

STOCHASTIC EROSION IN THE COMPOSITE BANKS OF ALLUVIAL RIVER BENDS

***A thesis
Submitted for the award of the degree
of***

Doctor of Philosophy

in

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By

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CERTIFICATE

This is to certify that the thesis entitled **“Stochastic Erosion in The Composite Banks of Alluvial River Bends”** submitted by Tapas Karmaker, Roll No. 07610402 to the Indian Institute of Technology Guwahati, for the award of the degree of Doctor of Philosophy in Civil Engineering is a record of bonafide research work carried out by him under my supervision and guidance. The thesis work, in my opinion, has reached the requisite standard fulfilling the requirement for the degree of Doctor of Philosophy.

The results contained in this thesis have not been submitted in part or full to any other University or Institute for award of any degree or diploma.

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STATEMENT OF ORIGINALITY

I do hereby declare that the matter embodied in this thesis entitled “**Stochastic Erosion in The Composite Banks of Alluvial River Bends**” is the result of original investigations carried out by me in the Department of Civil Engineering, Indian Institute of Technology Guwahati, Assam, India, under the supervision and guidance of Dr. Subashisa Dutta.

In keeping with the general practice of reporting scientific observations, due acknowledgements have been made wherever the work described is based on the findings of other investigators. The results contained in this thesis have not been submitted in part or full to any other University or Institute for award of any degree or diploma.

September, 2010

(Tapas Karmaker)



*Dedicated
to
The Supreme Lord*

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*IIT Guwahati
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ABSTRACT

The present research study is focused to find out the dominated bank erosion processes prevailing in the braided Brahmaputra river by conducting *in-situ* hydrodynamic and morphological observations. Better understanding of these bank erosion mechanisms is essential for designing a cost-effective bank protection system(s), which is one of major challenging tasks for river engineers.

River bank erosion occurs mainly due to three processes: fluvial entrainment, sub-aerial erosion and mass failure due to poor strength of the bank materials. One or the combination of these processes is responsible for the bank erosion. The nature of the erosion for a cohesive river bank is different from cohesionless (sandy) river bank. In a cohesive river bank, the erosion takes place as crumbs of soil rather than individual particles, as they are bound tightly by the electromechanical cohesive forces. The erosion of composite river banks is even more complex. Banks of the alluvial river reaches are mostly composed of stratified soil layers with varying grain sizes.

A lot of uncertainty in defining these causative processes and their controlling variables prevails in *in-situ* bank erosion. There are uncertainties in the river flow, sediment transport, soil erodibility and bank stratifications. Considering the importance of erosion prediction and its spatial extent, a detailed study on stochastic bank erosion modeling is required for the composite banks in alluvial river bends.

The middle reach of the Brahmaputra river of about 500 km long is considered for the stochastic model evaluation. However, a detailed field data collection on hydrodynamic and morphological characteristics in a river bend located at the upper part of the reach was focused to evaluate the model bank erosion prediction in details. The daily river stage records for nearly 20 years were used to develop the synthetic stage hydrograph generation. 58 *in situ* submerged jet tests were conducted at various sites along the river bank of Brahmaputra for determining the erodibility parameters of the fine bank soils. The result indicates that the fine soils of the Brahmaputra river bank fall in the very erodible to erodible class. *In situ* seepage erosion measurements were also conducted for model validation. At the river bend, hydrographic survey was conducted using acoustic doppler current profiler, hourly water level was recorded, and repeated bank line survey using differential global positioning system was carried out. Annual average bank erosions at the 45 river bends at the middle reach of the Brahmaputra river were measured from satellite imageries.

A new procedure to generate a synthetic seasonal hydrograph for a large river basin is proposed and evaluated in this research. The typical stage hydrograph of such a river has two components: flood waves and seasonal (monsoonal) response. The characteristics of the flood hydrographs are dependent on several variables e.g., time to

peak, response time period, time of occurrence of the flood waves, etc. The monsoonal response and the individual flood waves are approximated using Maxwell distribution. The Chi-square tests show that, annual maximum flood wave lift closely follows the lognormal distribution for both the gauging stations. Based on the best distribution function and relation to total seasonal rainfall, flood characteristics and monsoonal response are generated. The generated stage hydrographs were evaluated at two gauging stations of the Brahmaputra river for different flood wave return periods.

The mathematical relationship functions for seepage erosion induced mass failure with seepage gradient, bank height, bank slope and various arrangements of semipermeable layers have been investigated through lysimeter experiments. Moreover, a mathematical seepage erosion model by using these functions and river stage records has been developed to estimate the daily seepage erosion of the banks. Analysis shows that as the number of semipermeable layer increases, the time of collapse increases significantly.

A depth integrated hydrodynamic and morphological model is formulated for a river bend. In the bank erosion processes, near bank erosion is estimated from the excess shear stress concept. Three modes of cantilever mass failures are considered: shear failure, beam failure and tensile failure. Predicting river bed degradation and the morphological changes is based on the mass conservation of the sediment in a near bank control volume, including sediment from bank mass failure, sediment stored near the bed and sediment flux transported in and out from the control volume. The total bank erosion is computed as the sum of fluvial erosion and seepage erosion. The model prediction has been validated in detail for the channel bend of the Brahmaputra river at Jamuguri for the 4 monsoon seasons. Results indicate that the model prediction falls within the acceptable limit.

Considering the probabilistic distribution of bank soil erodibility and flood waves return periods, the validated model has been simulated to estimate stochastic bank erosion rate. In these simulation studies, variations in bed material size, geometry of the bend and monsoon condition were considered. The simulated results indicate the magnitude of the bank erosion increases from the dry monsoon to wet monsoon years. For high critical shear stress of the bank material, seepage erosion dominates over the fluvial erosion. Moreover, from the satellite imagery analysis of the 45 channel bends, the cumulative probability of the annual average bank erosion has been measured. From the simulation results, the range of the cumulative probability of annual bank erosion rate is estimated. The comparison between the two indicates that the developed analytical bank erosion model is able to predict the range of annual bank erosion rates in the river bends.

This bank erosion model is useful in predicting the vulnerable reaches of a river and can be used as a part of a river management tool for morphological and ecological change assessments.

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Nