inflowing sedi...

rate and the transport potential rate into sediment mass (for the given control volume). For each grain size, the incoming mass (of that grain size) is subtracted from the estimated transport capacity mass (for that grain size).the largest differential for any grain size is the potential transfer mass. The transfer mass starts with the potential transfer mass. Then it is subjected to other constraints, in arriving at the final value for a computational time step. The first test is the maximum scour mass. It is not allowed to exceed he maximum scour mass.

The maximum scour mass is the amount of material that is above the equilibrium depth. The maximum scour is usually the limiting factor in creating the subsurface layer. When this happens the final active layer in Exner 5 is approximately, the layer of material between the bed surface and a hypothetical depth at which no transport occurs for the given gradation of bed materials and flow conditions. However, there are a couple of additional constraints on the transfer mass. If the interactive layer is more than 10% clay and the clay transport option is turned on, the transfer weight is limited by he erosion rate of the clay material. As a amount of material equal to $2D_{100}$ (twice the largest grain size).

Stratification Weight: At the beginning of each computation time step, the stratification weight of the cover layer is computed. The weight of sediment for a depth of 0.5*one grain diameter, then the cover layer is no longer an effective shield against leaching of finer particles from the subsurface layer. The subsurface layer is combined into the cover layer and a new subsurface layer is formed from the interactive layer based on the previous computed transfer mass.

The stratification weight is the sum of the grain depth of each grain size. For instance, assume the cover layer is composed of only two sizes, coarse and fine sand. If the amount of material of the coarser sand was able to fill the cover layer to a depth of 1.5 times the diameter of fine sand was able to fill the cover layer to a depth of 0.3 times the diameter of fine sand, the overall depth (in terms stratification weight of cover layer is less than 1.0 grain, then the cover layer is no longer an effective shield against leaching of finer particles from the subsurface layer. Also as previously noted, if the stratification weight of the entire active layer is below 2.0, then there is an additional reduction in the amount of sediment host can be eroded.

Equilibrium Depth: Equilibrium depth is defined as the smallest depth at which all particle sizes in the bed surface mixture will resist erosion for the given Hydraullic forces imposed on the bed. Alternatively, it is the maximum potential scour depth.

Manning's Equation

$$V = \frac{1.49}{n} R^{\frac{2}{3}} S_f^{\frac{1}{2}}$$

Strickler's Roughness Equation

$$n = \frac{d^{\frac{1}{6}}}{29.3}$$

Einstein's Transport Intensity Equation

$$\Psi = \frac{\rho_s - \rho_w}{\rho_w} \cdot \frac{d}{D \cdot S_f}$$

Where:

V	=	Velocity
R	=	Hydraulic Radius
S_f	=	Friction Slope
n	=	Manning's n value
d	=	representative particle size
ρ_s	=	grain density
ρ_w	=	water density
D	=	Depth

Particle erosion, in the Einstein Equation , is assumed for $\Psi \geq 30$. the sediment particles are treated as quartz sand, for which the specific gravity is 2.65. The value of the submerged particle density term in the equation $(\rho_s - \rho_w/\rho_w)$ is 1.65. substitution allows Einstein's Transport Intensity equation to be reduced to:

$$S_f = \frac{d}{18.18D}$$

These three equations can be solved for unit water discharge by replacing the sub- sectional hydraulic radius in the manning equation with the panel depth, D, and the n-value with Strickler's equation.

$$q = \frac{1.49}{\left(\frac{d^{\frac{1}{6}}}{29.3}\right)} \cdot D^{\frac{5}{3}} \cdot \left(\frac{d}{18.18D}\right)^{\frac{1}{2}}$$

Or

$$q = 10.21 \cdot D^{\frac{7}{6}} \cdot d^{\frac{1}{3}}$$

Where: q = water discharge in cfs per ft of width

If all sediment particles in the bed were the same size, the equilibrium depth would be

$$D_e = \left(\frac{q}{10.21 \cdot d_i^{\frac{1}{3}}}\right)^{\frac{6}{7}}$$

Where: D_e = Equillibruim depth for particle size

Active Layer

A two layer active layer method (figure 11) is also included in HEC-RAS. A simple active layer approach has obvious disadvantages including less vertical discretization and no explicit armoring factor. It should be used with caution. However, it is more intuitive and transparent method, it can form a coarse of fine active layer or fine active layer and with an appropriate exchange increment, it may be preferable in some cases for modeling mobile armor systems (Gibsons and Piper, 2007).

Hirano (1971) is often credited with introducing the "active layer" approach for sediment transport modeling (through similar work was also going on at HEC at the same time). The approach divides the substrate into an active (mixing or surface) layer that is available for transport and an inactive layer that has no influence on the computation for a given step.

Since the active layers are composed of different gradations, there is a gradational discontinuity between them. As the bed aggrades and degrades material is passed across this interface in order to reset the active layer to the specified thickness (e.g. the d_{90}). In the erosive case, computing the gradational composition of this exchange increment is trivial. Material from the inactive layer is brought up into active layer.

The depositional case could be a simple mater of assuming that the material added to the active layer is fully mixed. Resetting the active layer thickness would involve transferring some of this mixed active layer material to the inactive layer. Alternately in the fully unmixed scenario, bed load material would be deposited on the top of the active layer, and unmixed material from the bottom of the active layer would be moved to the inactive layer (the active layer would the fuly mixed before the next computational time step). However , after field observation of gravel bed streams suggested that the surface layer is systematically coarser than the substrate, Parker (1991) tested a different hypothesis that the depositional exchange increment is composed of the bedload gradation rather than the initial active layer gradation. It was hypothesized that the deposited material penetrated the active layer and was essentially deposited directly into the active layer. This approach was limited because it disallowed bed evolution or downstream fining, but led to the hypothesis that the surface layer acts as a bias filter giving finer deposited bed load grains a higher chance of passing directly into the inactive layer.

Tiro-Escobar et al (1996) advanced the idea that the depositional exchange increment was a combination of the active layer gradation and the bed load gradation. They generate an approximate weighting function from their tests (without claiming generality):

$$f(i,j) = 0.7p(j)+0.3F(j)$$

Where: f(I,j),p(j) and F(j) represent the fraction of the exchange increment, bed load and active layer respectively, associated with grain class(i). this is default assumption used in HEC-RAS. During deposition, when using the active layer method, the exchange increment is composed of 30% of the composition of the active layer from the beginning of time step and 70% of the gradation of the deposited material. For example, if 10tons of material were deposited for a given time step (assuming the active layer remained the same thickness): 3 tons from the active layer would be transfer to the inactive layer, 7 tons of the deposited material would be added to inactive layer. The 3 remaining tons of the deposited material would then be mixed into the active layer.

2.4.5 Cohesive Transport

Most of the sediment transport equations were generated from data for particles sand sized or larger. Only Laursen (1968) included data from silt, and even then, only coarse silt was used. Therefore, most silt and all clay particles are outside of the range of applicability of the sediment transport function implemented in HEC-RAS. Transport of this fine particles, particularly clay, is further complicated by electrostatic and electrochemical forces that can cause particles to flocculate and "stick" to he bed surface. This makes deposition and erosion of fine particles fundamentally different than the cohesion less transport of sand and gravel.

Another difference is that silt and clay are often treated as wash load. Wash load is material that remains in suspension, since the vertical velocity component of the turbulent eddies exceeds the small settling velocity of the particle (Bagnold, 1966; Van Rijn, 1984). For many system, the assumption of fine particles staying in the wash load is reasonable, and an approach that simply passing them through the system is often sufficient. The assumption will not, however work for the system that have reservoirs or other areas of very low velocity. Further more, even when the wash load assumption holds, there still may be the issue of the erosion of fine particles within the model area.

For instance, even when the standard transport equations would show fine particles being entrained, the actual erosion rate, especially for clay, is usually much lower. When the concentration of clay in the bed material is high enough, it can even reduce the rate at which sand and gravel is eroded.

There are two methods available in HEC-RAS for silt and clay sized particles: using standard transport equations or implementing the Krone and Partheniades approach.

2.4.5.a) Standard Transport Equation

The default option for silt and clay simply uses whatever transport function was selected, for the other grain classes, for the fine material as well. This will result in extrapolation outside of he derived range of the transport equation and, and usually produces enormous transport potentials. These transport potentials should not be considered even remotely representative. They can be useful, however, for system where fines are not being added or removed from the bed in any appreciable amounts. Because of the huge transport potential, even a tiny amount of silt an clay in the active layer will generate a very large sediment transport capacity. This means the system will have essentially an unlimited ability to pas all the small particles to he system, leaving only a minute fraction in the acive layer. This method can be used to route fine wash load through the system, if the study objectives do not involve the erosion or depositon of the fine material.

2.4.5.b) Krone and Parthenaides

If the behavior of cohesive erosion ad deposition is of interest, however, the standard transport equation that compare capacity to supply are not sufficient. Cohesive particles are small enough that their behavior is usually dominated by surface forces rather than gravity. A fundamental concept of Krone deposition being the probability that a floc will "stick" to the bed (as opposed to sand and gravel that "sink" to the bed). Similarly in Pathenaides erosion, the issue is whether the bed shear is sufficient to overcome the electrochemical forces holding the grains together (rather than determining whether the bed shear is adequate to physically lift a grain particle of a given volume and weight to the bed). Krone and Pathenaides are simple functions that are used in HEC-RAS to quantify the deposition and erosion of cohesive material.

These equation are part of a general framework in which a single process controls cohesive sedimentation in each of three hydrodynamics states: Deposition, Particle erosion, and Mass erosion. These zones are delineated by two threshold shear stresses input by the user:

τ_c: Critical shear threshold for particle erosion

τ_m: Critical shear threshold for mass erosion

such that $\tau_c \leq \tau_m$. The calculated bed shear stress (τ_b) for each cross Section is compared to the two thresholds and the appropriate zone identified. Computation then proceed based on the given zone (figure 13)

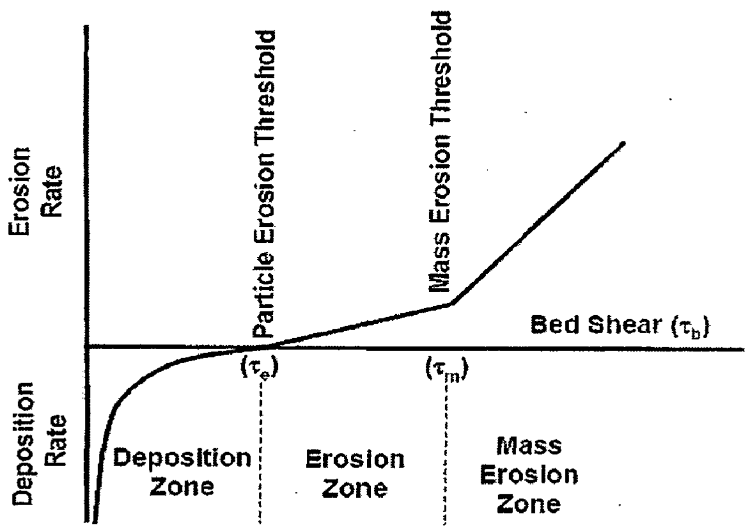


Figure :-2.13: Schematic of cohesive sedimentation zones and processes as a function of shear

In the past, a fourth zone was hypothesized. The equilibrium zone, at shear below ζ_c and greater than a deposition threshold ζ_d was assumed to be a state where the binding forces exceeded the erosion forces, but turbulence was sufficient to keep transport particles in suspension. In this approach, no bed change would occur for bed concept to fallout of favor (Sanford and Halka, 1993). Therefore, a single erosion threshold, above which particles erode and below which they deposit, is used in HEC-RAS.

2.4.5.c) Deposition

Deposition in HEC-RAS is based on the work of Krone (1962). Krone's primary contribution was the observation that suspended sediment decreased logarithmically, in his experiments, for concentrations less than 300mg/l. he therefore, quantified the rate of deposition as:

$$\left(\frac{dC}{dt}\right)_d = -\left(1-\frac{\tau_b}{\tau_c}\right)\frac{V_s C}{y}$$

where: C = sediment concentration

t = time

τ_b = bed shear stress

τ_c = critical shear stress for deposition

Vs = fall velocity

y = water depth (Effective Depth in HEC-6)

By separating variables and integrating, the following relationship emerges:

$$\int \frac{dC}{C} = \int -\left(1-\frac{\tau_b}{\tau_c}\right)\frac{V_s}{y}dt \rightarrow$$

$$\ln\left(\frac{C}{C_o}\right) = -\left(1-\frac{\tau_b}{\tau_c}\right)\frac{V_s t}{y} \rightarrow$$

$$C = C_o e^{\left(-\left(1-\frac{\tau_b}{\tau_c}\right)\frac{V_s t}{y}\right)}$$

With the logarithmic assumption, this is a theoretical equation that does not require empirical coefficients. The erosion shear threshold is the only user input parameter that governs this behavior. (Although it should be noted that there are multiple option for computing bed shear stress and fall velocity.)

If the calculated bed shear (τ_b) is less than the critical erosion shear (τ_c –a user input parameters) deposition will occur. The ratio of these shear stresses, subtracted from one, is referred to as the probability factor which represent the likelihood of a floc sticking to the bed. It approaches one (100% probability of deposition) as the bed shear (and, therefore the ratio of the shears) decreases, and it approaches zero as the bed shear approaches the critical shear of deposition (0% probability of depositon). The equation is not applicable for shear stresses greater than the depositional threshold.

Krone (1962) further posited that the deposition rate is dependent on flocculation rate. The flocculation rate, in turns, is a function of the concentration of the sediment and chemical composition of the flocculation-deposition modeling since Krone's initial work. However, HEC-RAS does not attempt to compute flocculation. The grain size distribution, therefore, should reflect the distribution of flocculants rather than discrete grains.

2.4.4.3.d) Erosion

Erosion is more difficult and far more empirical than deposition. HEC-RAS follows the approach of the work of Parthenaides (1962). He posited that the force resisting erosion is mainly electrostatic in nature, since the average electrochemical force exerted on a clay particle is a million times greater than the average weight of the particle. He further concluded that erosion rates could be approximated by a pair of linear functions of bed shear. When the critical shear of the cohesive material is exceeded, particle erosion begins as individual particle or flocs are removed, one at a time, at a rate that is approximately a linear function of shear. When the (even higher) mass erosion shear is exceeded, the bed start to erode in multi-particle chunks or clods. This process, referred to as mass erosion or mass mass wasting, occurs at ahigher rate than particle erosion, and it can also be approximated with a linear function of the bed shear.(figure:-14)

Particle Erosion ($\tau_e < \tau < \tau_m$)

According to the Parthenaides equation (1965):

$$\left(\frac{dm}{dt}\right)_e = M\left(\frac{\tau_b}{\tau_c} - 1\right)$$

where: m = mass of material in the water column

t = time

τ_b = bed shear stress

τ_c = critical shear stress for erosion

M = empirical erosion rate for particle scour

$$\int dm = \int M\left(\frac{\tau_b}{\tau_c} - 1\right)dt \rightarrow m = M\left(\frac{\tau_b}{\tau_c} - 1\right)t + m_o$$

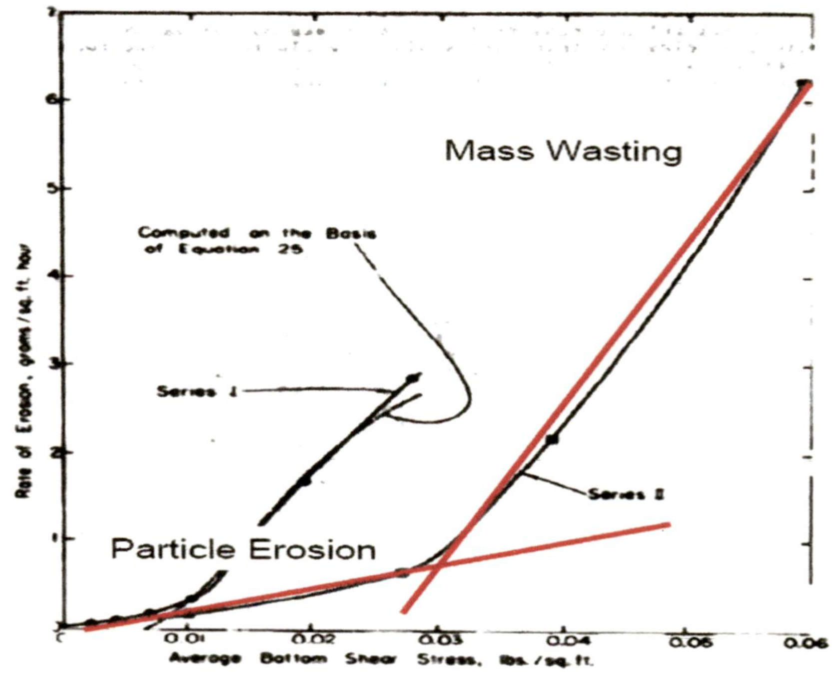


Figure :-2.14: Shear stress-rate of erosion relationship from Partheniades

This is an essential linear interpolation of mass erosion between the lower and upper end of the particle erosion zone (where the erosion rate is M at he shear threshold for mass erosion and 0 at the lower end of the range).

Mass Erosion ($\tau_c < \tau < \tau_m$)

Beyond the threshold of mass erosion, erosion rates are linearly extrapolated from the rate specified at the threshold based on a similar linear relationship as employed in the particle erosion zone (through with a larger slope and corresponding larger M). Therefore, a similar equation will be used to extrapolate linearly from M_t.

2.4.4.3.e) Estimating Cohesive Thresholds and Rates

The key to success for the Partheniades method is estimating the process thresholds and the erosion rates. These parameters are strongly site specific and even vary significantly with location and depth at a given site. Therefore, the variables can either be developed computationally, by calibrating them to some other measured parameter, or experimentally with a SEDFLUME apparatus.

There is limited published data on the erosion threshold and rate for cohesive materials. Chow (1959) included some basic data from the USSR permissible velocity data base (figure:-15). This data is a function of void ratio and clay plasticity. It only provides one of the four parameters required and should be used, very cautiously, only as a starting point for a calibration.

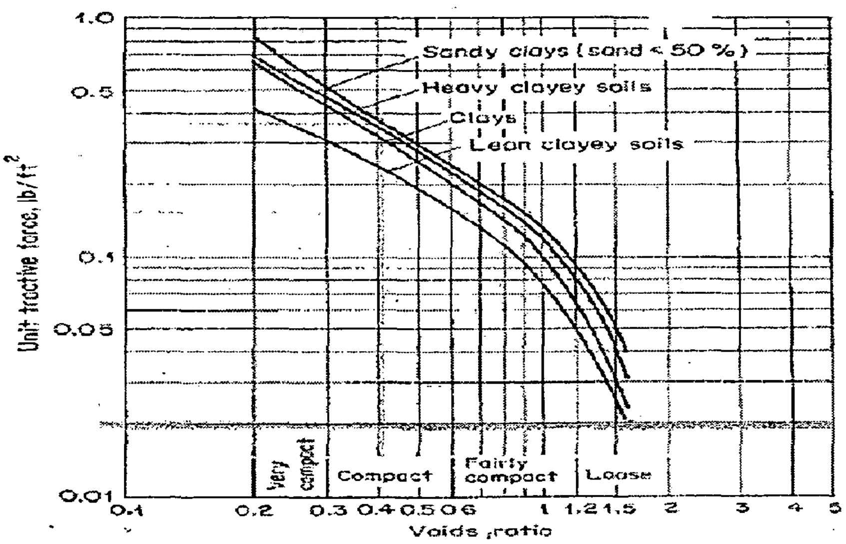


Figure :2.15: Permissible unit tractive forces for canals in cohesive material as converted from USSR data on permissible velocities (Chow, 1959)

In the absence of robust calibration data, some experimental data is usually necessary to get good results with the Parthenaides method. The most common apparatus used to measure the cohesive parameters is the SEDFLUME. This device pushes a core of the cohesive bed material through the bottom of the flume. For several different shears (velocities), the rate at which the core is introduced into the flow field is adjusted to match the rate at which it is eroded. The Corp's sediment lab in ERDC, and several different universities, can perform these experiments. ERDC's lab has the advantage of being able to travel to the project site. This avoid the disturbance of the core that is caused by shipping the material (the sample can be frozen prior to shipment, but the freeze/thaw cycle is itself disruptive).

2.4.5 Bed Change

Once the surplus or deficit is determined for the physical processes, a final deposition or erosion mass is computed. This mass must then be added or subtracted from the control volume by changing the cross section station/elevation points.

The mass is converted into volume and this change in volume is effectively spread over an upstream and downstream "wedge" (assuming an internal cross section) which allows the height of the wedge to be computed (so that it gives the correct volume). An exaggerated bed change is shown at river station 2 in figure:- 16

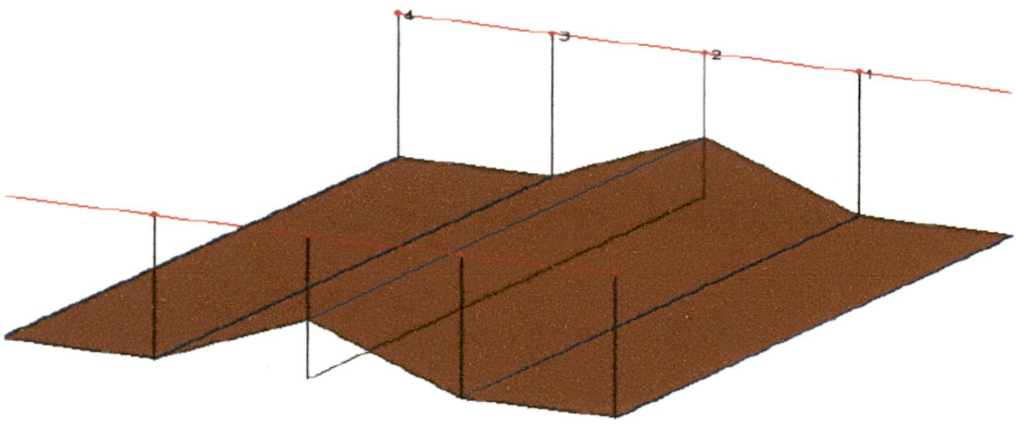


Figure :-2.16: "Wedge" used to distribute erosion or deposition volume longitudinally over the control volume

2.4.6 Deposition

Currently the only method available for translating erosion or deposition into changes in the cross section shape is to deposit or erode each wetted, movable cross section station/elevation point equally. Following these guide lines, an example of across section update for erosion or depositional cases is included in figure :-17. The points that move are both within the erodible bed limits and beneath the water surface elevation. For the erosion case, a duplicate point is generated if the mobile bed limit is wet.

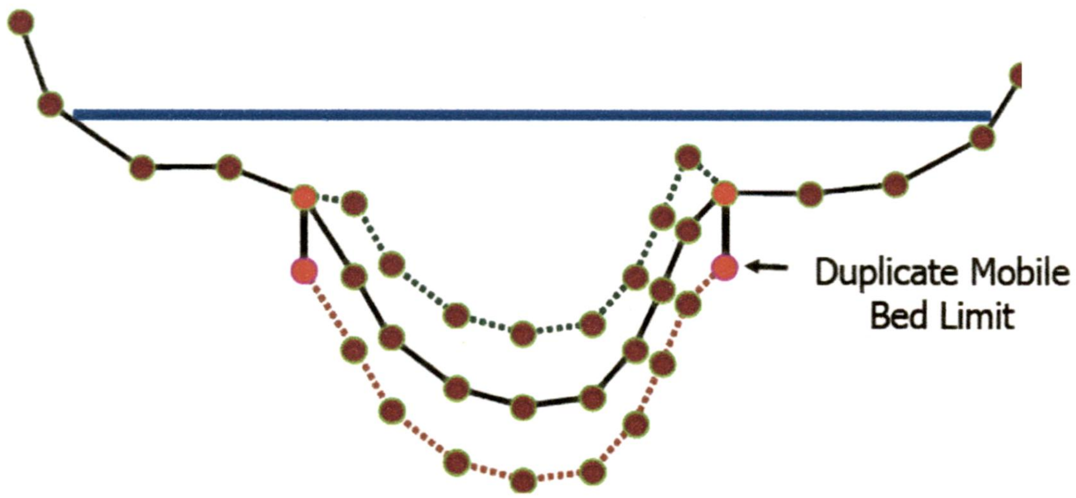


Figure :-2.17: Example of standard bed change rules used to update cross

There are a couple of exception to these basic rules however. First there is an alternate method that can be used by selecting the Allow Deposition outside of the movable bed limits entry under option →Bed change options menu in the sediment data editor. This option handles erosion in precisely the same way as the default method, confining erosion to the movable bed limits. For the depositional case, however, bed change is distributed equals between the erodible bed limits or not. The principle behind his method is that eroding velocity or shear are limited to the channel, but deposition can occur in the flood plain where slowly moving water allows material to settle out (figure:18)

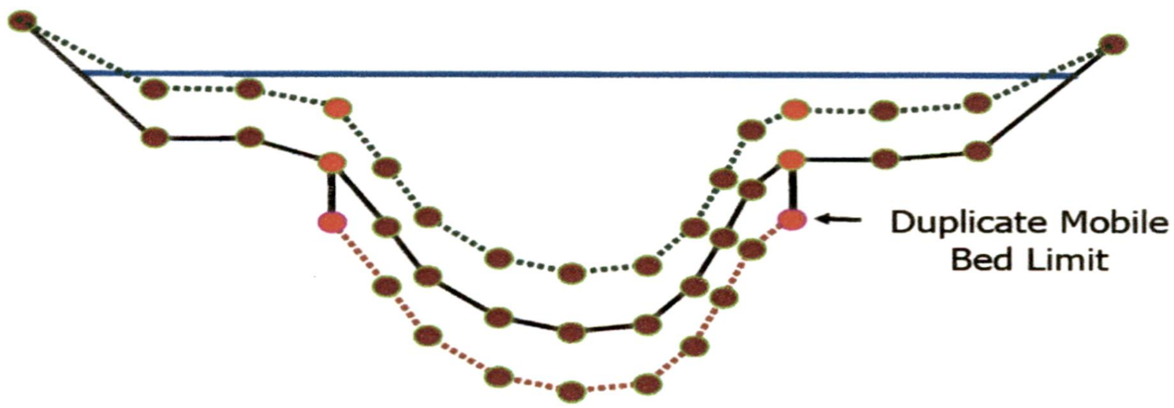


Figure :-2.18: Alternate bed change method that confines erosion to the Erodable limits but allows deposition at any wetted node

Finally, it should be noted that erosion will not allowed at any node included in an ineffective flow area regardless of which method is selected or where erodible bed limit are placed. Water velocity in an ineffective flow area is, by definition, zero. Therefore scour cannot occur at the cross section points in an ineffective flow area. However depositional bed change computed for points in an ineffective flow area is allowed.

Chapter 3

MODEL REACH AND DATA

3.1 Survey and Hydraulic Data

One dimensional mathematical model HEC-RAS-4 version is capable of handling sediment transport on mobile bed in network of river channels was used to simulate flows in Kosi river from barrage to about 47km downstream. It may be mentioned that the model being one dimensional the values of various parameter such as water level, bed level and discharge / velocity are arranged over the cross sections. For calibration/validation all water level studies are carried out assuming rigid bed during short duration flood event. Long term river bed changes may take many years and long term simulations are necessary for predicting those changes. The Topographical data in the form of river cross section is required to simulate topography. The water level and discharge data at boundaries and two or three locations within model reach is required for supplying data for boundary condition and model calibration. Similarly for long term river bed changes required topographical data to stimulate river topography. The water level, discharge data and sediment data required as upstream and downstream condition.

The required river cross sectional data was collected by M/S Wapcos India Ltd. in May 2002. The cross sections of the river within the embankment were at a regular interval of 1.0 km (Figure 3.1) and Figure 3.2 of the kosi river between Kosi barrage and 47 km. downstream of river. Similarly in May 2002 two gauge site namely at Dagmara 22 km. downstream and at Bhaptiahi 32 km. downstream of barrage was installed. The water level at this gauge sites were recorded every six hours during the period from June to December 2002. The work of establishing the gauge site and recording the gauge data was carried out through M/s Wapcos, New Delhi. Data regarding water levels, sediment and discharges, recorded at the Kosi barrage was supplied by Project Authorities, Kosi Barrage Project, Birpur, Bihar. The highest discharge, recorded at Kosi Barrage during this period was of the order of 10960cum/s. (387000 cusecs) The detailed plan of the leveed/ embankment reach of the Kosi river is shown in figure 3.1 and the distance between leveed/ embankment are shown in Table 3.1.

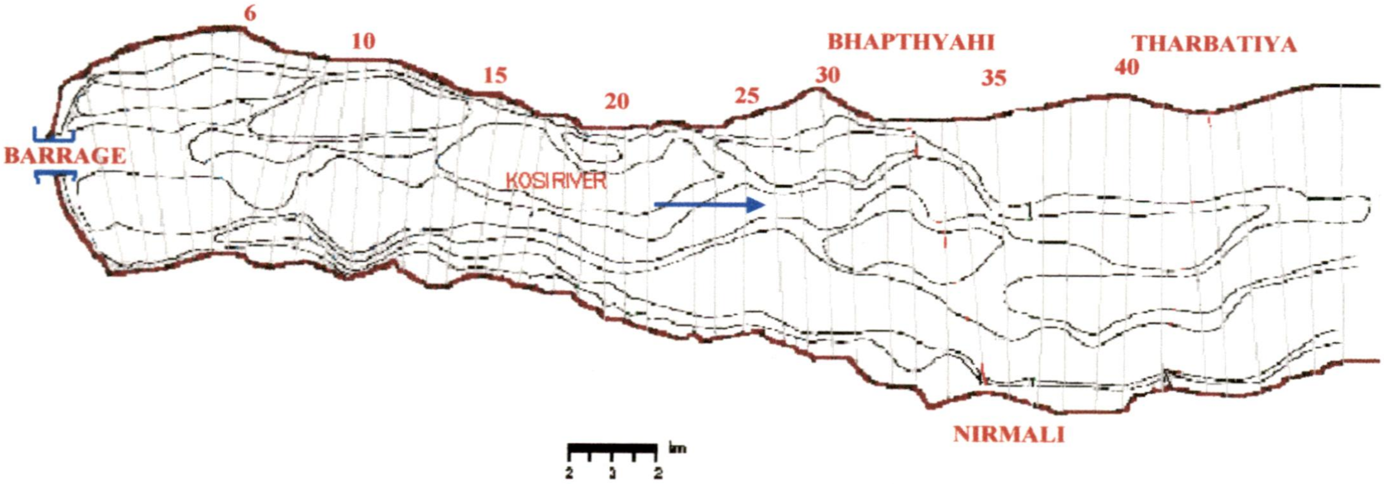
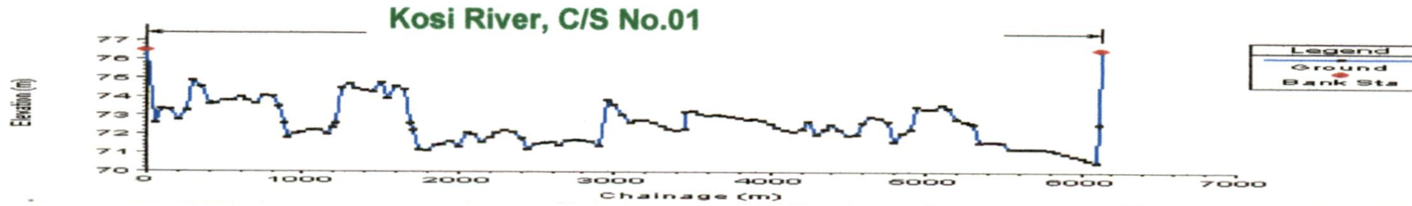
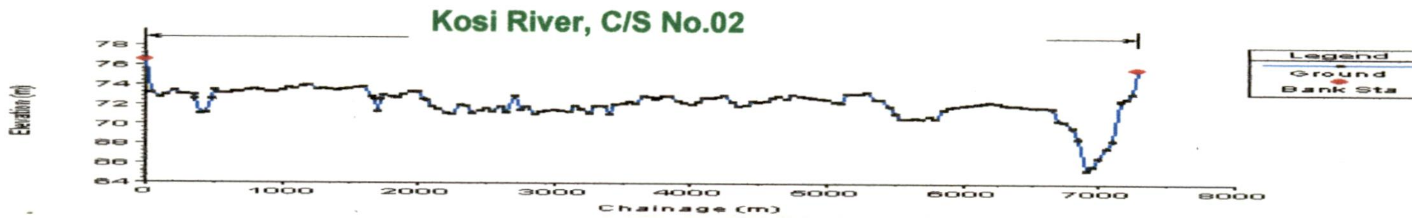
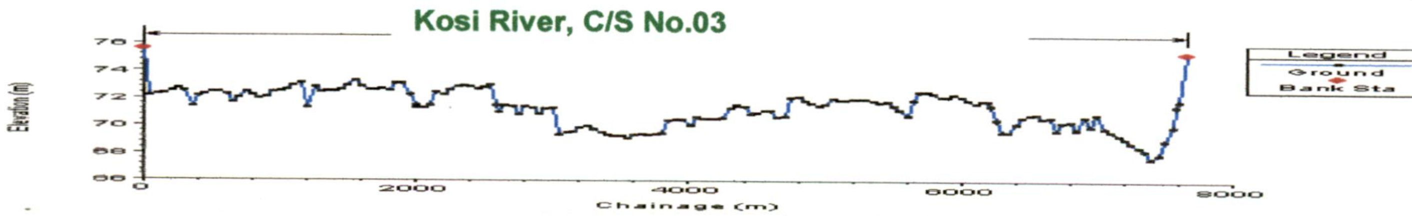
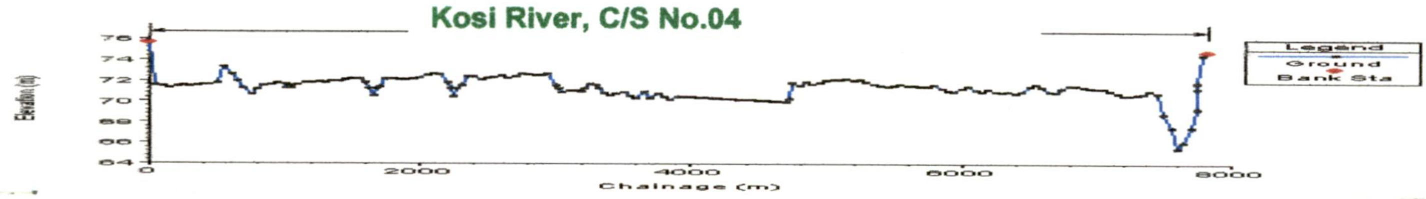


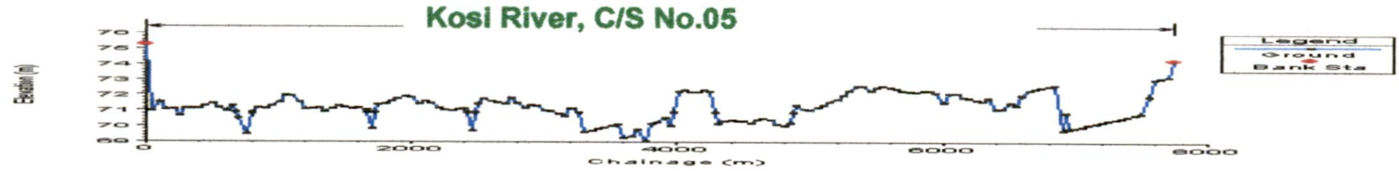
Figure:-3.1: Plan of Kosi river

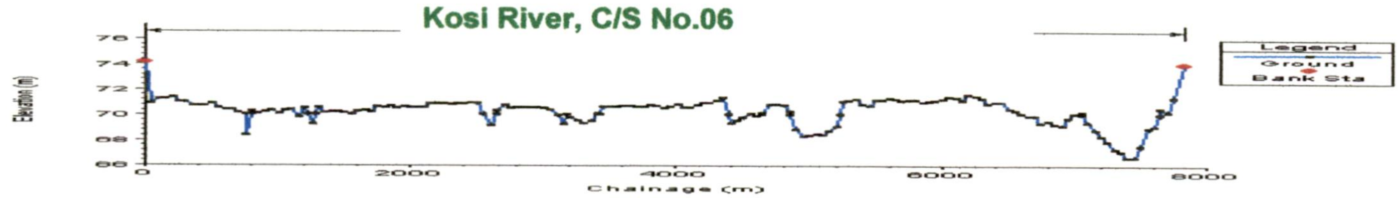


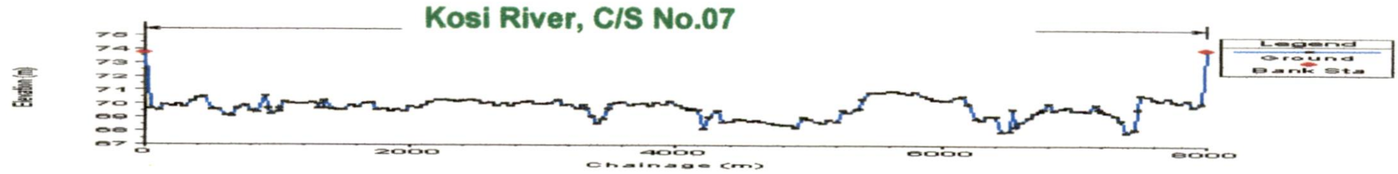


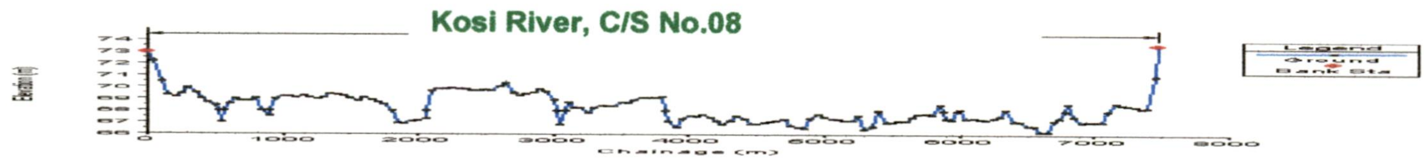


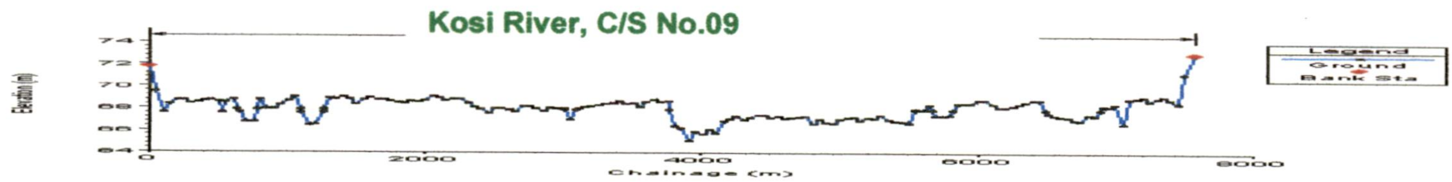

Kosi River, C/S No.04
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

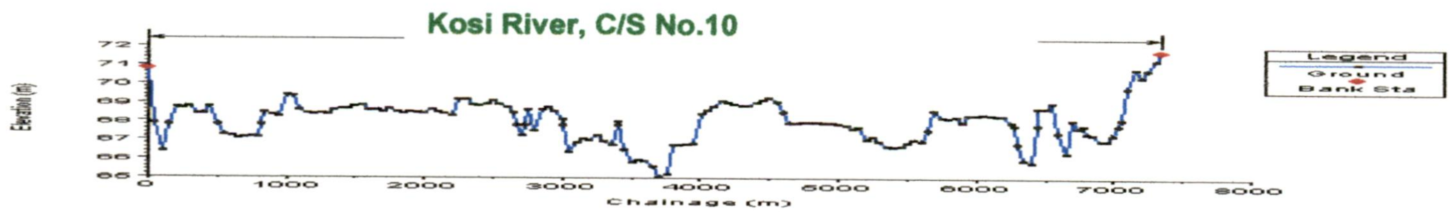

Kosi River, C/S No.05
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

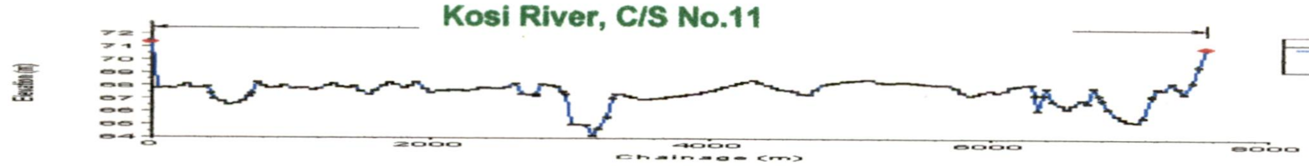

Kosi River, C/S No.06
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

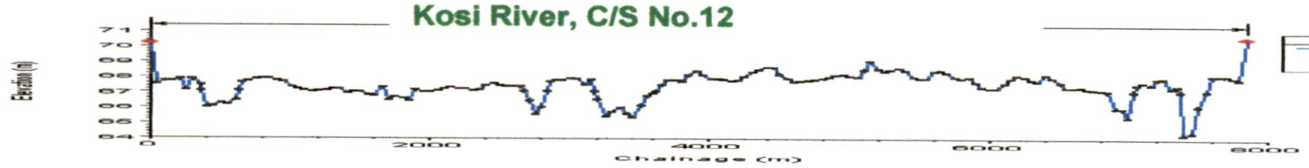

Kosi River, C/S No.07
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

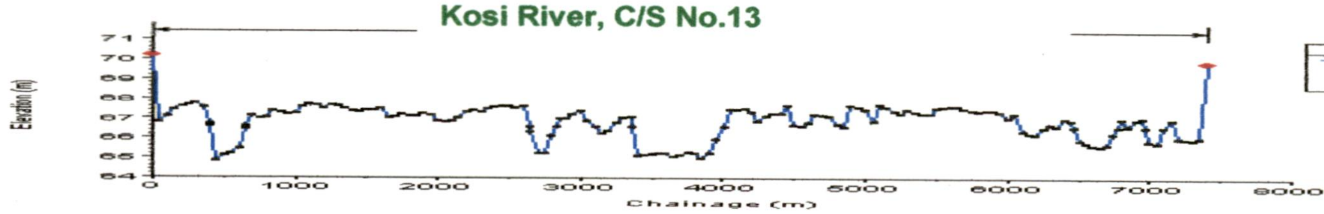

Kosi River, C/S No.08
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

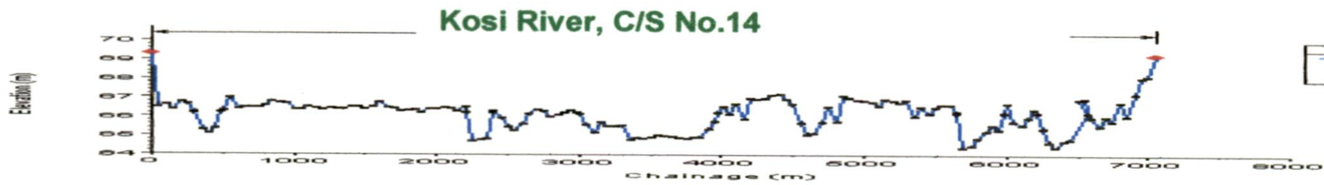

Kosi River, C/S No.09
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

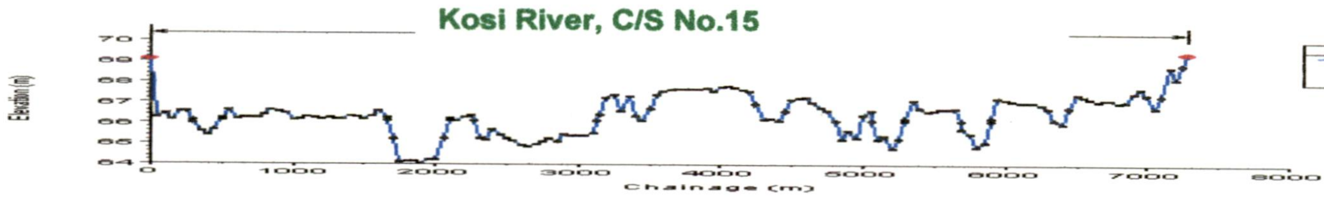

Kosi River, C/S No.10
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

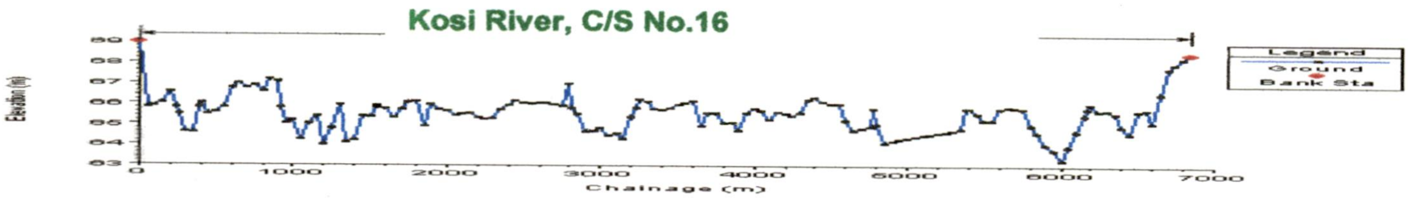

Kosi River, C/S No.11
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

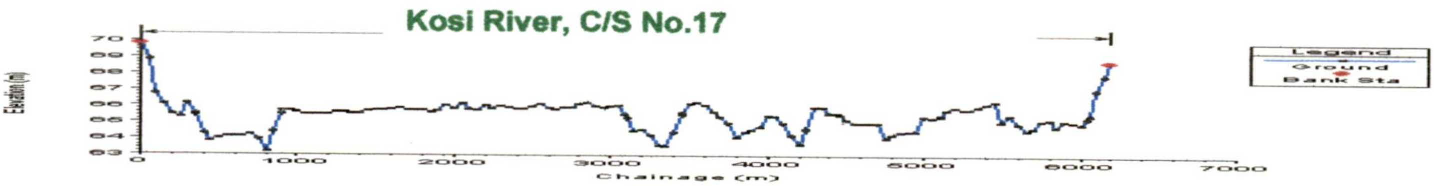

Kosi River, C/S No.12
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta


Kosi River, C/S No.13
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

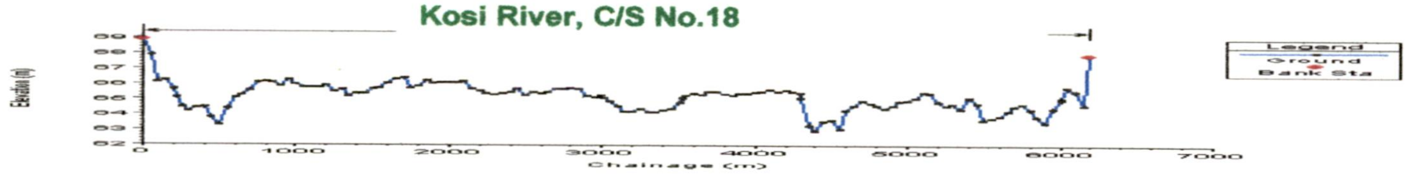

Kosi River, C/S No.14
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta


Kosi River, C/S No.15
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta


Kosi River, C/S No.16
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta


Kosi River, C/S No.17
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

Kosi River, C/S No.18



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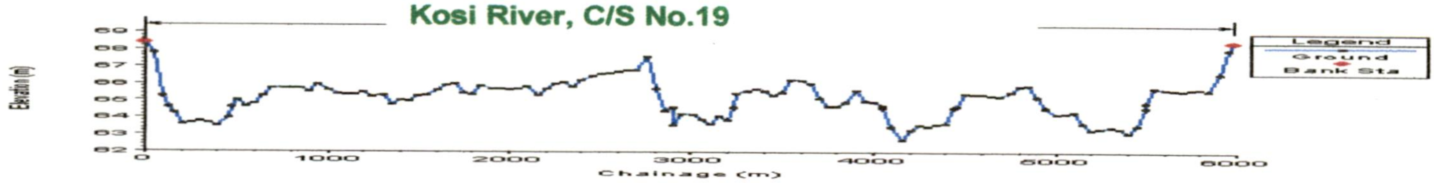

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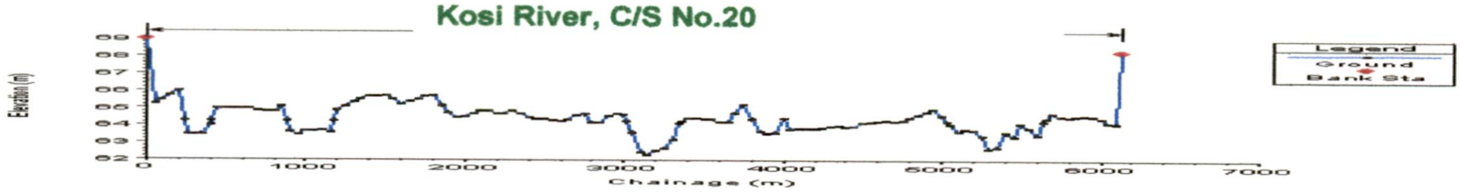

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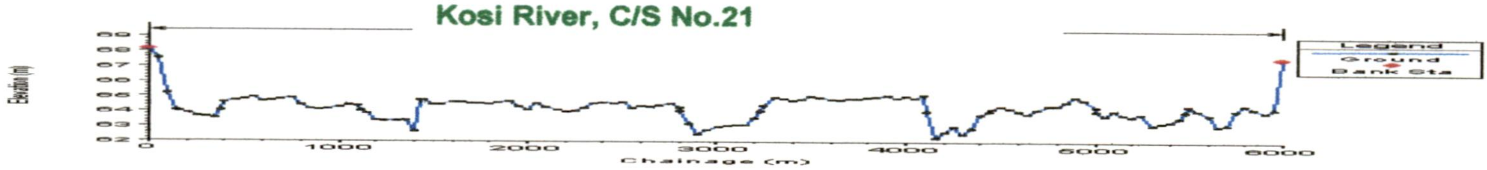

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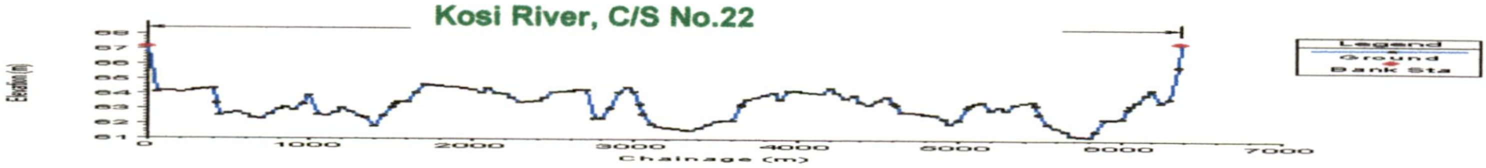

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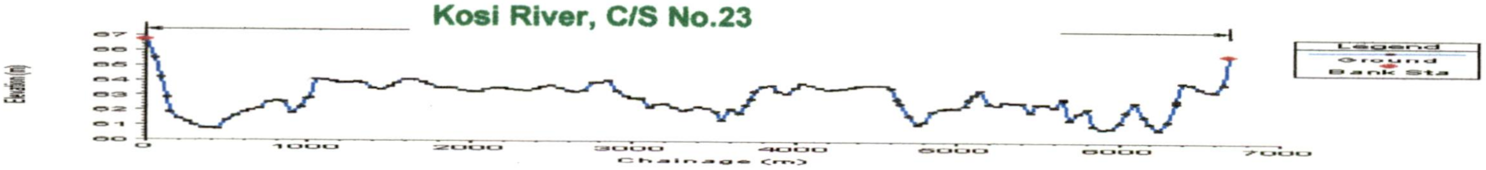

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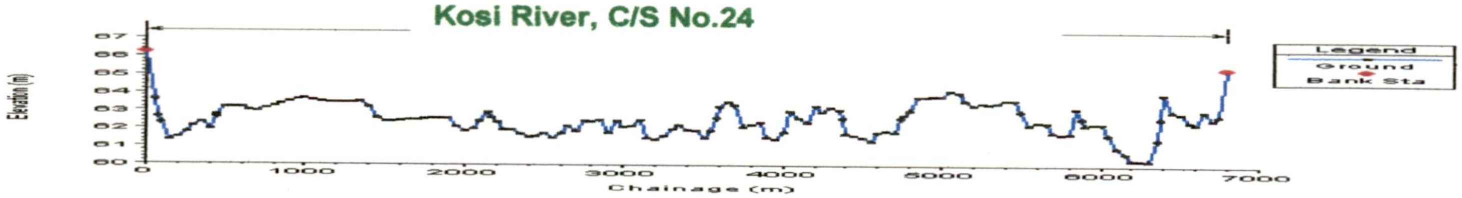

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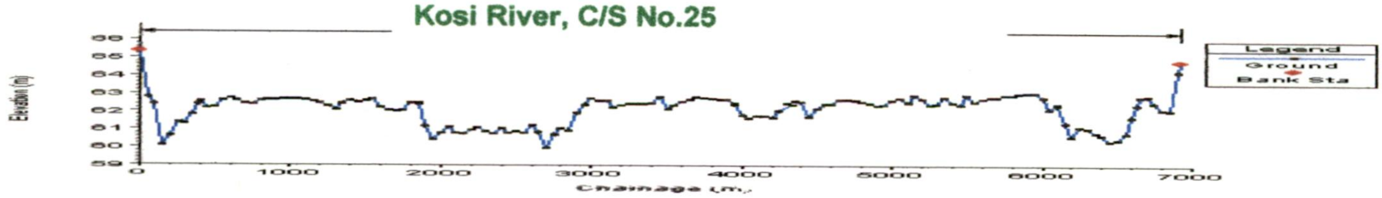

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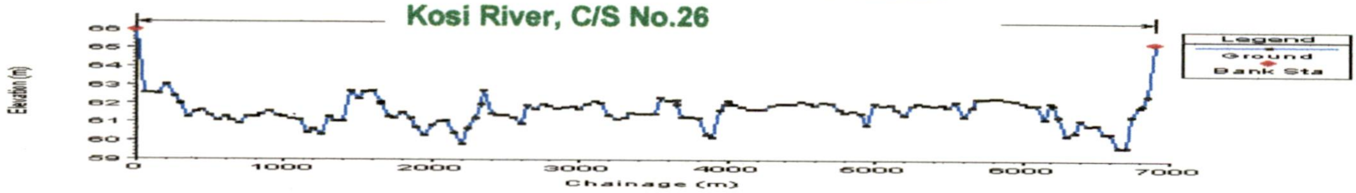

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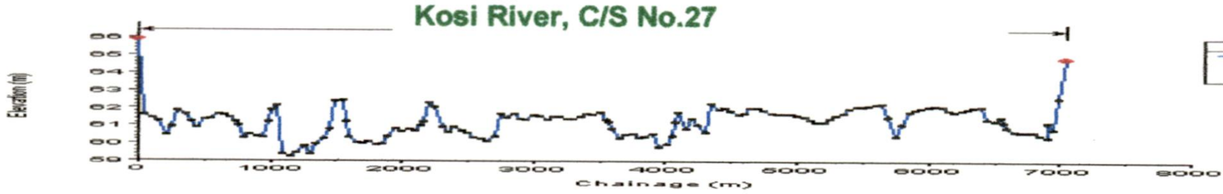

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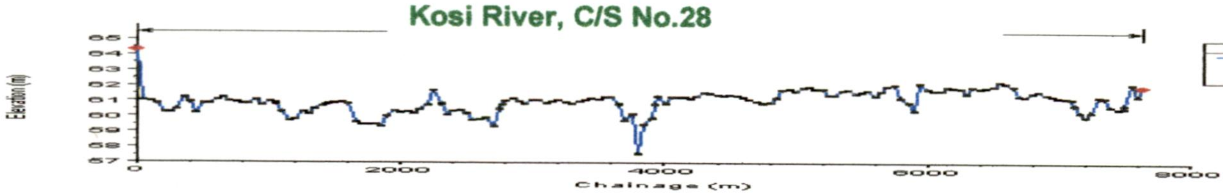

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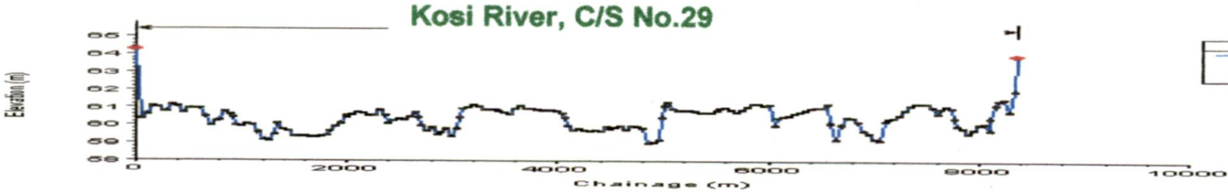

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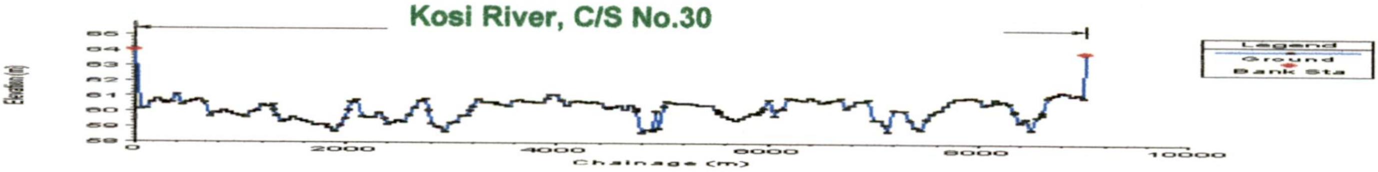

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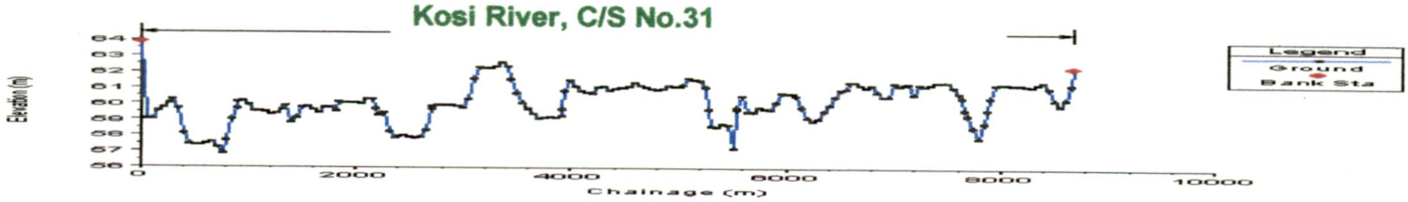

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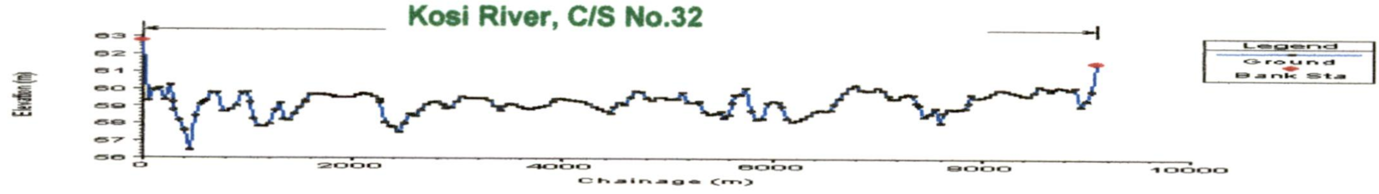

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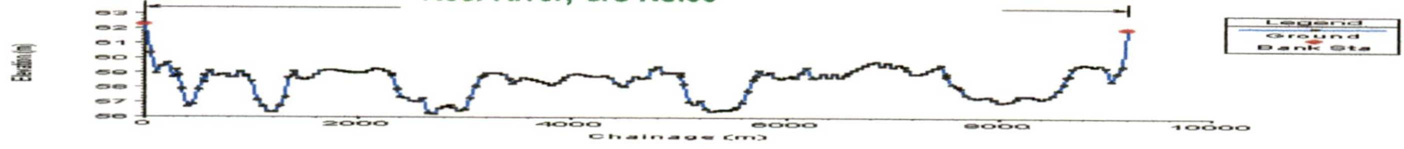

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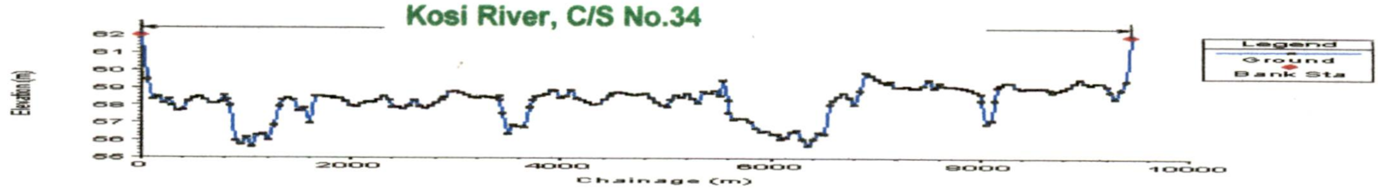

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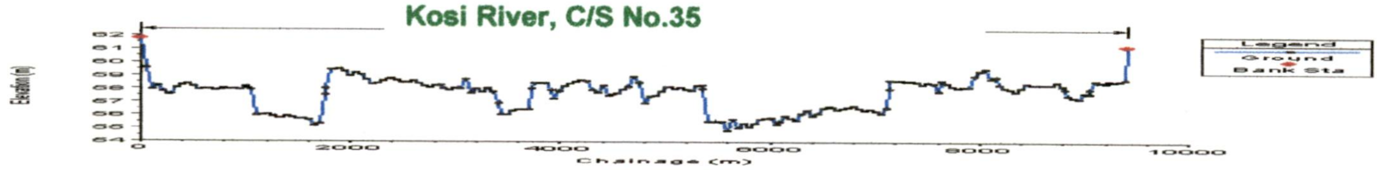

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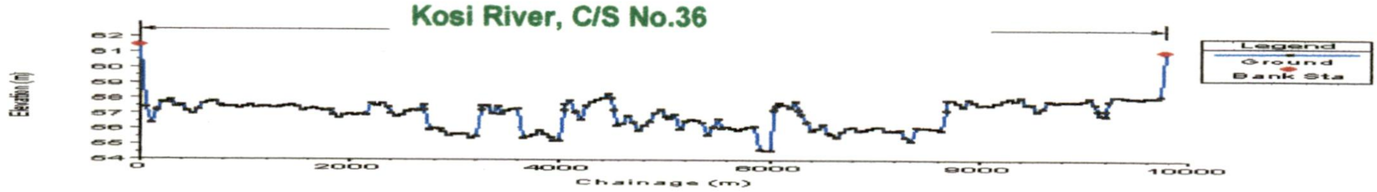

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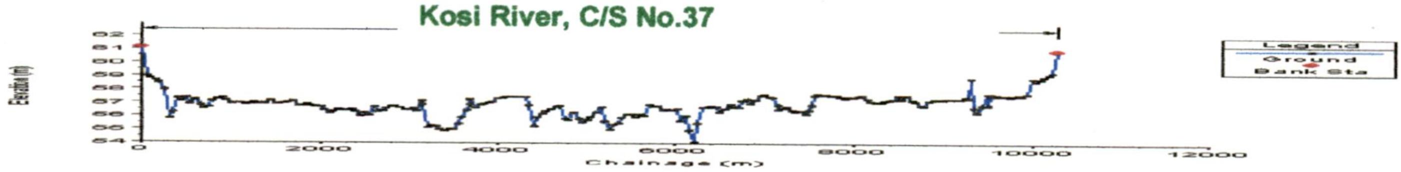

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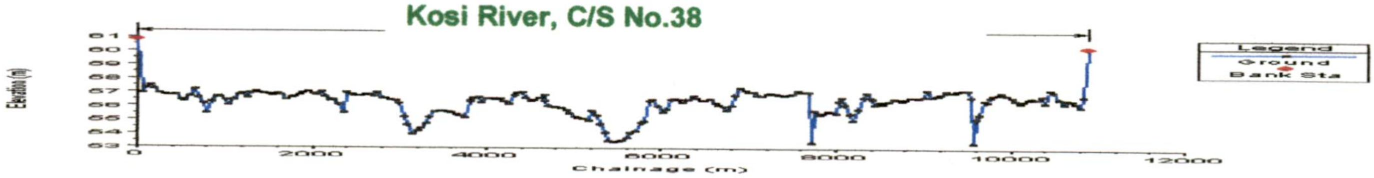
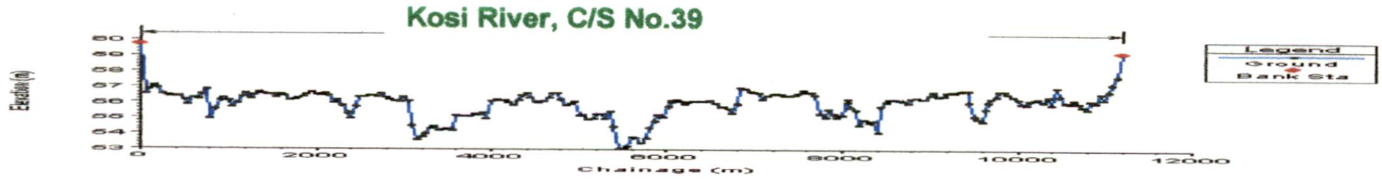
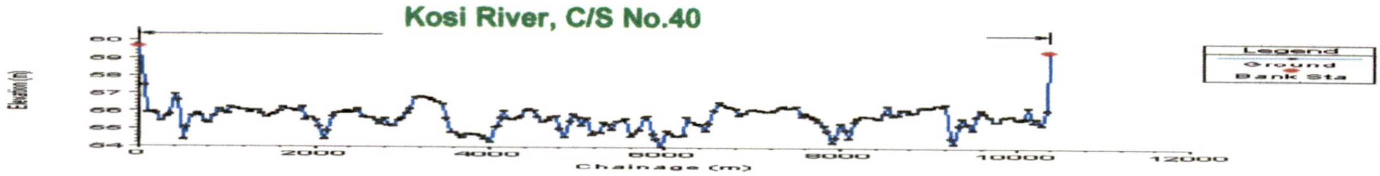



Kosi River, C/S No.39
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta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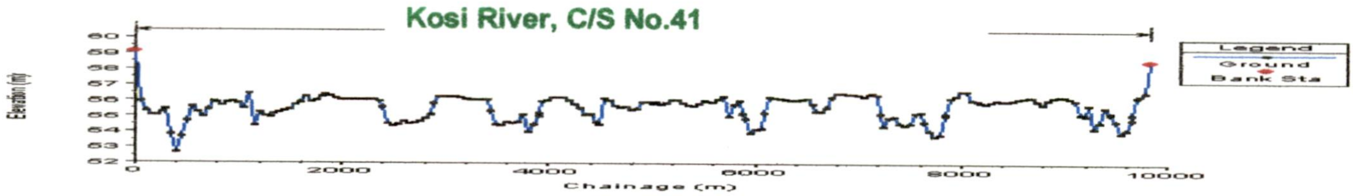
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta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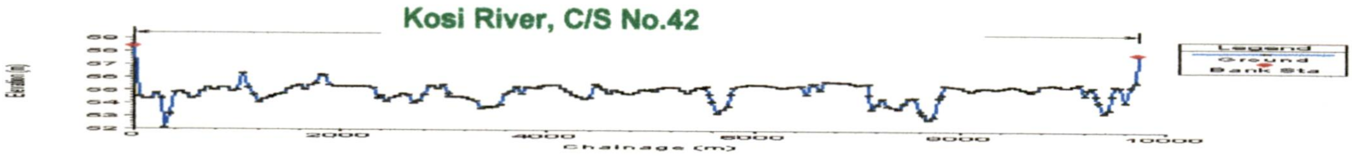
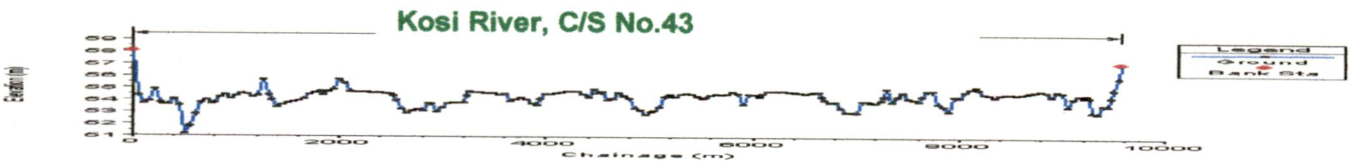
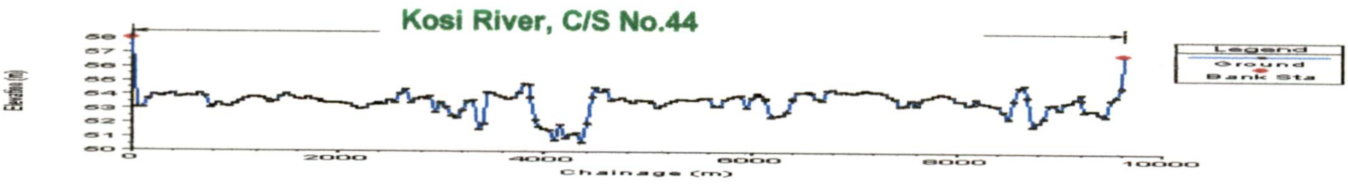
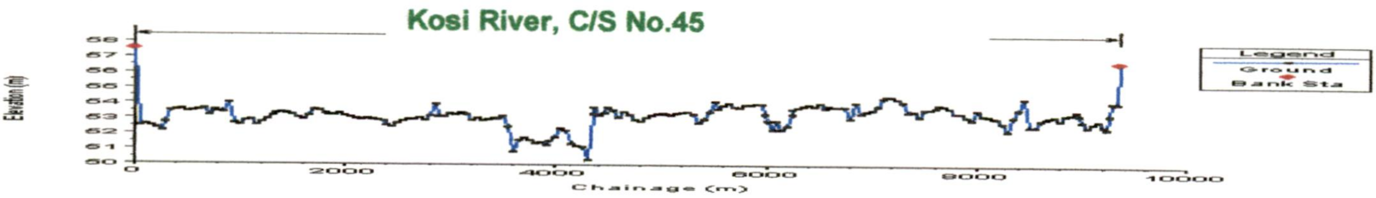
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta


Kosi River, C/S No.42
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta


Kosi River, C/S No.43
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta


Kosi River, C/S No.44
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta


Kosi River, C/S No.45
Elevation (m)
Chainage (m)
Legend
Ground
Bank Sta

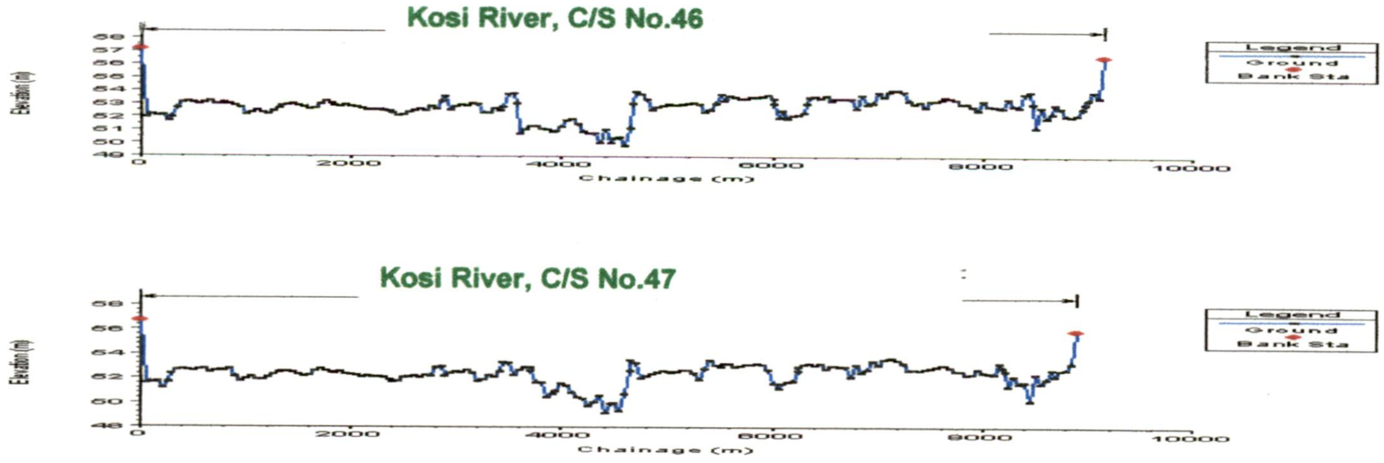
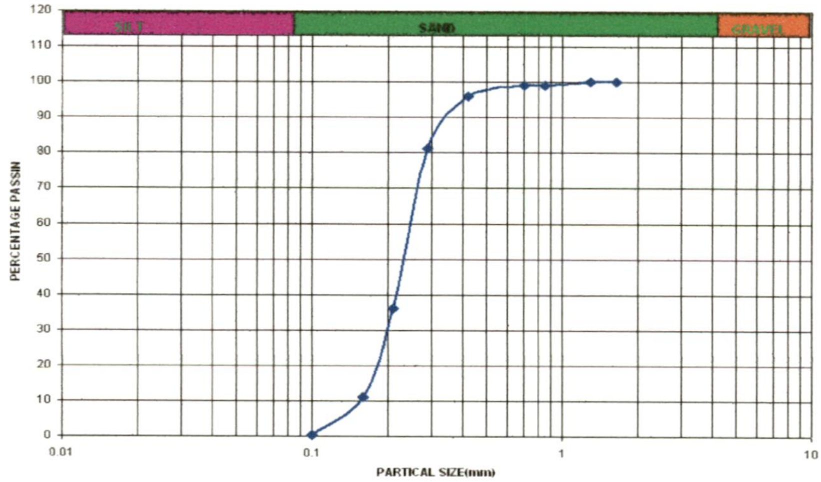
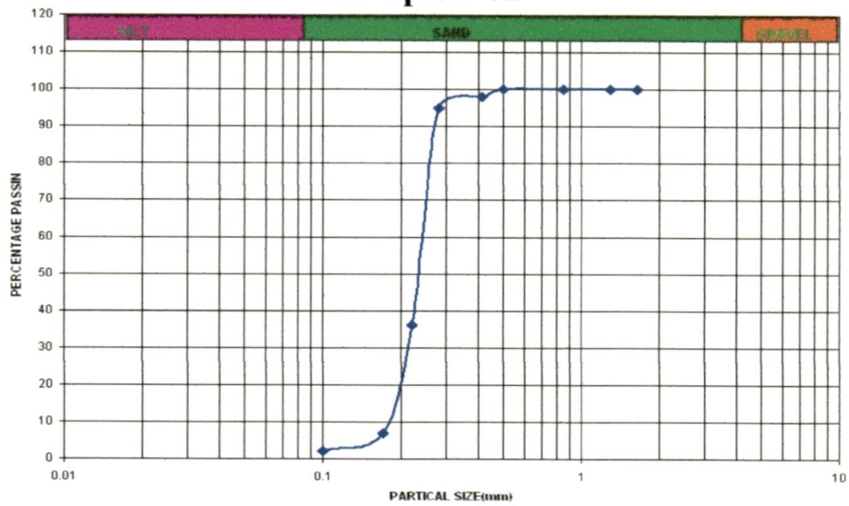


Figure :-3.2 Cross Sections From Barrage to 47 km. Downstream



Sample:- 02



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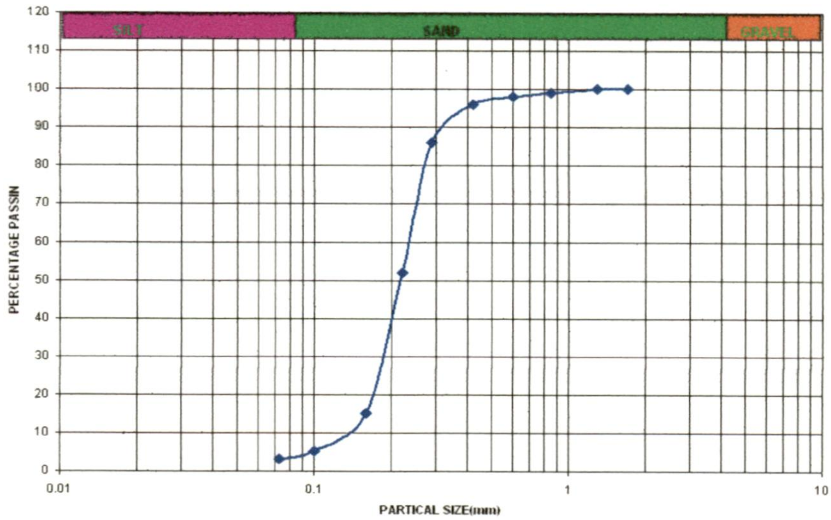

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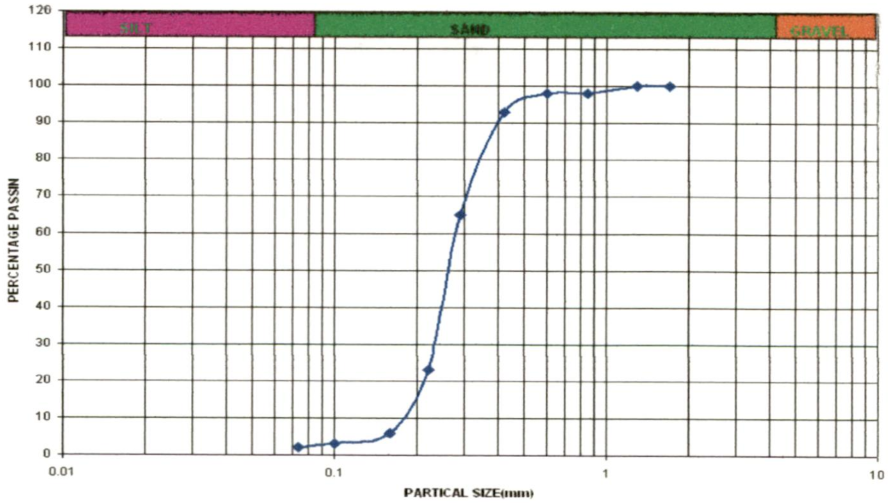

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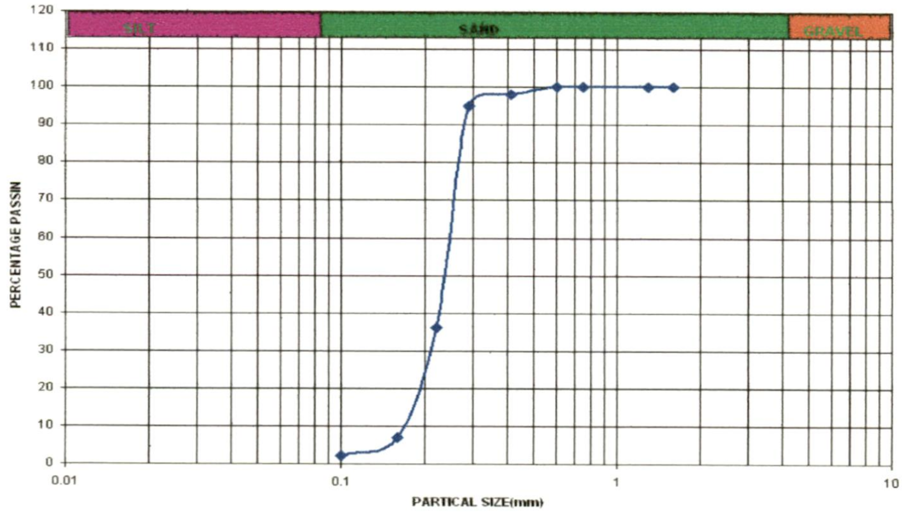


Sample:- 06

Figure 3.4: Gradation Curve's of Samples

3.3 Discharge and Sediment Data

Water and sediment discharge have been measured regularly for a number of years at Barakshetra located upstream of Chatra and similarly at Kosi Barrage. Regarding accuracy in measurement Kosi Barrage data is more reliable. So for computation purpose Kosi Barrage date is used. Here daily discharge and sediment data is used. Table 1.1 shows the annual maximum discharge for Kosi river.

Immediately downstream of the barrage sediment concentration sample is taken. For a period of June 2001 to April 2003 water discharge and sediment data is supplied by

Kosi project Authority. This data is collected daily and recorded at Kosi barrage. With the help of discharge-sediment data, a relationship between water discharge Q and the sediment discharge G plotted on log-log graph is developed which is required for computation with HEC-RAS-4 and was obtained using data of sediment and water discharge for different ranges of sediment size. These are shown plotted in **figure3.4.** These relation were approximated by the following equation and the values calculated for different discharge are in **table No-3.2**

$$Q = A * G^P$$

Where Q = water discharge (m^3/s)

G = sediment discharge

$A = 11.739$

$P = 0.3962$

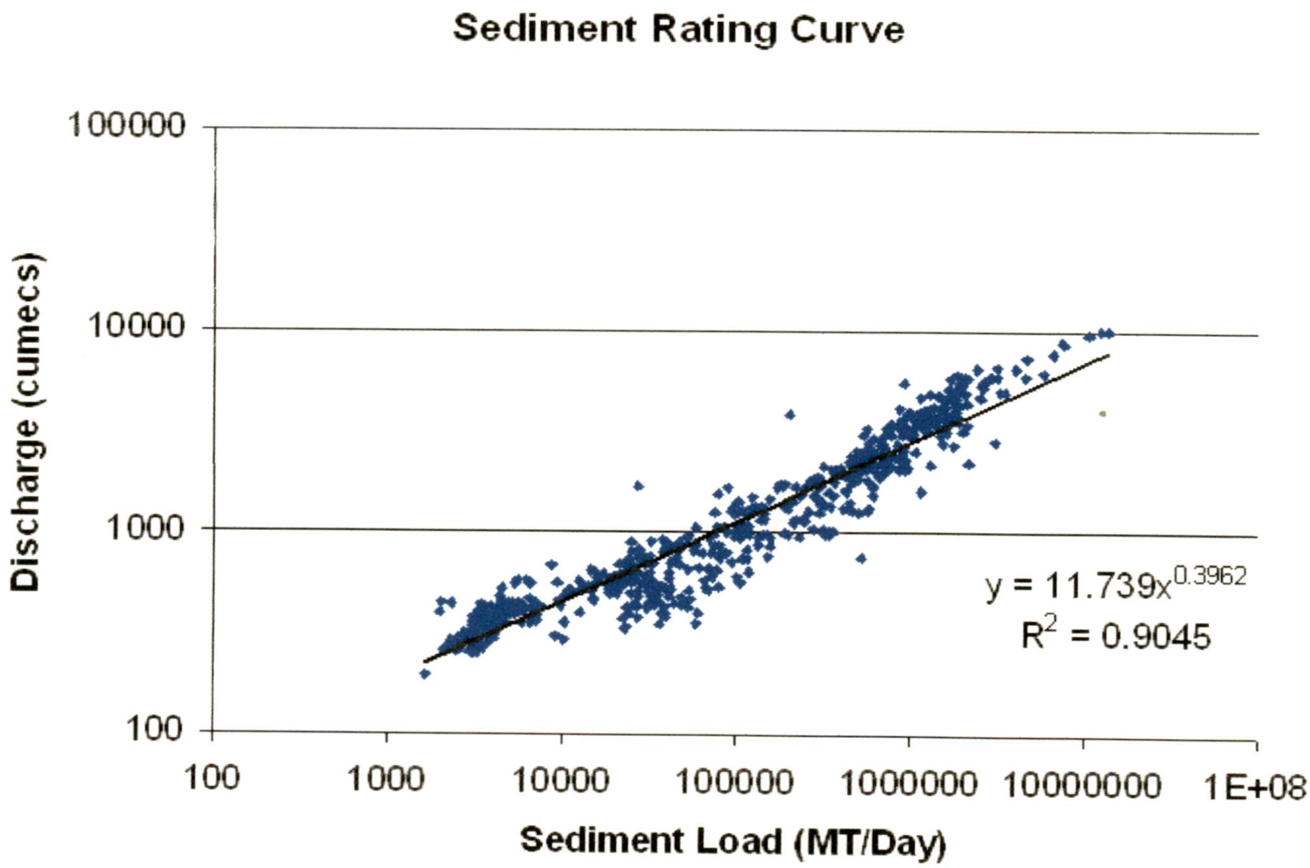


Figure – 3.5: Sediment load Vs Discharge graph on log-log

Table No.3.2

Discharge (Cumecs)	Sediment Load (MT/Day)	Discharge (Cumecs)	Sediment Load (MT/Day)
50	38.76478769	13500	53106239.21
500	12954.33723	14000	58211652.34
1000	74508.91197	14500	63602657.79
1500	207328.3294	15000	69284555.85
2000	428549.7484	15500	75262560.49
2500	752661.956	16000	81541803.56
3000	1192481.558	16500	88127338.67
3500	1759640.225	17000	95024144.77
4000	2464871.409	17500	102237129.5
4500	3318196.476	18000	109771132.2
5000	4329053.845	18500	117630926.8
5500	5506393.015	19000	125821224.6
6000	6858745.591	19500	134346676.6
6500	8394280.515	20000	143211875.9
7000	10120848.04	20500	152421359.7
7500	12046015.41	21000	161979611.7
8000	14177096.33	21500	171891063.9
8500	16521175.58	22000	182160098.1
9000	19085129.92	22500	192791047.9
9500	21875645.91	23000	203788200.5
10000	24899235.37	23500	215155797.6
10500	28162248.79	24000	226898037.6
11000	31670887.13	24500	239019076.3
11500	35431212.24	25000	251523028.6
12000	39449156.07	25500	264413969.7
12500	43730529	26000	277695935.9
13000	48281027.16		

Chapter 4

MODEL PROVING STUDIES

4.1 Introduction

Proving studies were carried out for verifying the conformity between mathematical model and proto type data in respect of water levels and water surface profile in Kosi river using different Manning's co-efficient n and as well as from previous experience.

For calibration/validation all water level studies are carried out assuming rigid bed during short duration flood event

4.2 Required Data

The data required for HEC- RAS-4 computation are Geometric and Hydrologic data the details are described as follows

a)Geometric data:- i) Cross section and Reach length ii) Manning's n values

iii) Bed width iv) contraction/Expansion coefficient.

i) Cross section and Reach length: - in this the modular develop the geometric data by drawing in the river system schematic on the geometric data window.

The water level and discharge data at boundaries and two or three locations within model reach is required for supplying data for boundary condition and model calibration

ii) Manning's n values:- various n values were used using same geometric and hydrodynamic data for fairly matching the water level for known gauge site

iii)) Bed width:- the Kosi river reach which is considered for mathematical model studies is from Kosi Barrage to 47 km. Downstream, which is constricted by constructing embankment on both site.

iv) contraction/Expansion coefficient:- By default HEC-RAS software takes contraction/Expansion coefficient as 0.1/0.3 for steady flow.

4.3 1st Method for Calibration of model

The discharge and water level data of Kosi river recorded during monsoon of year 2002 was utilized for this purpose. The maximum discharge recorded at Kosi barrage during this period was 10960 cum/s. (387000 cusecs.). For discharge of 10960 cum/s. at Kosi barrage, the corresponding water levels recorded at Dagmara and Bhaptiahi were of the order of 64.3m and 60.20m respectively. The village Dagmara and Bhaptiahi is situated at c/s no. 22 and c/s no. 32. Using various n values for a discharge of 10960 cum/s , run of mathematical model is taken and the result which are as follows. (Table-4.1)

Table -4.1

Maximum discharge in the year 2002 = 10960 cum/s(387000cusecs)			
Gauge site	**Gauge levels**		
	As per Proto year 2002 recorded	As per mathematical model n = 0.025	As per mathematical model n = 0.022
Dagmara	64.30m	64.79	64.67
Bhaptiahi	60.20m	60.21	60.11

It is seen from table that for n = 0.022, water levels of mathematical are fairly well matching with recorded during the year 2002.

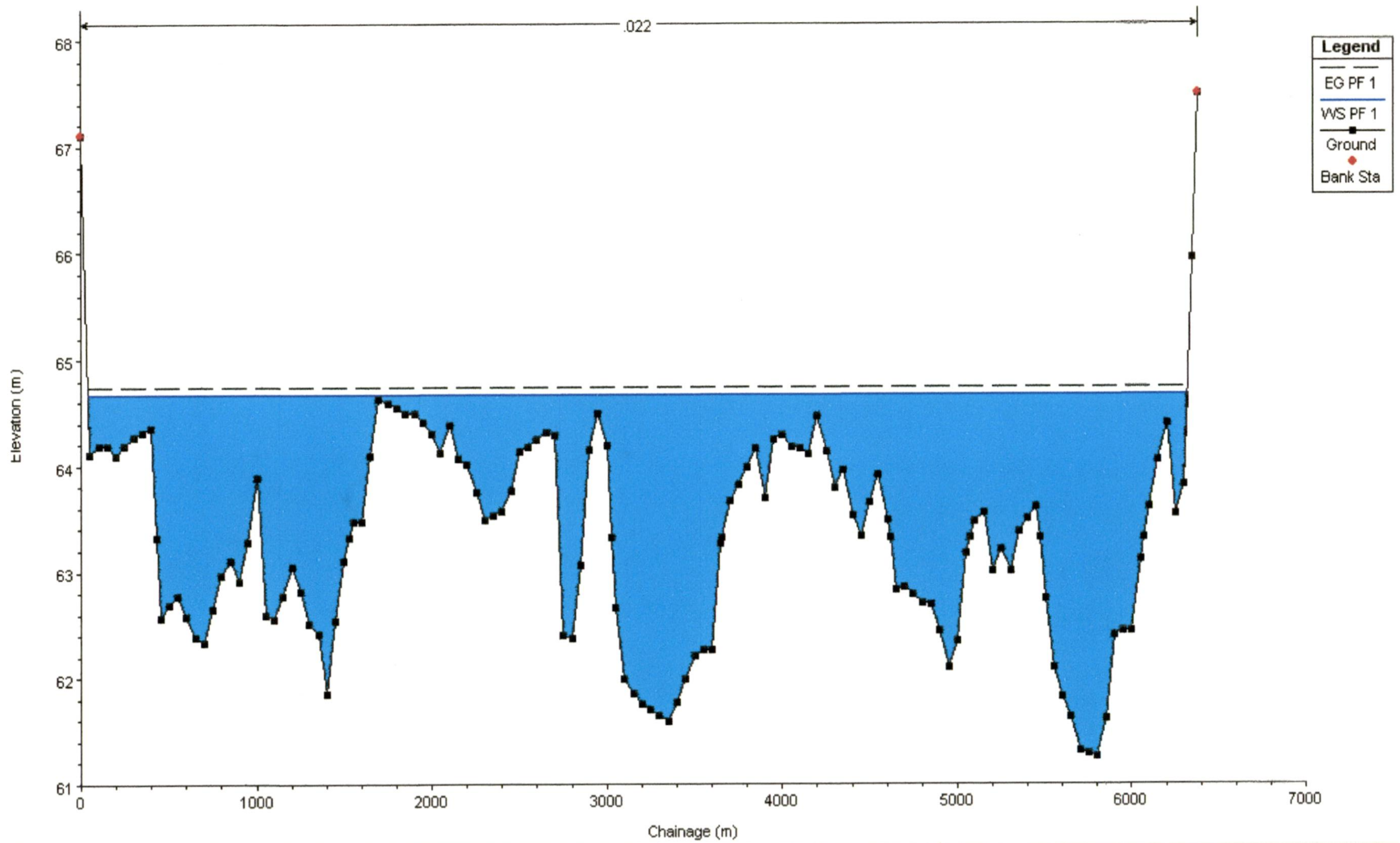


Figure:-4.1 C/S No. 22 at Dagmara fof Q = 10960, n = 0.022

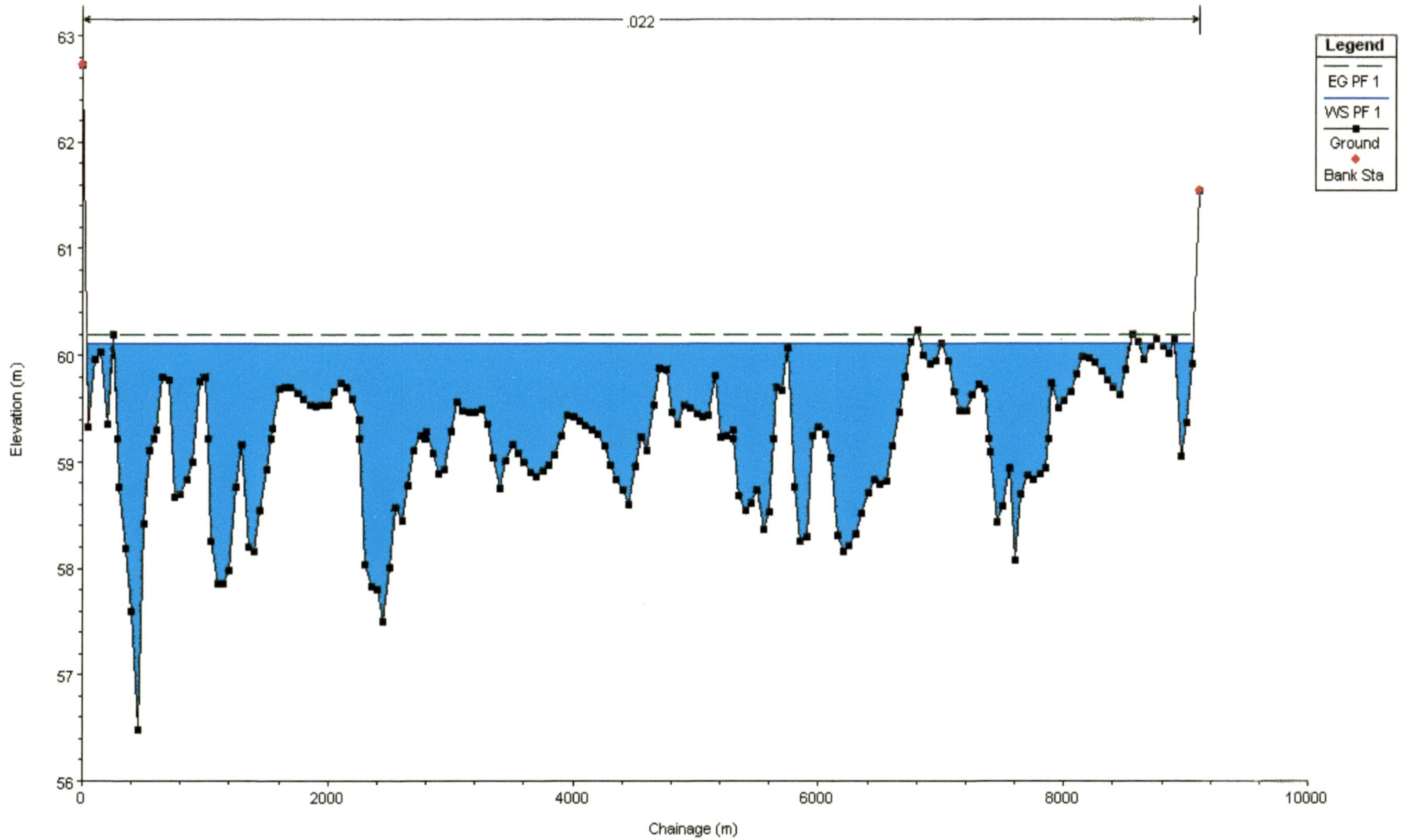


Figure:-4.2 C/S No. 32 at Bhaptiahi fof Q = 10960, n = 0.022

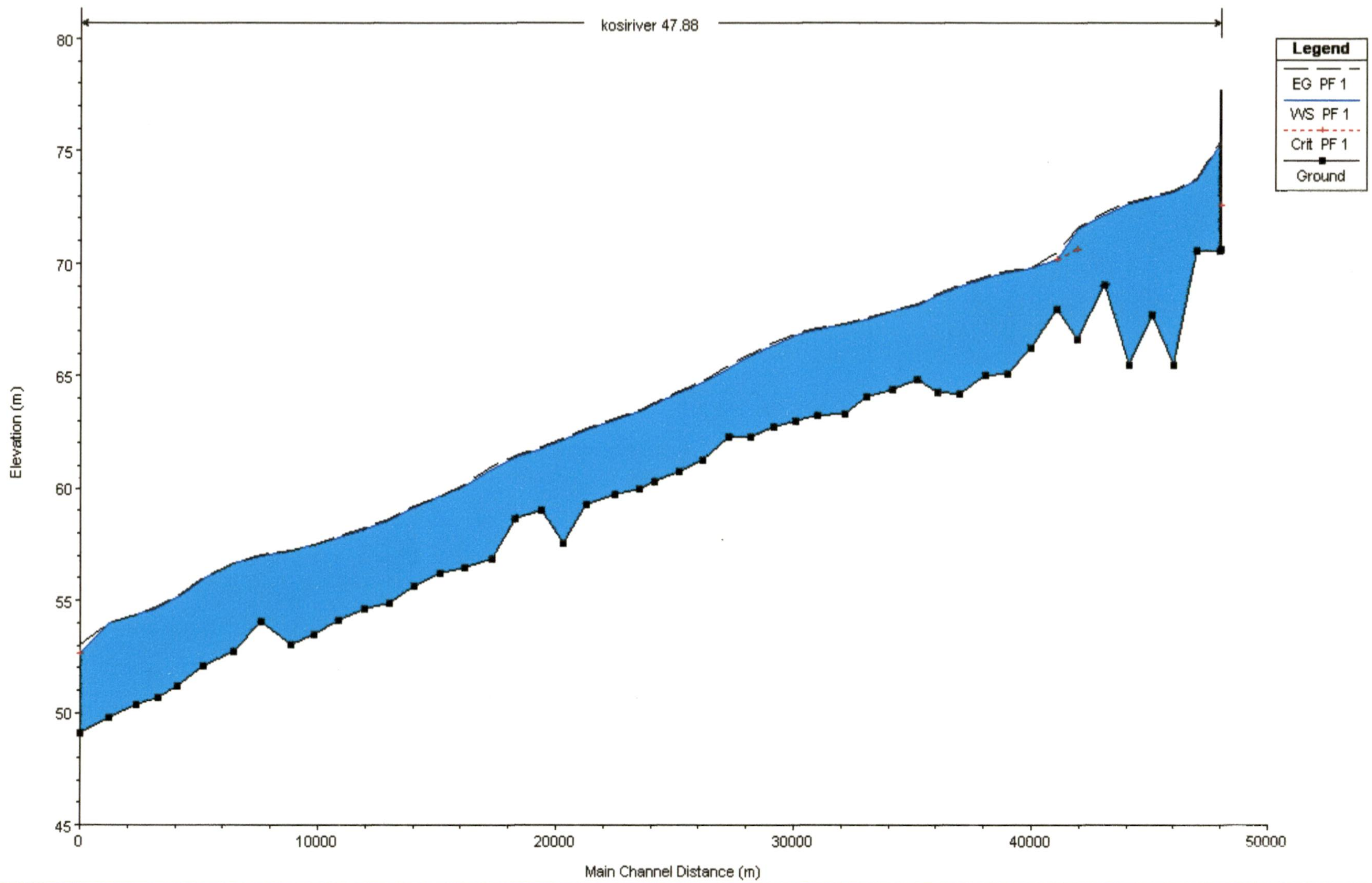


Figure:-4.3 Longitudinal Section showing water surface and bed profile.

4.4 2^{nd} Method for Calibration of model

The Kosi river reach 47 kms downstream of barrage was reproduced in the mathematical model using cross sectional data supplied by Project Authority. As per above calibration mannings Roughness coefficient of 0.022 was adopted. During survey work, which was done by WAPCOS on the month of April/May for the discharge of 2000 cu.m/s. water level was also recorded. For these model runs, discharge of 2000 cu.m/s was given as upstream boundary condition water level at cross section no.47 as per WAPCOS Survey was given as downstream boundary condition. Figure 4.4 and Table no.4.2 shows that the predicted and observed water levels are fairly matching.

Table -4.2

Cross Section Nos.	Observed Water Level in (m)(Proto)	Predicted Water Level in (m)(Proto)
1	72.6	72.25
2	72.55	71.61
3	71.52	71.52
4	71.25	71.42
5	70.88	70.9
6	70.1	70.28
7	69.1	69.24
8	68	68.39
9	67.9	68.26
10	67.8	68.1
11	67.4	67.83
12	67.1	67.63
13	66.6	66.84
14	66.21	66.57
15	66.12	66.2
16	65.5	65.97
17	65.52	65.7
18	65.12	65.42
19	64.61	65.08
20	64.28	64.77
21	64.07	64.1
22	63.33	63.23
23	62.87	62.93
24	62.66	62.53
25	62.38	62.25
26	61.98	61.89
27	61.22	61.48
28	60.78	61.04
29	60.42	60.64
30	60.11	60.16
31	59.68	59.79
32	59.22	58.75
33	58.8	58.03
34	58.4	57.44
35	58	57.16
36	57.28	57
37	56.87	56.65
38	56.45	56.3
39	56.05	56.15
40	55.64	55.93
41	55.23	55.54
42	54.82	54.99
43	54.41	54.15
44	53.85	53.69
45	53.344	53.22
46	53.034	52.71
47	52.624	51.17

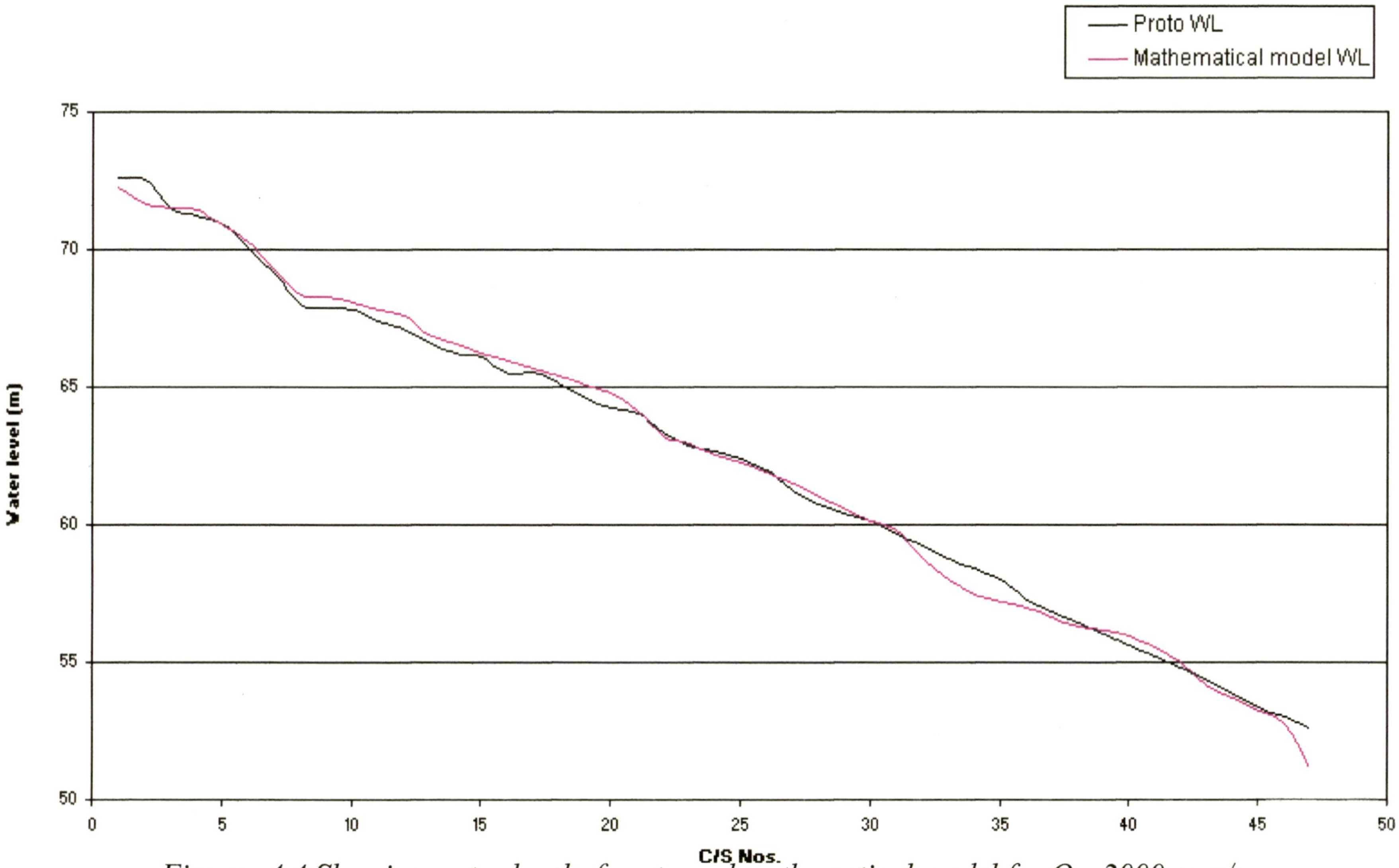


Figure:-4.4 Showing water level of proto and mathematical model for Q= 2000 cum/s.

Chapter 5

METHODOLOGY OF HEC-HAS (Version 4) 1D MATHEMATICAL MODEL

5.1 Introduction

Hydrologic Engineering Centre of U.S. Army Crops of Engineers, USA has designed and developed River Analysis System (HEC-RAS, Version 4.0, March 2008).

The U S Army Crops of Engineers, River Analysis System (HEC-RAS) is software that allows to perform one- dimensional steady and unsteady flow river hydraulic calculation.

HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking, multi-user network environment. The system 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 contains three one-dimensional hydraulic components for: 1)Steady flow computation; 2) Unsteady flow simulation; and 3) movable boundary sediment transport computation. 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 the basic water surface profiles are computed.

5.2 Steps in developing a Hydraulic Model with HEC-RAS.

There are five main step in creating a Hydraulic model with HEC-RAS :

5.2.1 Starting a new project

The first step in developing a Hydraulic model with HEC-RAS is to establish which directory you wish to work in and to enter a title for the new project. To start a new project, go to the file menu on the main HEC-RAS and select new project.

5.2.2 Entering geometric data

The next step is to enter the necessary geometric data, which consist of connectivity information for the stream system (river system schematic), cross section data, and hydraulic structure data (bridges, culvert, weirs etc).

Geometric data entered by selecting geometric data from the edit menu on the main HEC-RAS window. The modeler develops the geometric data by first drawing in the river system schematic. This is accomplished, on a reach-by-reach by basis, by pressing the river reach button and then drawing in a reach from upstream to downstream (in the positive flow direction). After the reach is drawn, the user is prompted to enter a “River” and a “Reach” identifier. The river and River identifiers can be up to 16 characters in length. As reaches are connected together, junctions are automatically formed by the interface. The modeler is also prompted to enter an identifier for each junction.

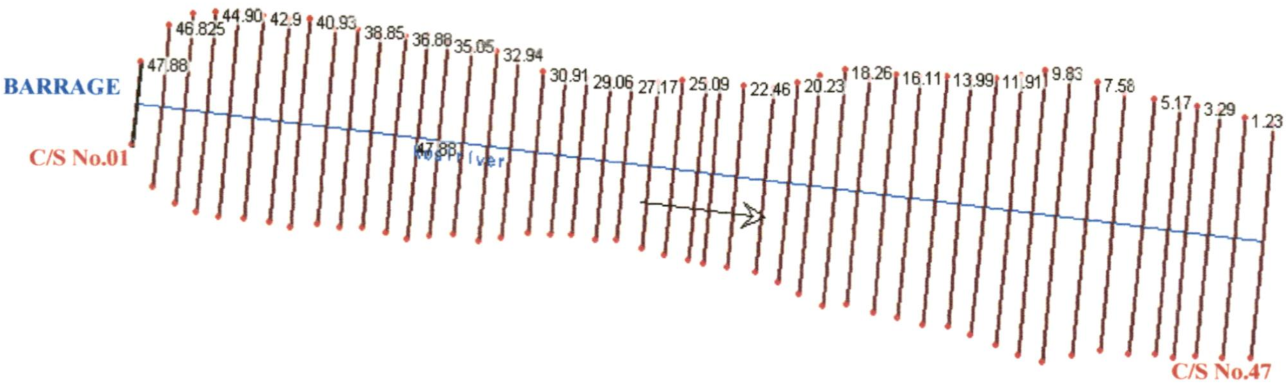
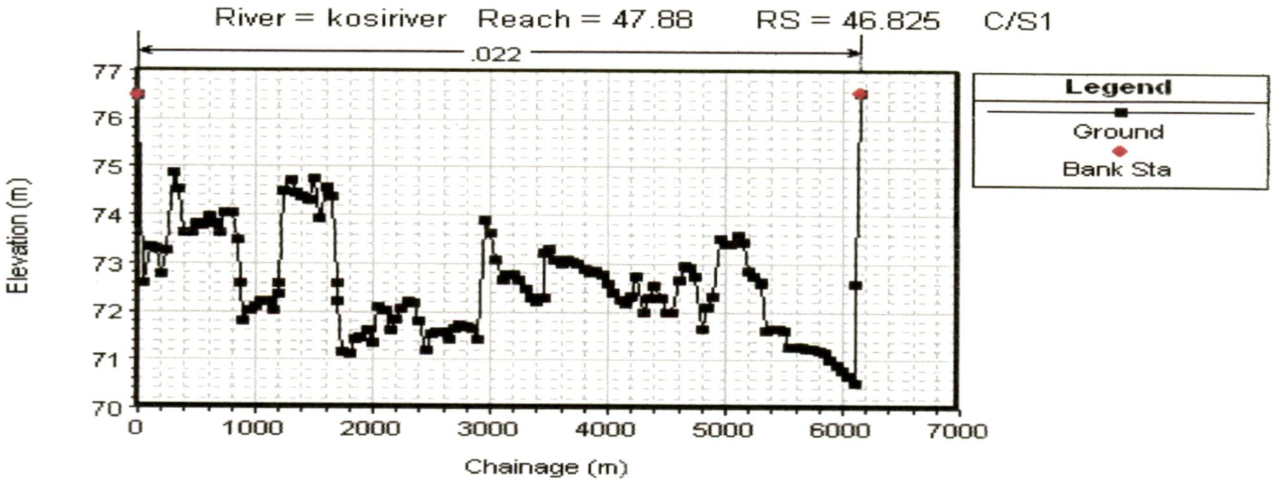


Figure-5.1 Plan of river showing cross section from Barrage to 47km. downstream. (as per mathematical geometric data)

After the river system schematic is drawn, the modeler can start entering cross section and hydraulic structure data. Each cross section should have a river name, reach name, river station and a description. The river, reach and river station identifiers are used to described where the cross section is located in the river system. The “river Station” identifier does not have to be the actual river station (miles or kilometer) at which the cross section is located on the stream, but it does have to be a numeric value (e.q. 1.1, 2, 3.5, etc)

The numeric value is used to place cross section in the appropriate order within a reach from the highest river station upstream to the lowest river station downstream.

Once the cross section data entered, the modeler can them add any hydraulic structure such as bridges, culverts, weirs and spillway. Data editor, are available for the various types of hydraulic structure. If there are any stream junctions in the river system, additional data are required for each junction. The junction data editor is available from the geometric data window.



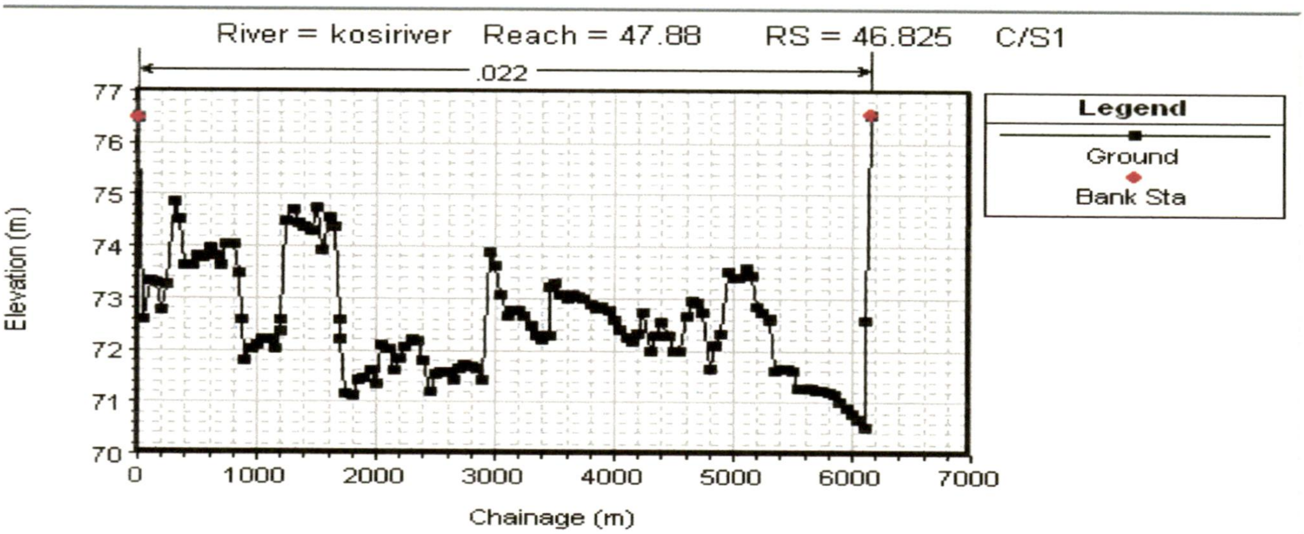


Figure- 5.2 Typical cross-section near barrage and 47 km. downstream

5.2.3 Entering Manning' n values

Manning's n values are required to be specified in the input data. The n values can be changed with distance and also with elevation. Use of different Manning's n values for different portions of the cross section is also permissible.

Here the value of Manning's n is 0.022, which is as per table no. 4.1 of chapter no.4

5.2.4 Movable bed width

The movable bed width of the cross section is that width over which the scour can occur. The movable bed width and the depth of sediment in the movable bed are required in the input data of the programme. The former was specified as the total width of the river between levees at every section in the present study. A value of 5.00 metre was specified as the depth of sediment. this value should be more than the scour depth of each cross section.

5.3 Sediment Data

The following data are required in this category

5.3.1 Inflow sediment load

The inflow sediment load is given as input in the form of a total sediment load versus water discharge relationship. The program requires that the sediment loads be given in tons/day.

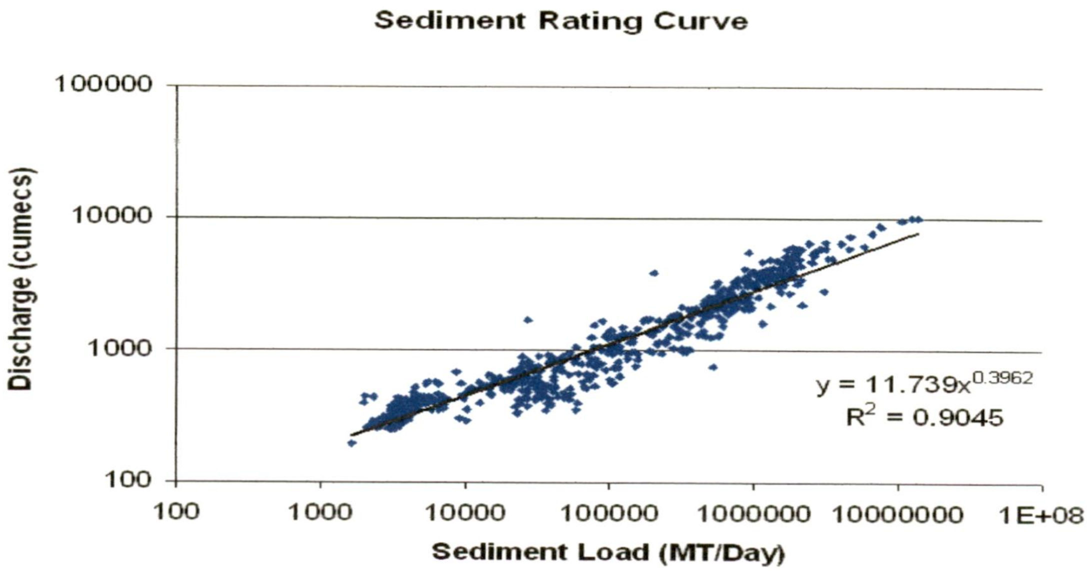


Figure – 5.3 plot of sediment load Vs discharge on log-log paper

With the help of discharge-sediment data, a relationship between water discharge Q and the sediment discharge G plotted on log-log graph is developed which is required for computation with HEC-RAS-4 and was obtained using data of sediment and water discharge for different ranges of sediment size.

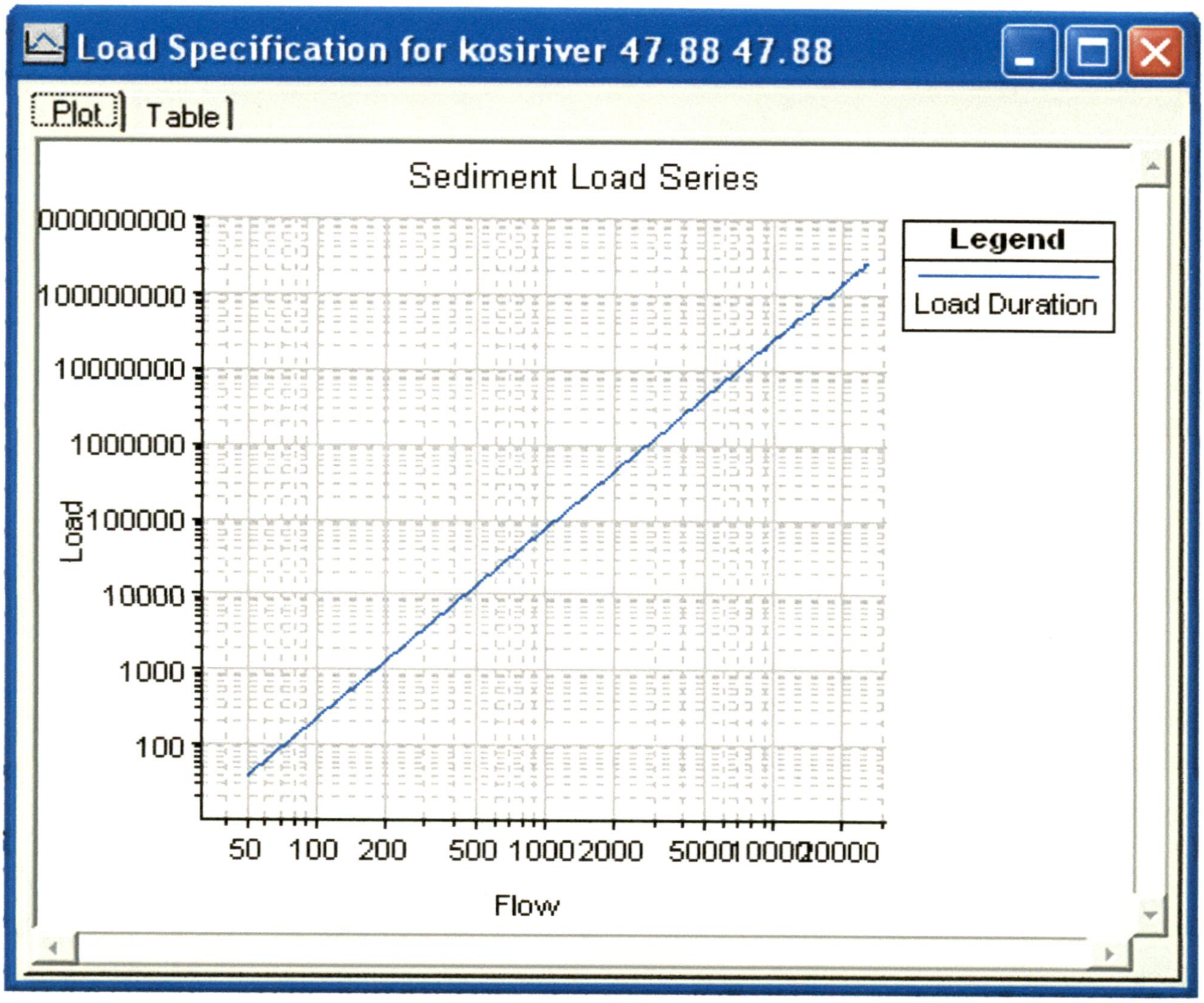


Figure – 5.4 Plot of sediment load Vs discharge by HEC-RAS

5.3.2 Gradation of Stream (bed material)

The fraction of stream bed material contained in each grain size is required to be given as input to describe the stream bed material gradation.

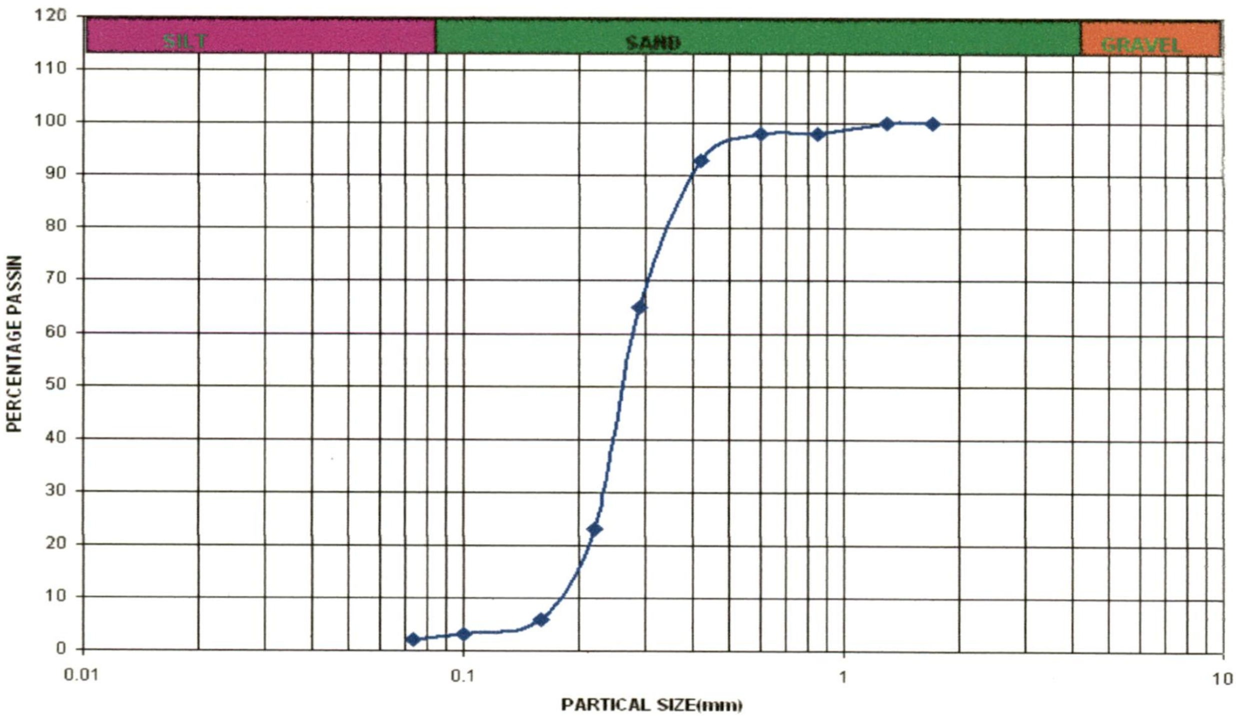


Figure – 5.5 Typical size distribution of bed material

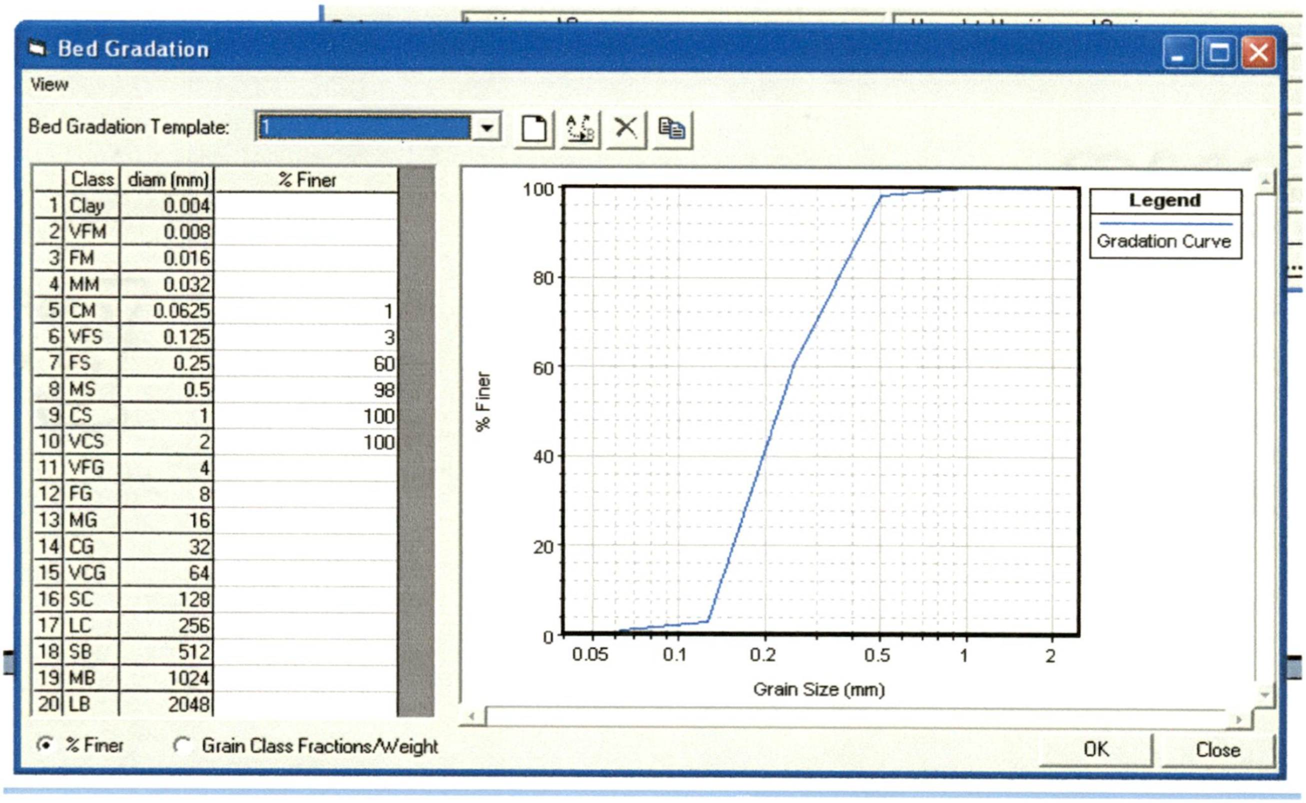


Figure – 5.6 Plot of Grain size Vs %Finer by HEC-RAS

5.5.3 Sediment Transport Potential

Sediment transport potential is the measure of how much material of a particular grain class a hydrodynamic condition can transport. Transport potential is computed with one of the number of sediment transport equation available in the program. Since most of this equations were developed to be used for a single grain size, like the d_{50} (or at the most, two grain sizes like d_{50} and the d_{90}), the equation is applied independently to each grain class present in the system.

2.4.3.2.a) Acker and White

Acker and White (1973) is a total load function that was developed from flume data for relatively uniform gradations ranging from sand to the fine gravels. Hydrodynamic were selected to cover a range of bed configurations including ripples, dunes and plane bed conditions. Suspended sediment is a function of shear velocity while bedload is a function of shear stress.

2.4.3.2.b) England Handersen

England Handersen (1967) is a total load transport equation that was developed flume data. Relatively uniform sand sizes between 0.19 to 0.93 mm were used. The attraction of England Handersen is that it is not complicated function. Instead, it is relatively simple function of channel velocity, bed shear and the d50 of the material. Application should be restricted to sand systems.

2.4.3.2.c) Laursen-Copeland

Laursen (1968) is also total load function that was initially based on flume equation and later expanded by Madden to include the Arkansas river data. It is a basic function of excess shear and a ratio of the shear velocity to the fall velocity,. Later, Copeland (1989) generalized the equation for gravel transport so the equation could be used for graded beds. The distinctive future of Laursen is that the sediment material the function was developed for extends down into the silt range. None of other functions currently included in RAS were developed for silt sized particles. Any sediment potentials computed for silt, by the other function, would be extrapolation, compounding extrapolation errors on top of the standard uncertainty associated with computing transport capacity. Recent work at Colorado State has demonstrated that the Laursen equation out performs other transport function in the silt range.

2.4.3.2.d) Meyer-Peter Muller

the Meyer-Peter and Muller (MPM) equation (1948) was one of the earliest equation developed and is still one of the most widely used. It is a simple excess shear relationship. It is strictly a bed load equation developed from flume experiments of sand and gravel under plane bed condition. Most of the data was developed for relatively uniform gravel substrates-MPM is most successfully applied over the gravel range. It tends to under predict the transport of finer material. Recently, Wong (2003) and Parker (2007) demonstrated that this function over predicted transport by, approximately, a factor of two. This conclusion was not based on new data but on a reanalysis of MPM's original results. To improve the function , they recast the base , excess shear equation:

$$q_b^* = 8(\tau^* - \tau_c^*)^{3/2} \quad , \quad \tau_c^* = 0.047$$

as

$$q_b^* = 3.97(\tau^* - \tau_c^*)^{3/2} \quad , \quad \tau_c^* = 0.0495$$

Where: q^*_b is the Einstein bedload number (correlated with bedload), τ^* is the Shield's stress which is compared to, τ^*_c which is the 'critical' Shields stress.

2.4.3.2.e) Toffaleti

Like England-Hansen, Toffaleti (1968) is a total load fuction developed primarily over sand sized particles. Toffaleti is generally considered alarge river function however, since many of the data sets used to develop it were large suspended load systems. The function is not heavily dependent on shear velocity or bed shear. Instead, it was formulated from regressions on temperature and an empirical exponent that describes the relationship between sediment and hydraulic characteristics.

The Toffaleti equation uses two different grain sizes, a d50 and a d65 in an attempt to quantify transport dependence on the gradational deviation from the mean.

2.4.3.2.f) Yang

Yang (1973, 1984) is a total load transport equation which bases transport on Stream Power, the product of velocity and shear stress. The function was developed and tested over a variety of flume and field data. The equation is composed of two separate relations for sand and gravel transport. The transition between sand – gravel is smoothed over in order to avoid large discontinuities. Yang tends to be very sensitive to stream velocity, and it is more sensitive to fall velocity than most.

2.4.3.2.g) Wilcock

Wilcock (2001, generalized from of the initial two fraction equation in Wilcock and Crowe, 2003) is a bedload equation designed for graded beds containing both sand and gravel. It is a surface transport method based on the theory that transport is primarily dependent on the material in direct contact with the flow. It was developed based on the surface gradations of flumes and rivers. Therefore, the bed gradations should reflect the bed surface properties. Wilcock, additionally has a hiding function that reduces the transport potential of smaller particles based on the premise that they are nestled between larger gravel clasts and do not experience the full force of the flow field (or the turbulent boundary layer).

Finally, the central theory of the Wilcock equation is that gravel transport potential increases as sand content increases. A dimensionless reference shear is computed for the substrate which is a function of the sand content of the bed surface:

$$\tau^*_{rm} = 0.021 + 0.015 \cdot e^{-20 \cdot FS}$$

Where τ^*_{rm} is the reference shear stress and FS is the sand content in percent. As the sand content increases: the reference shear decreases, the excess bed shear increases, and the total transport increases. The Wilcock equation is very sensitive to this sand content parameter. It tends to be most appropriate for bimodal systems and tends to diverge from the other equations for unimodal gravel or sand transport.

.**Laursen-Copeland** sediment transport equation was considered for computation of HEC-RAS Mathematical model because the distinctive future of Laursen is that the sediment material the function was developed for extends down into the silt range. None of other functions currently included in RAS were developed for silt sized particles.

5.5.4 Hydrologic data

The computer programme treats a continuous as a sequence of discrete steady flow events, each having a specified duration. Each discharge value of the hydrograph is given as input. The duration, in days for which this discharge remains constant and the water temperature are also as input. In the absence of any temperature measurements, a constant value of 70^0 F(21^0 C) was specified as the water temperature.

5.5.5 Generation of Discharges

10 daily average inflows were established on the basis of daily discharge data available for the year 1948 to 1966 and recent data from 2001 to 2003. The maximum discharge data per year was available and the daily discharge data available from the year 1948 to 1966 was used to generate the discharge data from year 2003 to 2008 **(Figure 5.7 Hydrograph)**

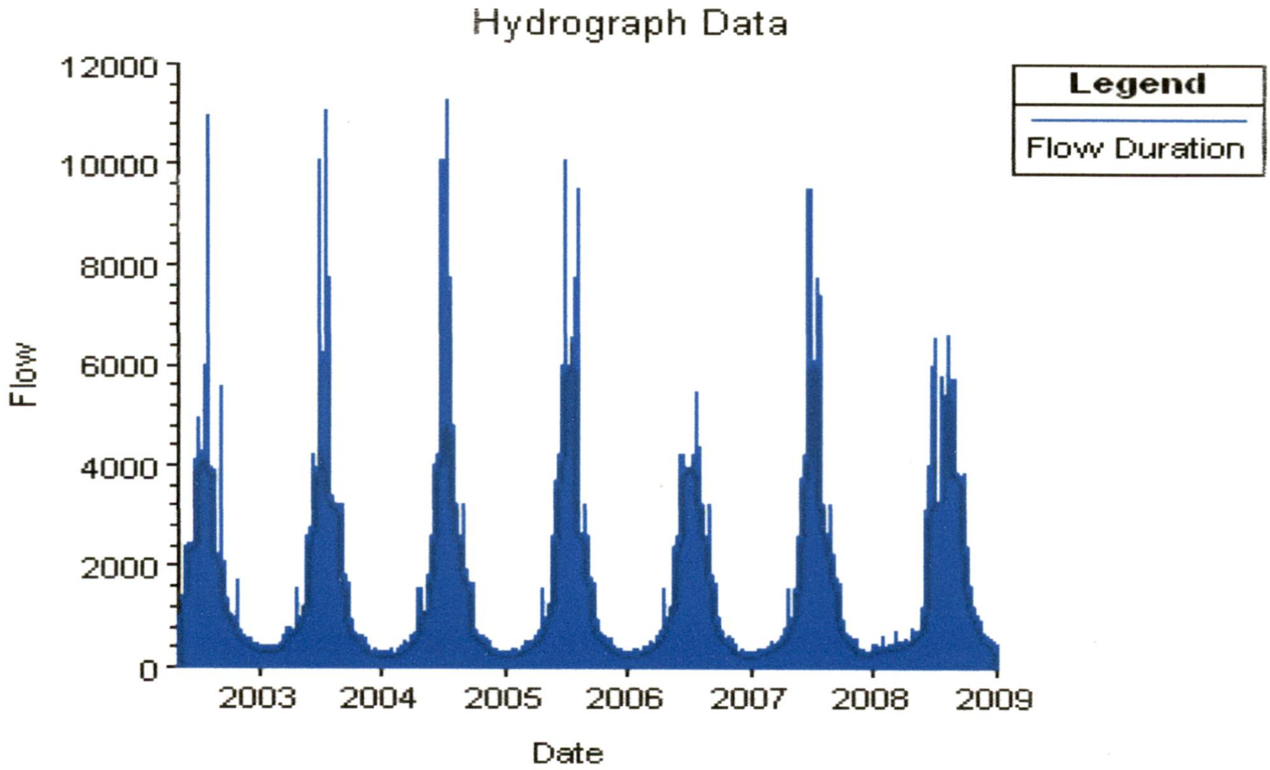


Figure 5.7 Hydrograph

5.5.6 Performing the hydraulic calculations

Once all of the geometric data are entered, the modeler can begin to perform the hydraulic calculations. As stated previously, there are three types of calculations that can be performed in the current version of HEC-RAS; Steady flow analysis, quasi-unsteady flow analysis, unsteady flow analysis and hydraulic design functions.

5.5.7 Viewing and printing results

Once the model has finished all of the computations, the modeler can be begin viewing the results. Several output features are available under the view option from the main window. These options include: cross-section plot: profile plot; rating curve plots; X-Y-Z perspective plots; tabular output at specific locations (detailed output tables); tabular output for many locations (Profile Summary Table); and the summary of error, warning and notes.

The longitudinal water surface profiles, cross-section, velocity plots, stage flow hydrographs change in bed level graphs were observed in rivers. The results are discussed in Chapter 6.

Chapter 6

RESULT AND CONCLUSION

6.1 Introduction

One dimensional mathematical model, HEC-RAS (Version 4) capable of handling sediment transport on mobile bed, was used to stimulate flows in Kosi river from barrage to about 47 km. downstream

For stimulate the flow, the data and condition are as follows:-

1) Water discharge Vs. Sediment load relationship used data from June 2002 to April 2003.
2) Grain size distribution curve.
3) Geometric data in the form of cross-section used year May 2002 and its plan.
4) Downstream boundary condition:- Gauge discharge curve.
5) Upstream boundary condition:- discharge from the year 2002 to 2008.(Generated discharge data between 2003 to 2008)
6) Temperature:- considered 21^{0}C
7) Scour depth 5m

5.2 Plan of Kosi River generated by HEC-RAS

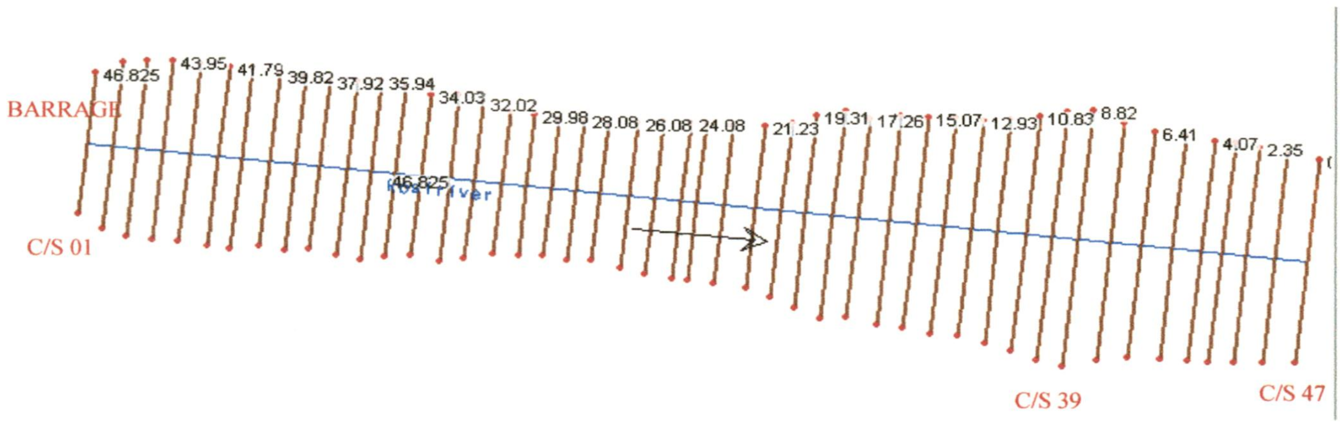


Figure – 5.1 Plan of Kosi River

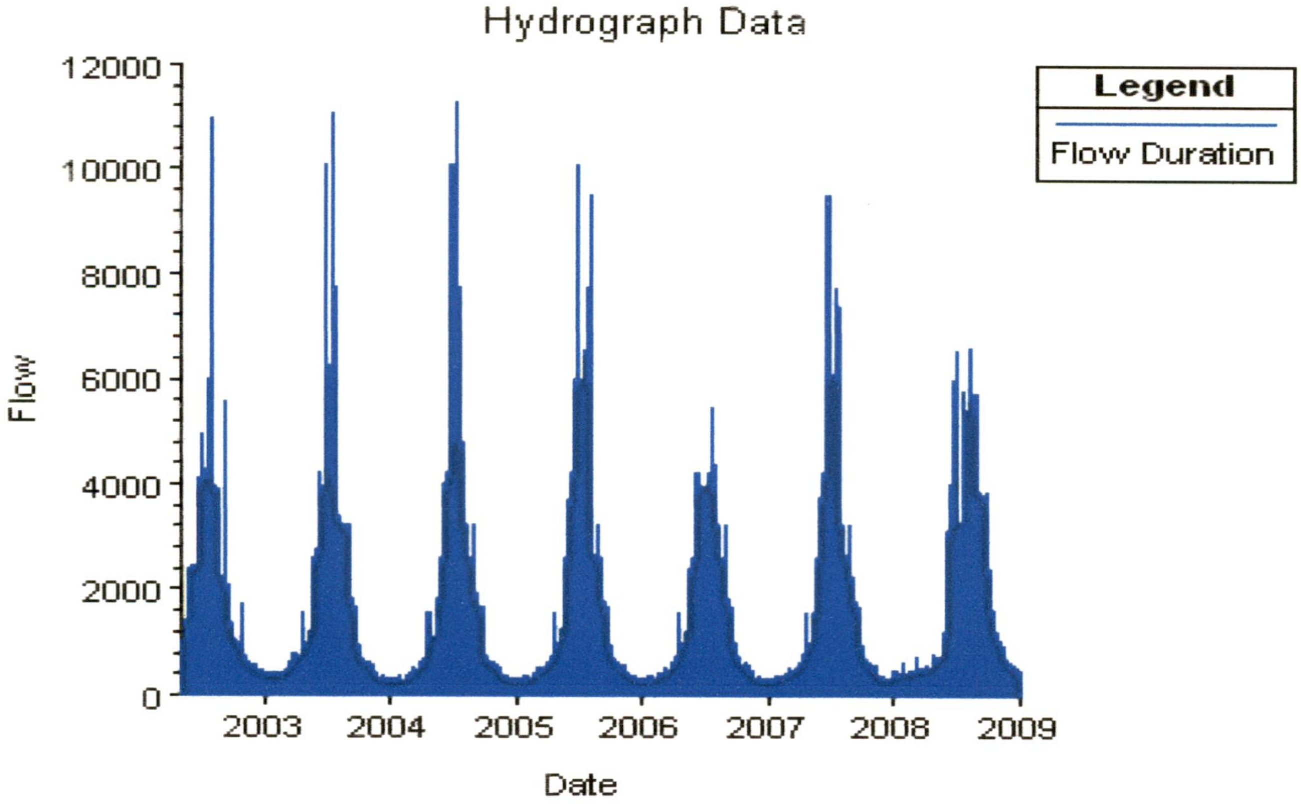


Figure – 5.2 Hydrograph

5.3 Longitudinal Section of River showing year wise Aggradation and degradation

Using discharge sediment relationship for the period of June 2001 to April 2003,cross section for the year May 2002 and generated discharge the Mathematical model run was taken and the changed year wise bed level predicted are as follows:-

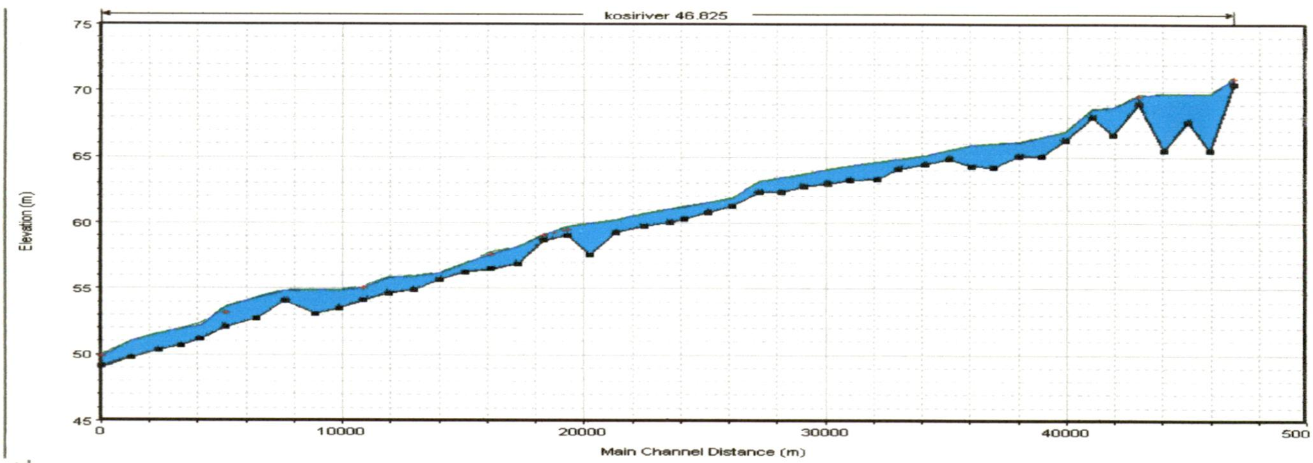


Figure-5.3a) Showing bed level profile in 01 May2002

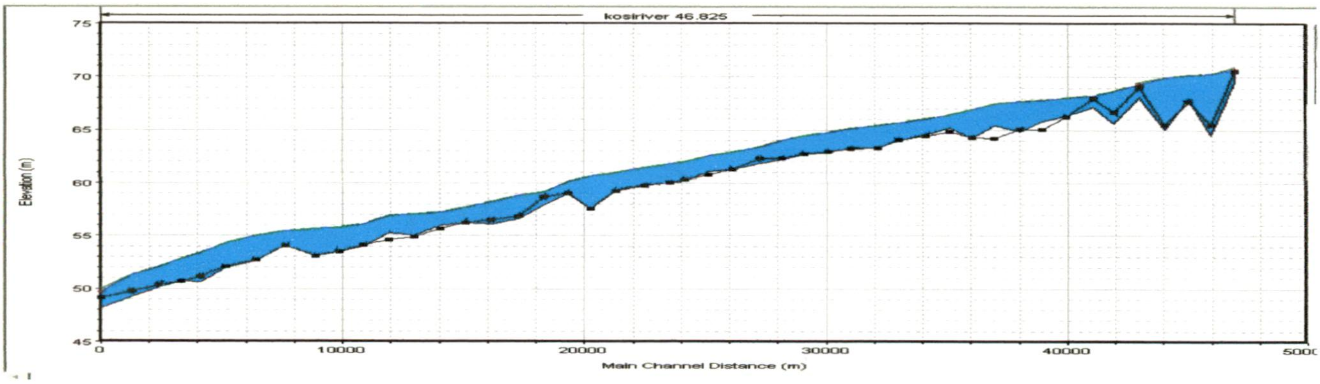


Figure-5.3b) Showing bed level profile in 01 May2003

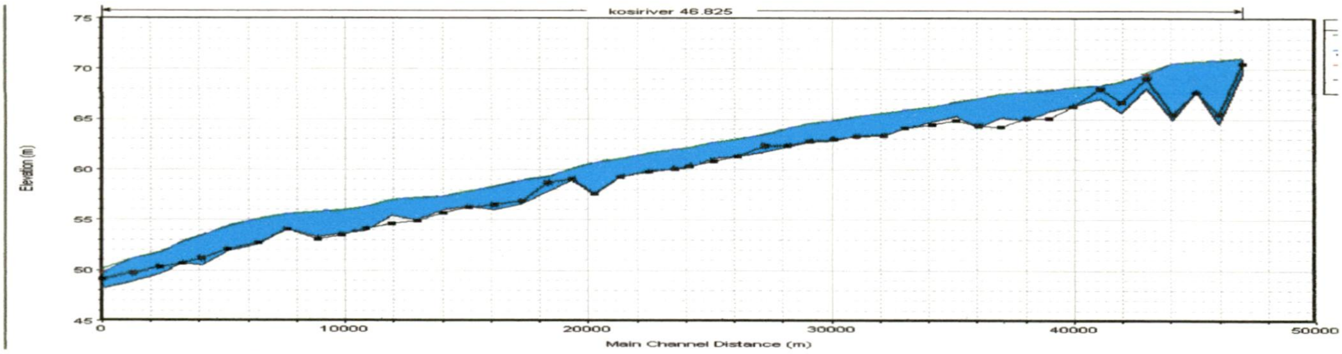


Figure-5.3c) Showing bed level profile in 01 May2004

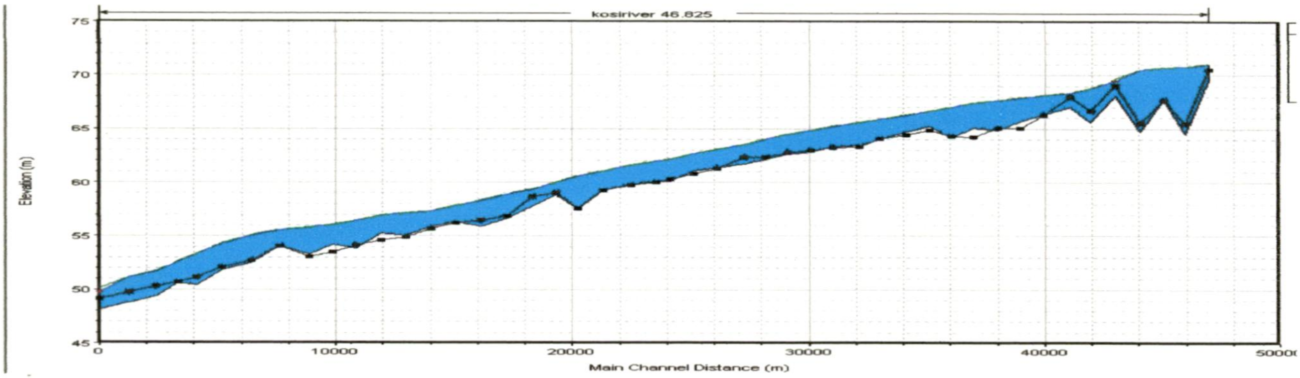


Figure-5.3d) Showing bed level profile in 01 May2005

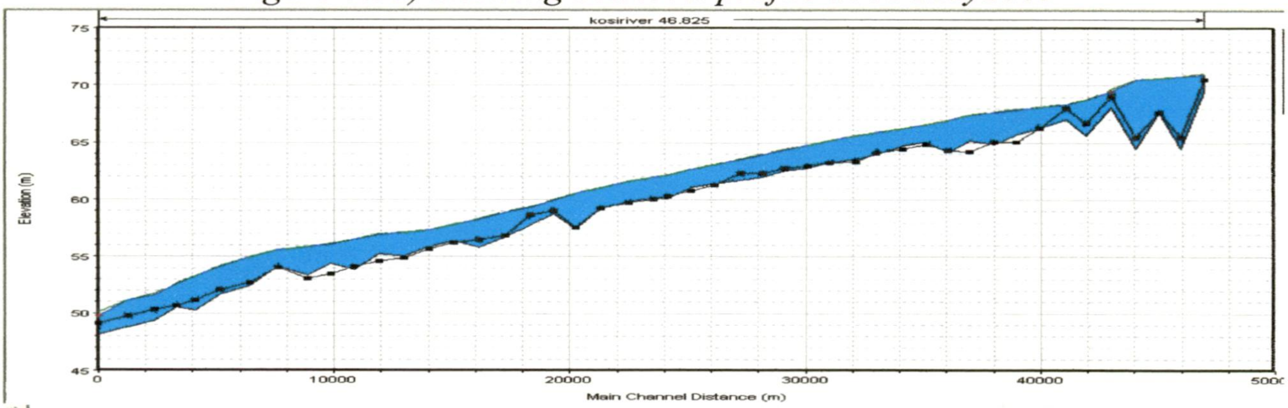


Figure-5.3e) Showing bed level profile in 01 May2006

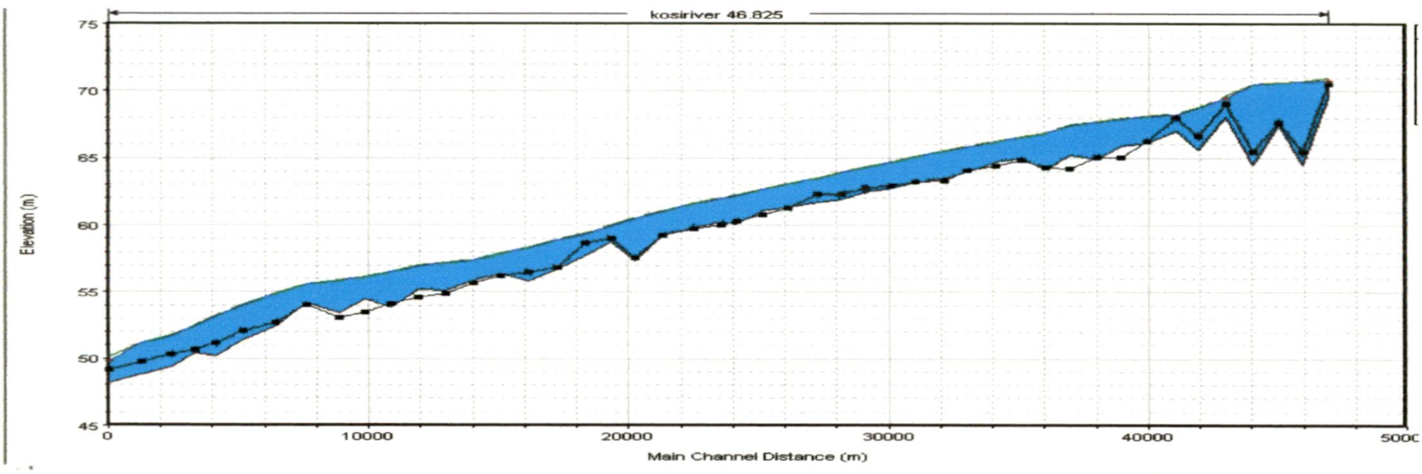
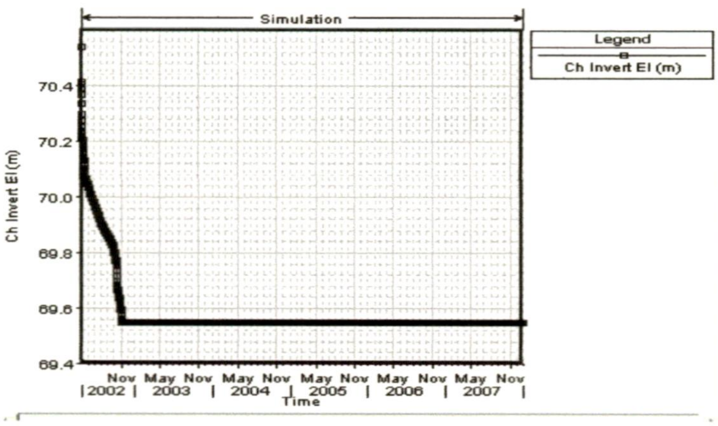


Figure-5.3f) Showing bed level profile in 01 May2007

5.4 Year wise change in bed level (Cross Section No. 01 to 47)



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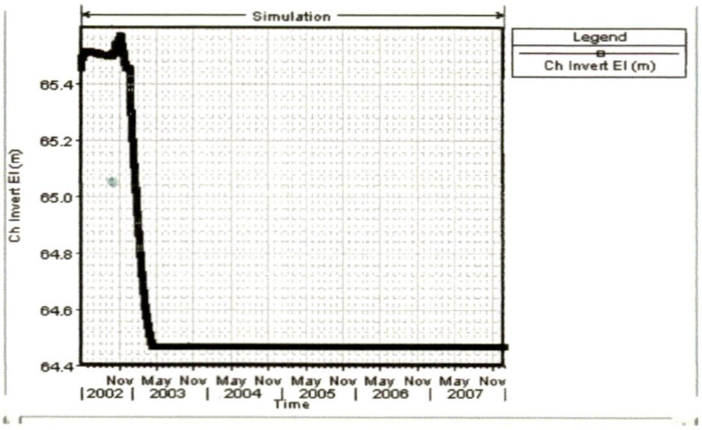

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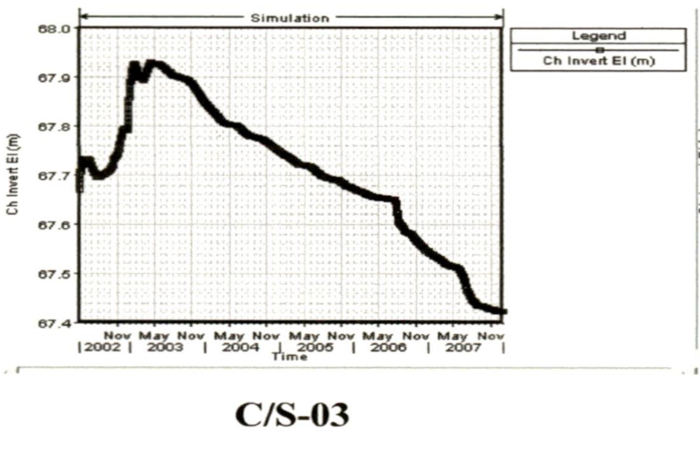

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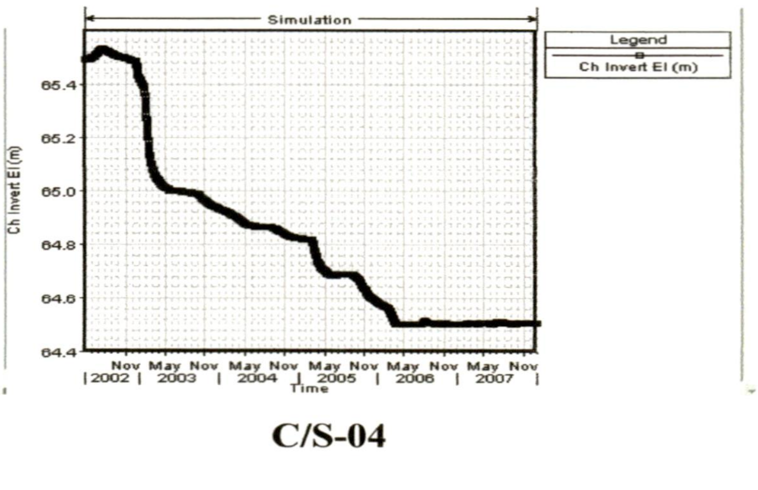

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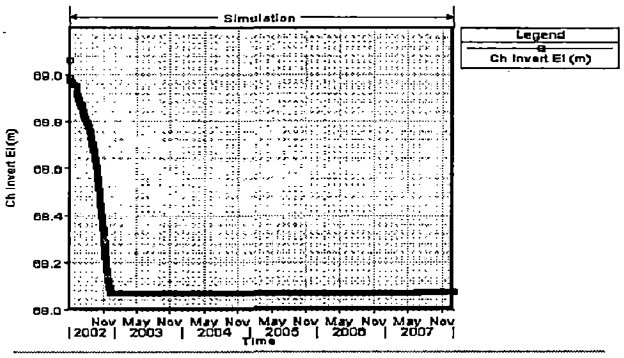

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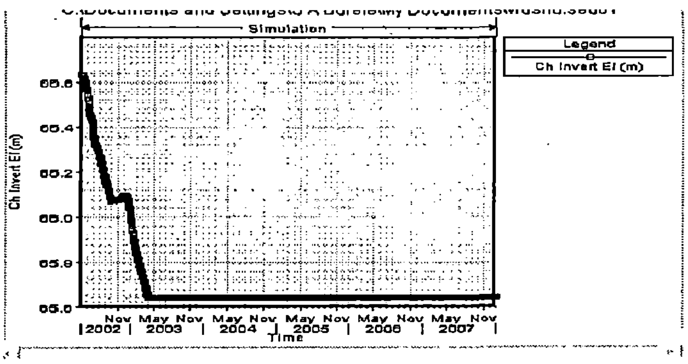

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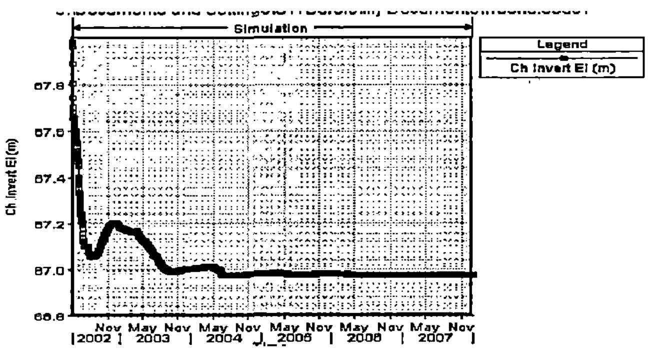

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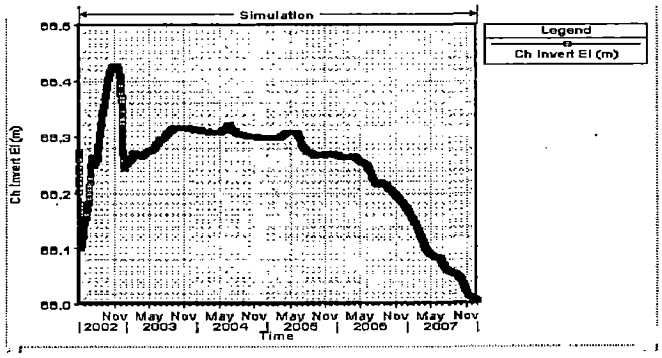

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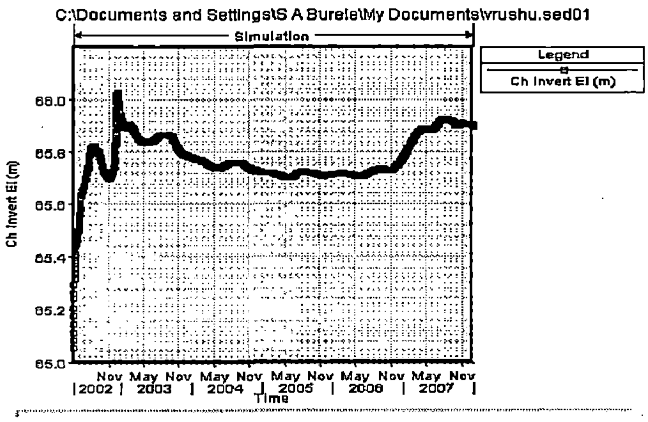

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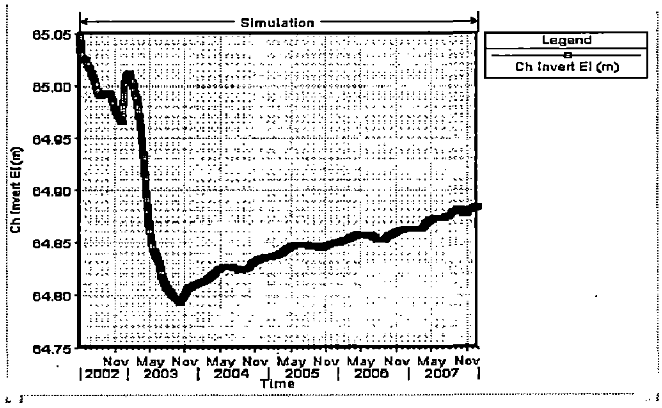

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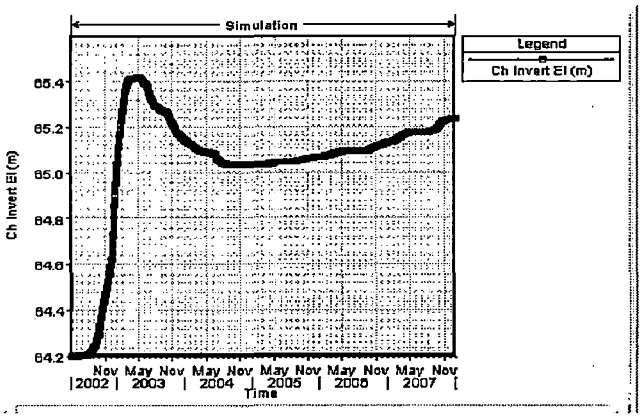

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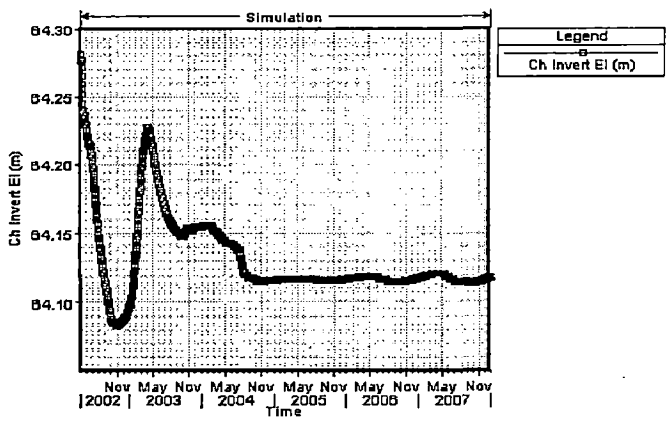

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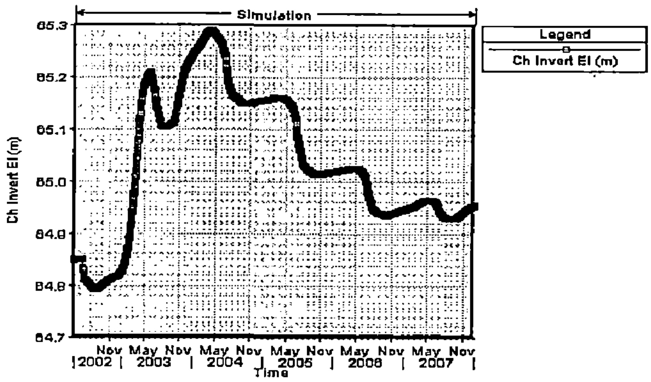

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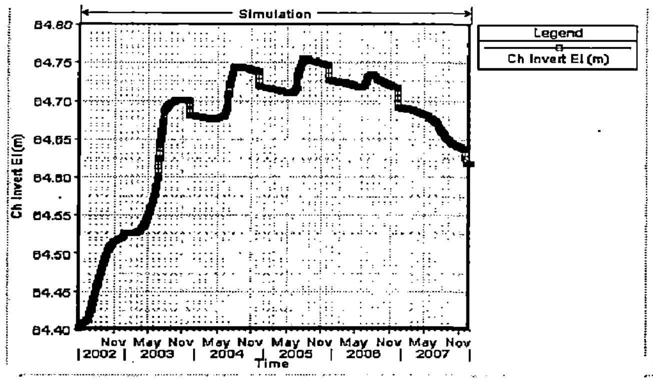

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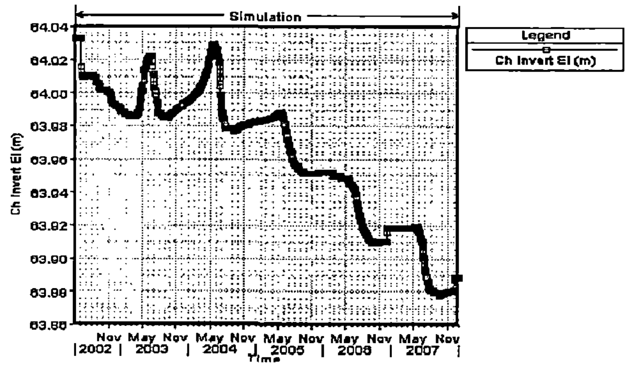

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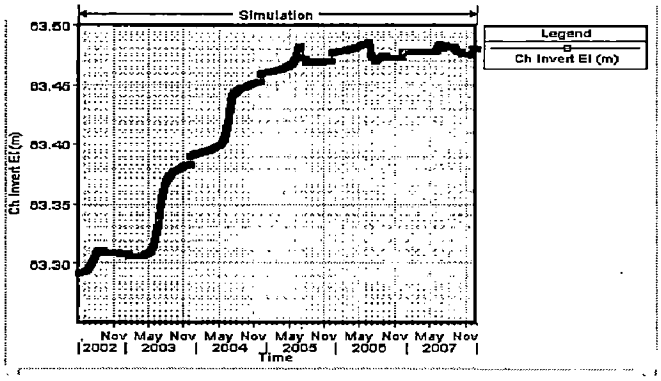

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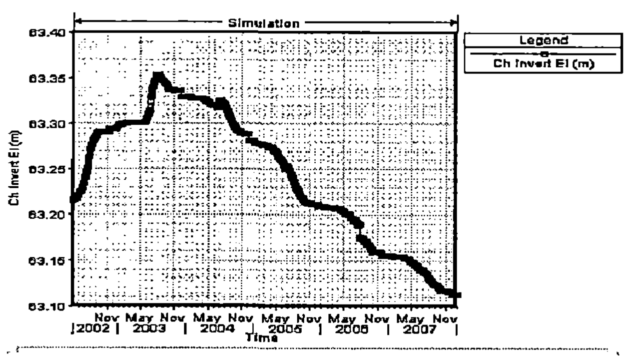

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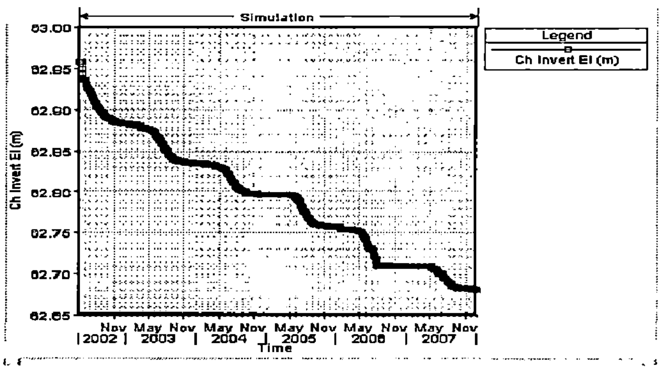

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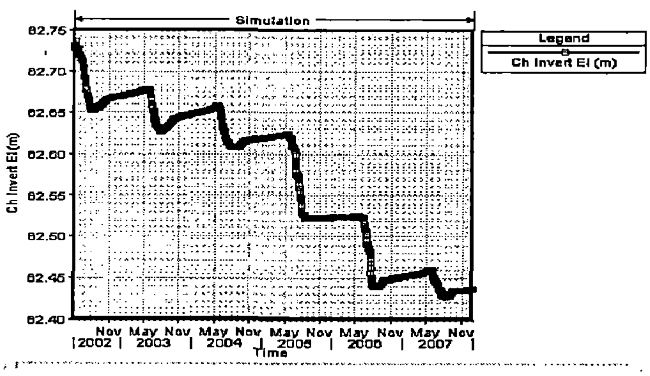

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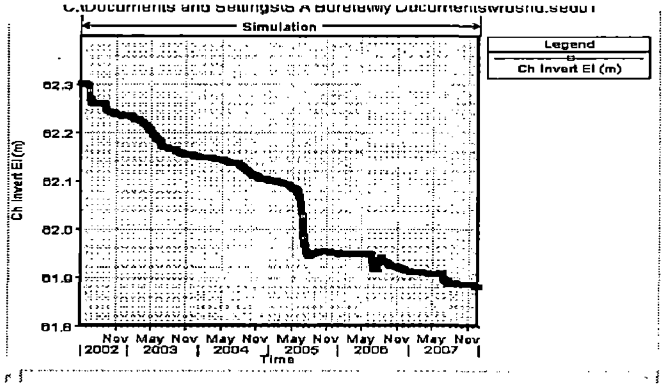

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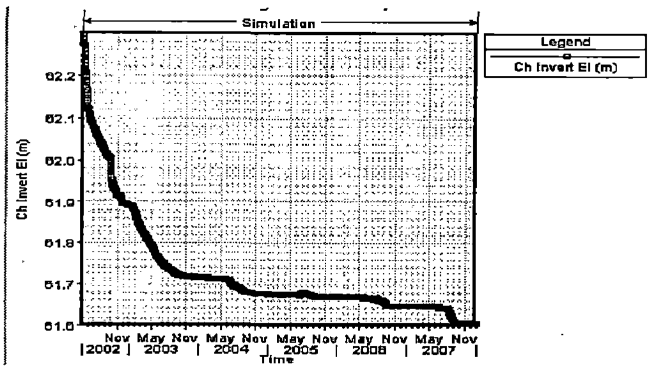

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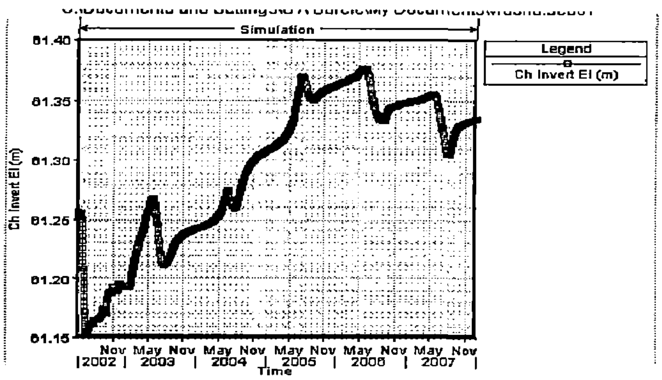

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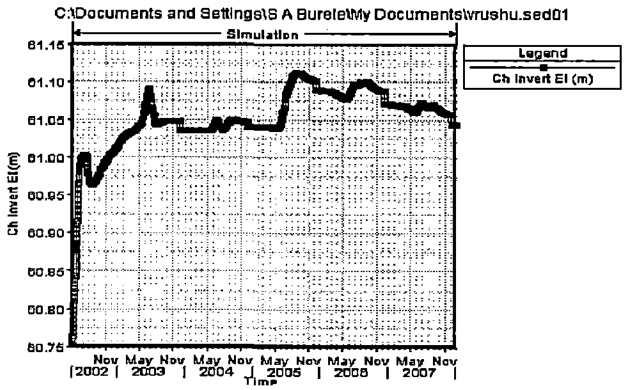

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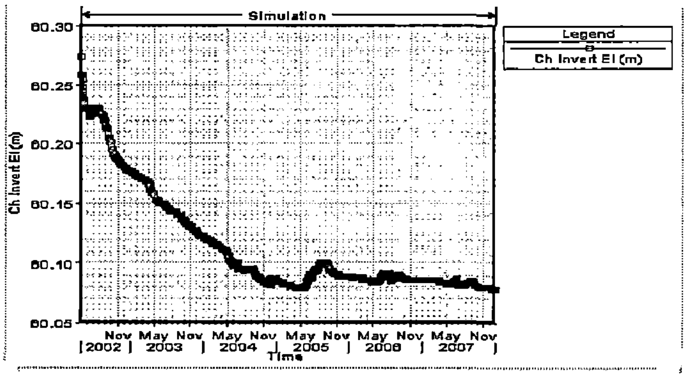

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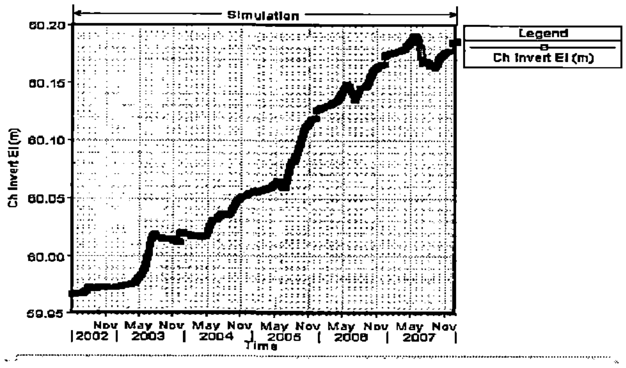

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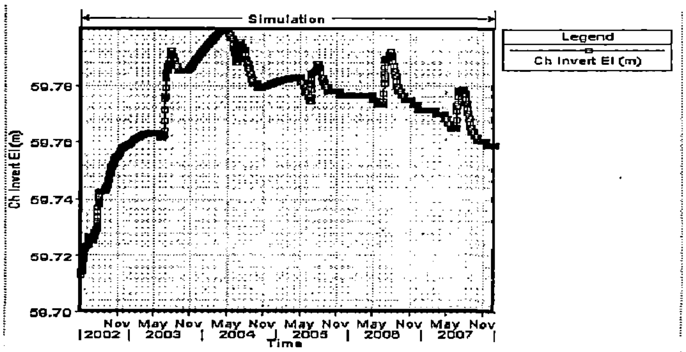

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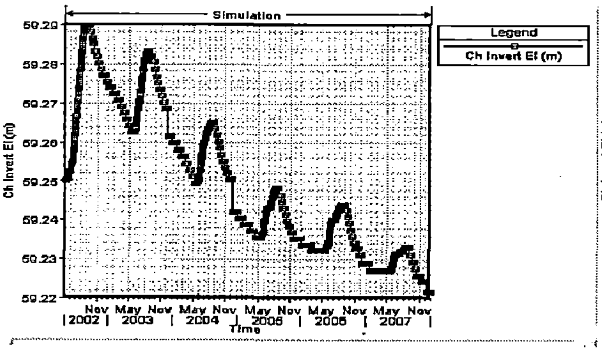

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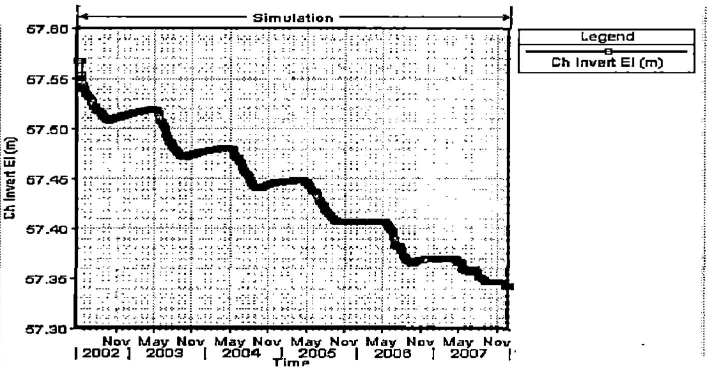

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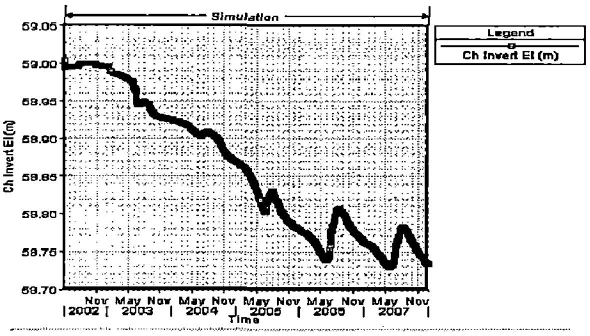

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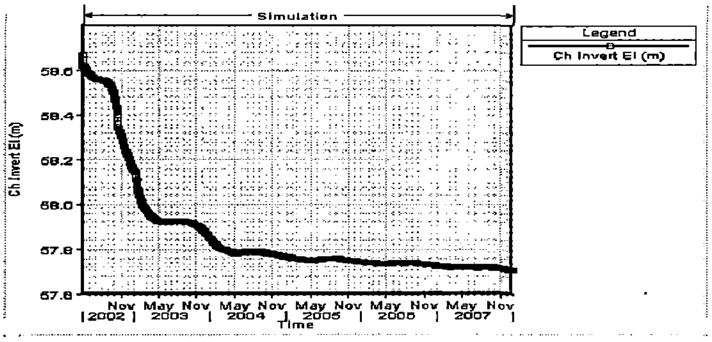

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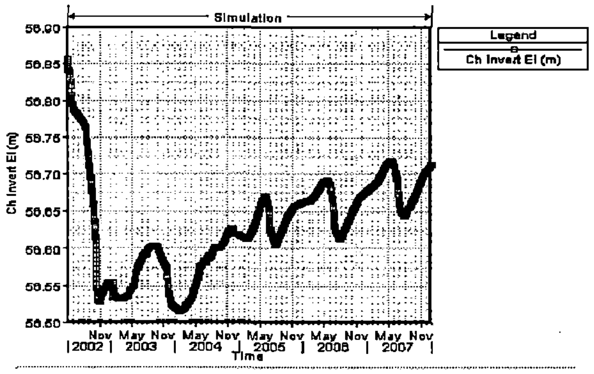

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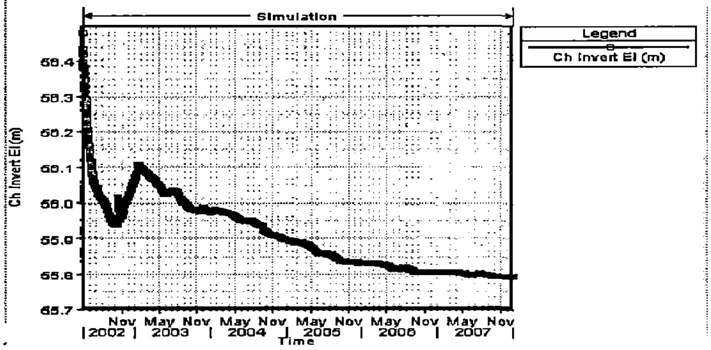

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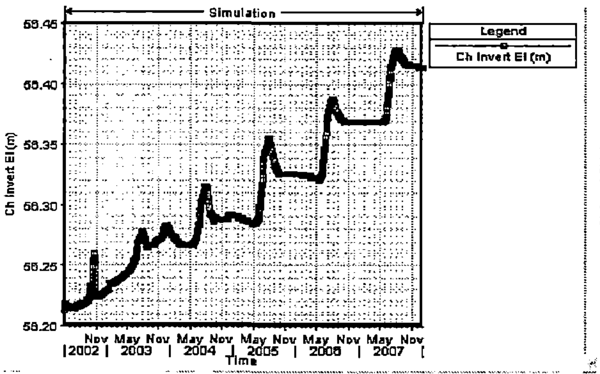

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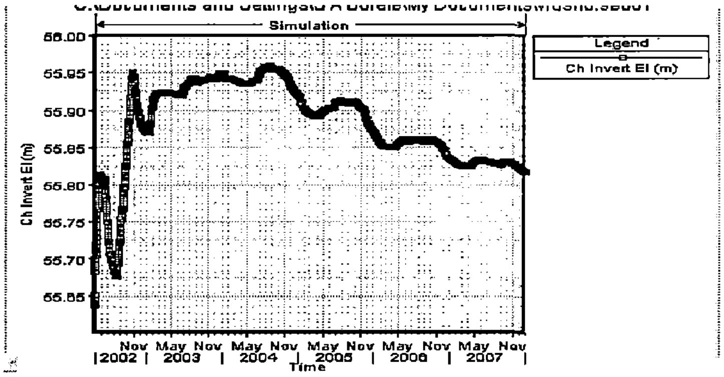
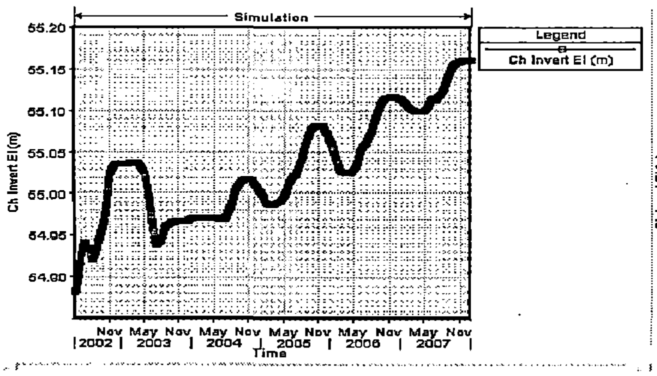


C/S-34



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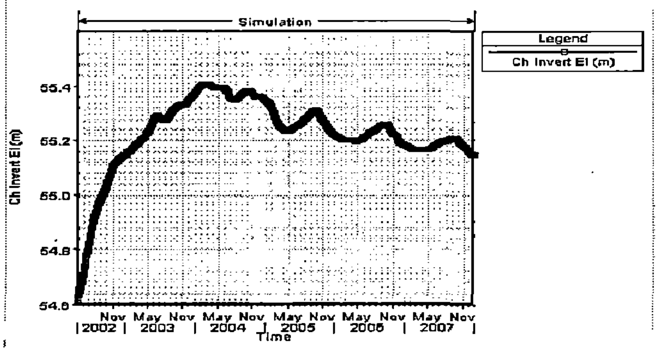

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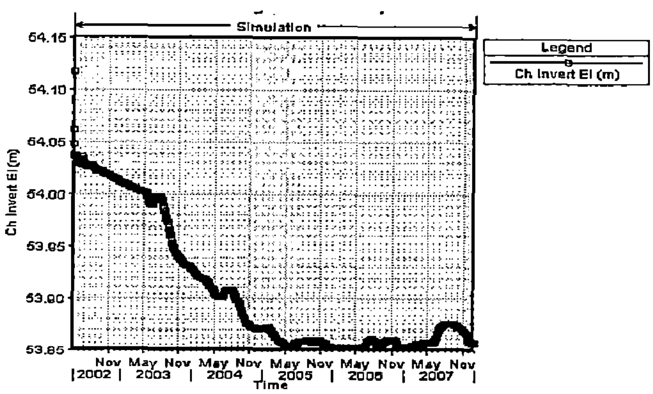

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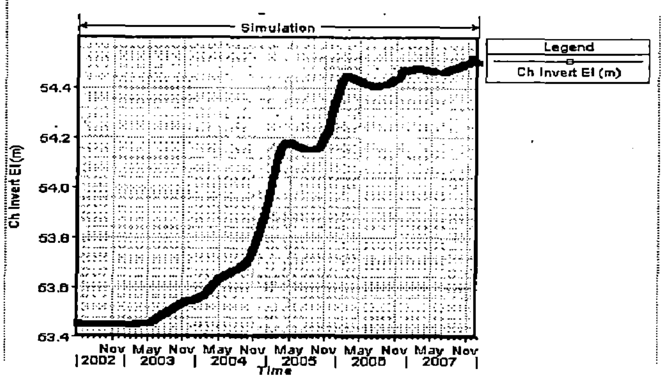

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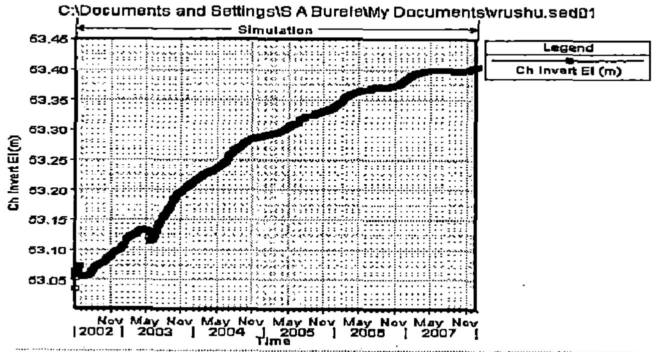

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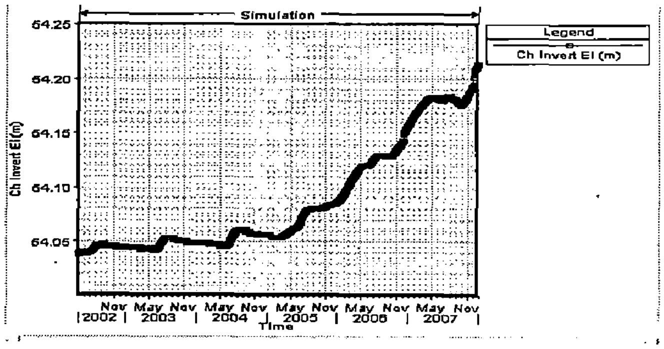

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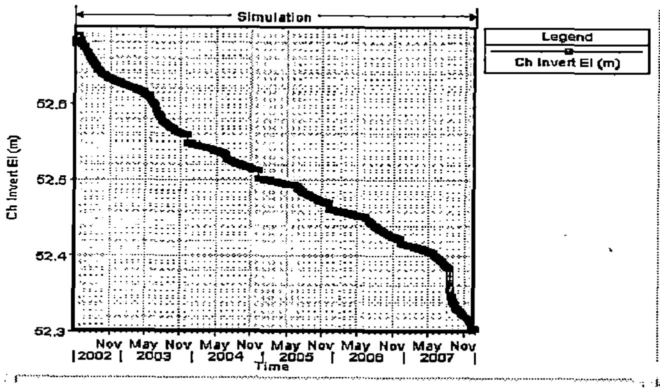

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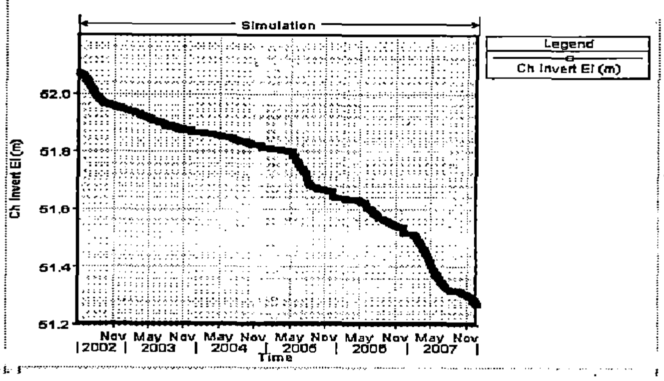
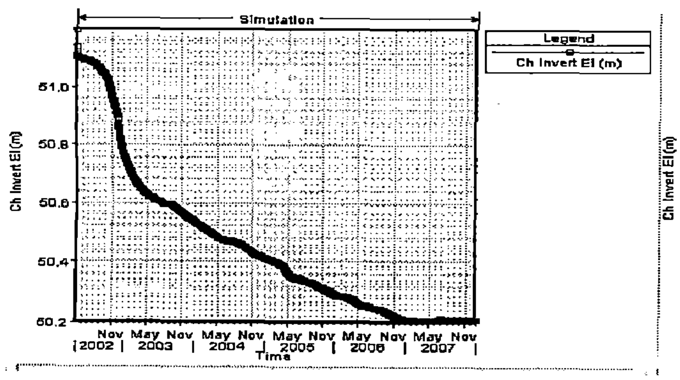
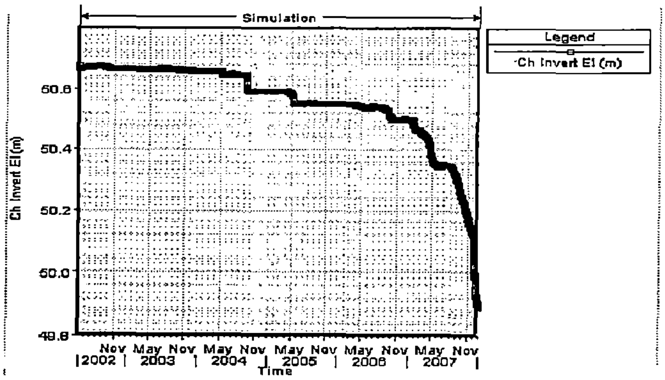


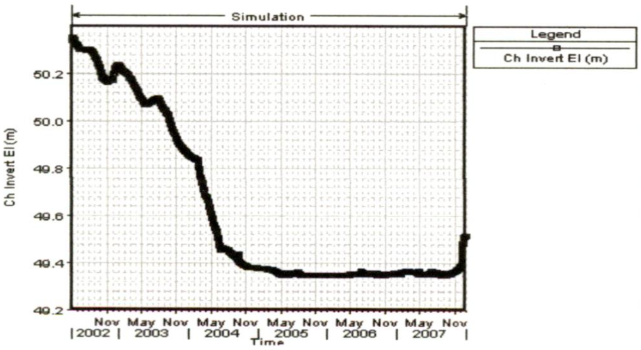
C/S-42



C/S-43



C/S-44



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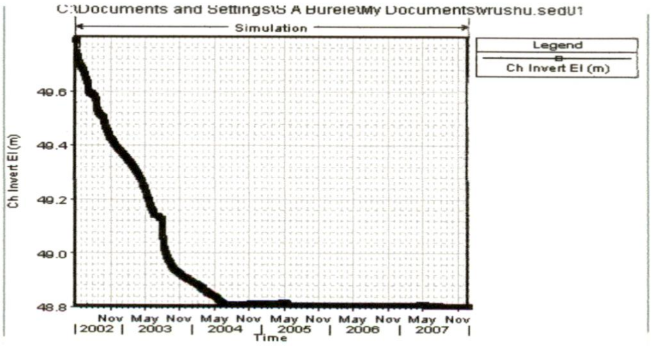
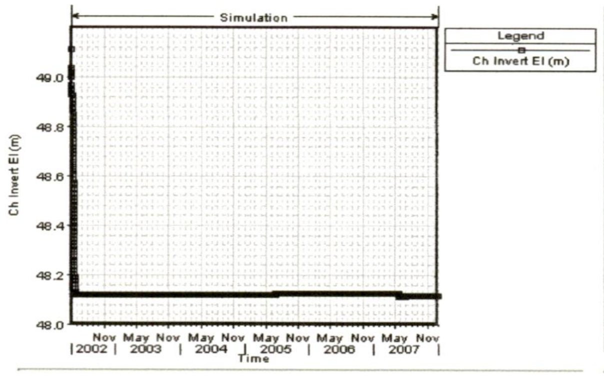
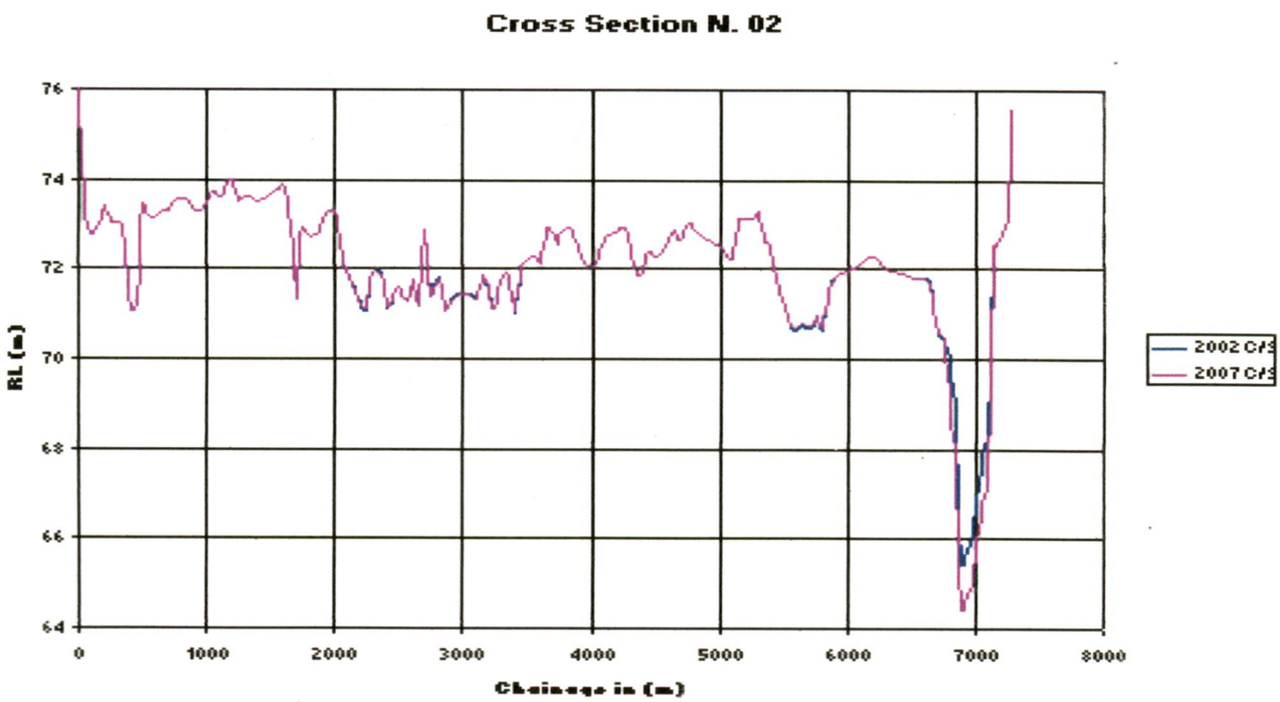


C/S-46

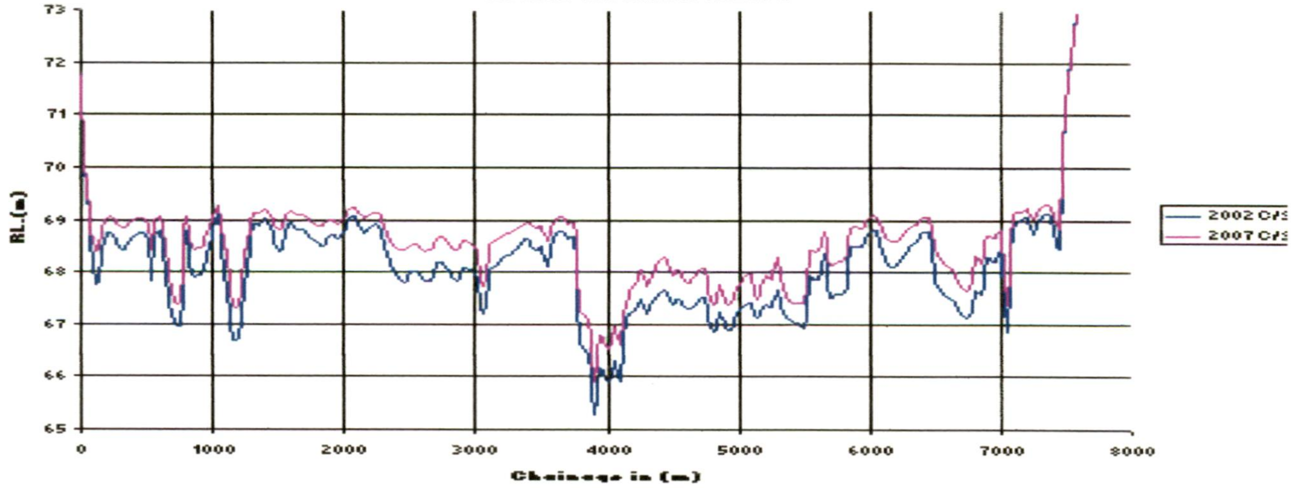


C/S-47

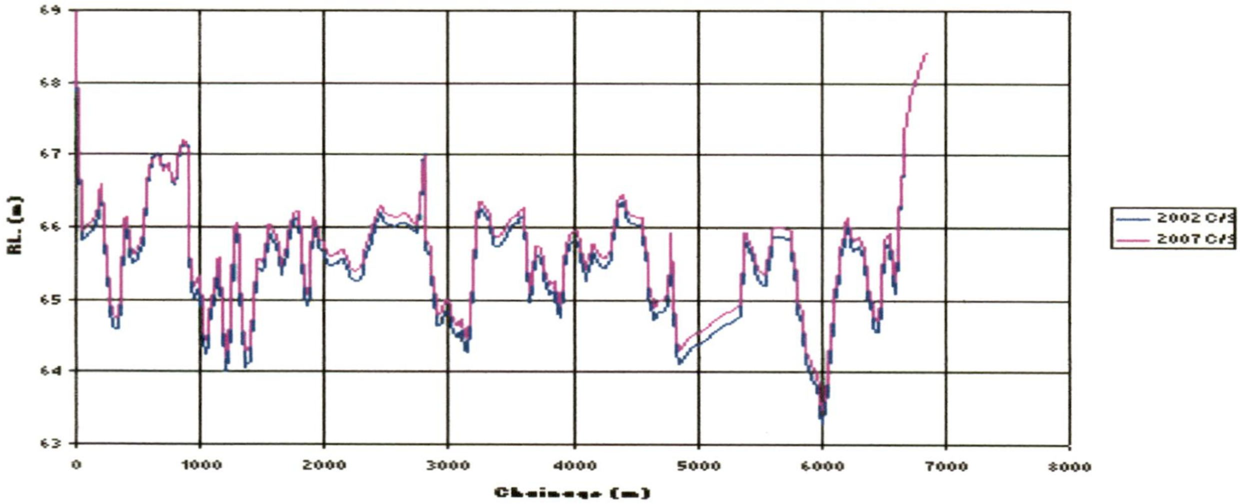
5.5 Change in bed level along cross section (Cross Section No.02, 09,16,22,29 39,42)



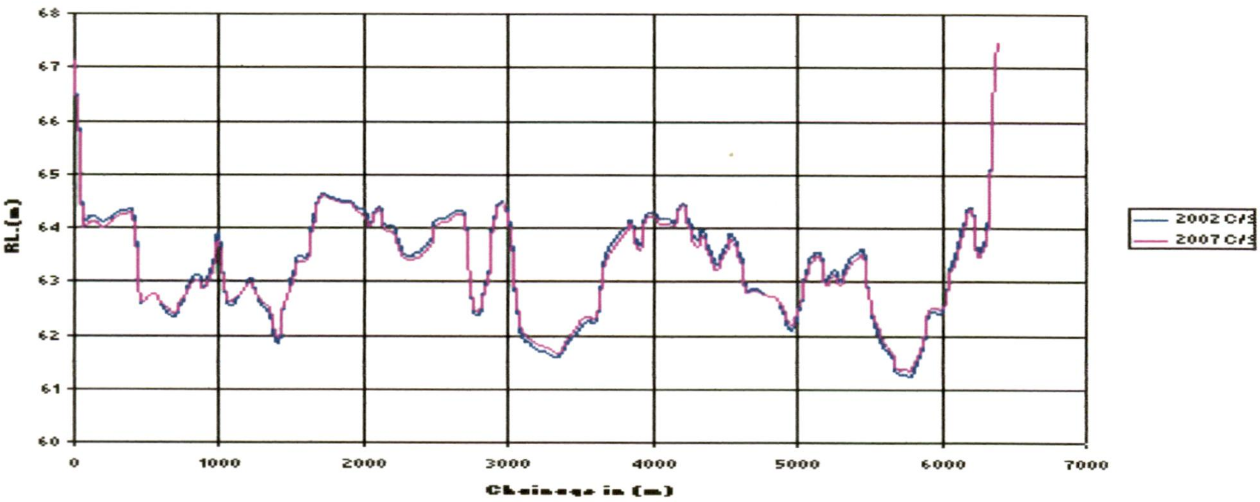
Cross Section No.09



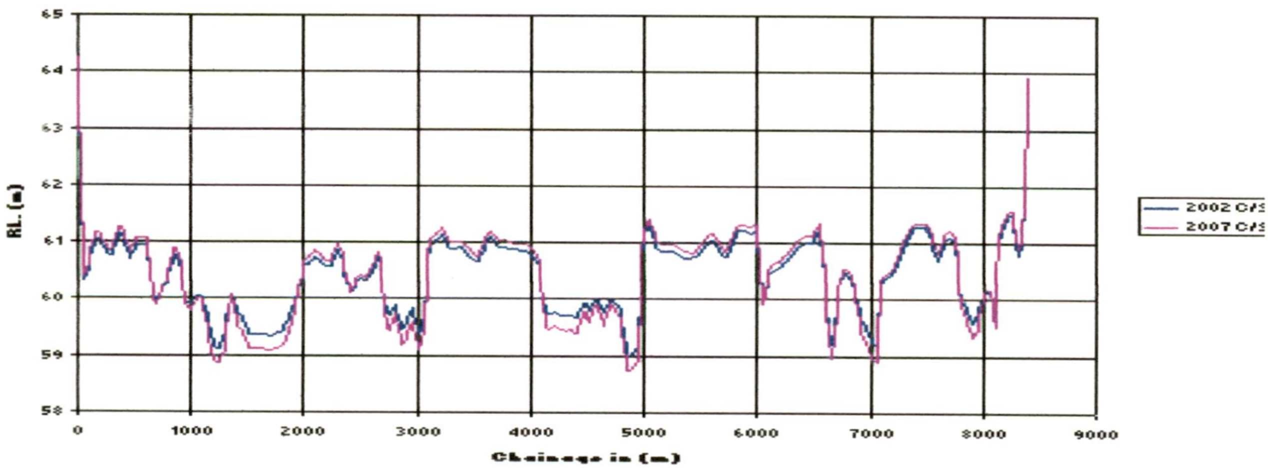
Cross Section No.16



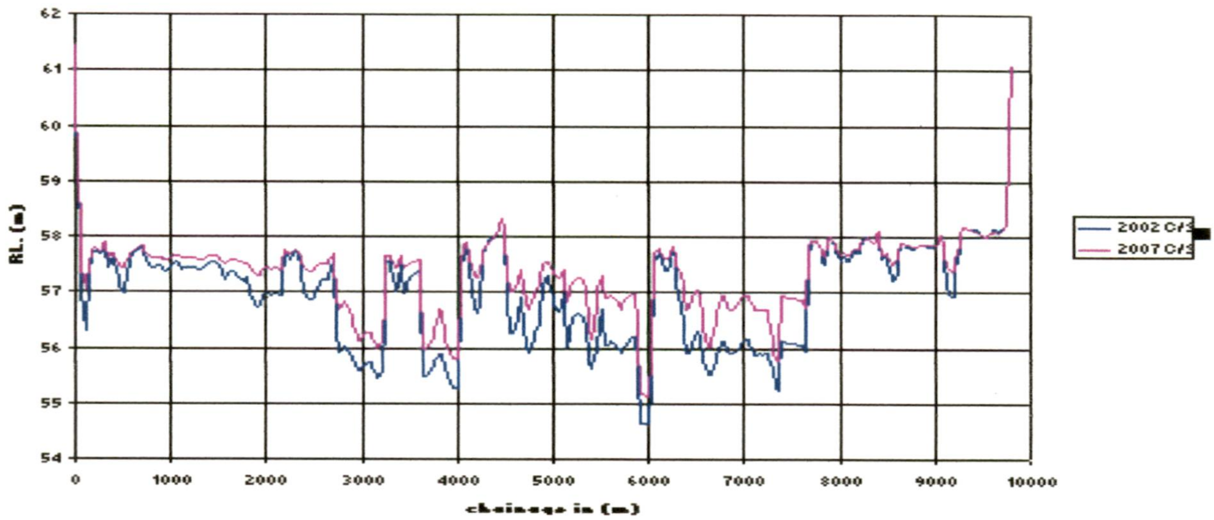
Cross Section No. 22



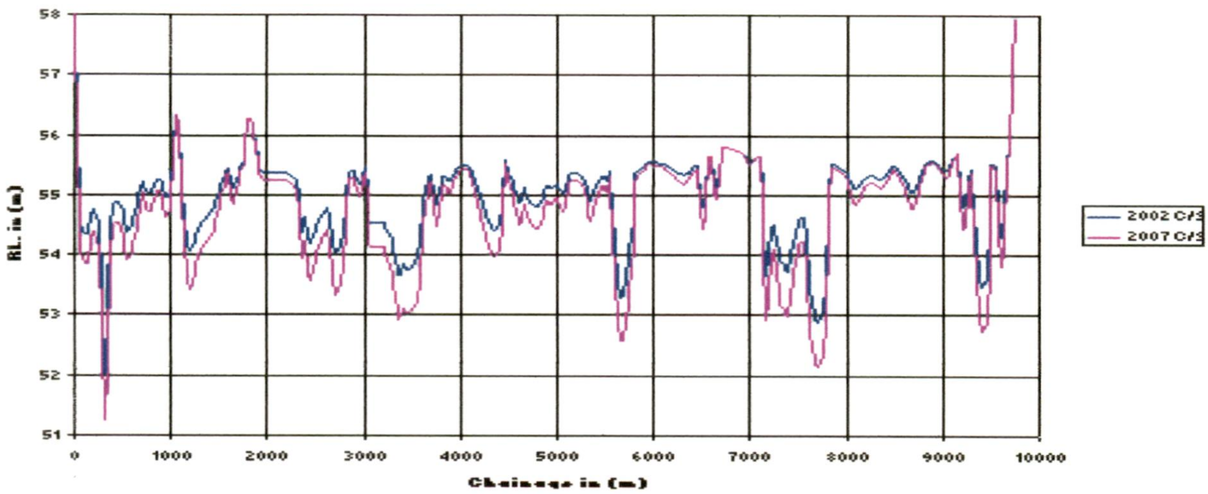
Cross Section 29



Cross Secton No. 39



Cross Section No.42



CONCLUSION

The study has been carried out using a well known mathematical model, viz.HEC-RAS 4. For discharge of 10960 cum/s, at Kosi barrage, the corresponding water level recorded in May 2002 at Dagmara & Bhaptiahi are fairly matching with mathematical model water levels for n = 0.022.

During survey work, which was done by WAPCOS in the month of April/May 2002 for the discharge of 2000 cu.m/s. water level was also recorded. For these model runs, discharge of 2000 cu.m/s was given as upstream boundary condition water level at cross section no.47 as per WAPCOS Survey was given as downstream boundary condition. Figure 4.4 shows that the predicted and observed water levels are fairly matching.

10 daily average inflows were established on the basis of daily discharge data available for the year 1948 to 1966 and recent data from 2001 to 2003. The maximum discharge data per year was available and the daily discharge data available from the year 1948 to 1966 was used to generate the discharge data from year 2003 to 2008

Using discharge sediment relationship for the period of June 2001 to April 2003,cross section for the year May 2002, generated discharge, Laursen-Copeland transport Law, Manning's n value of 0.022 in HEC-RAS-4 Mathematical model is seen satisfactorily reproduce the observed bed level of the Kosi river.

From figure15 to 20 shows aggradation/degradation . Similarly 5.5 para's. figure shows typical degradation and aggradation at cross section no. 02, 09,16,22,29 39,42.

From figure 5.3a to 5.3f longitudinal bed profile between year 2002 to 2007shows that the sediment are shifting towards downstream side

SCOPE OF FUTURE STUDIES

- In this study 'n' value is considered as 0.022, which is constant throughout the reach. Relationship should be developed between various discharges and Manning's 'n', by considering bed form regimes.
- In HEC-RAS 4, stream banks are taken as rigid. So improvements in modeling approach are required to incorporate erodible bank behavior.
- Improved techniques should be developed to simulate effect of hydraulic transients on mobile bed condition.
- Improved model proving study should be evolved for sediment laden flow condition for better reproduction of prototype fluvial behavior.
- There is need to develop better mathematical modeling techniques to appropriately account for turbulence and secondary flow behavior for realistic simulation of fluvial forces impacting on stream bed and bank changes.

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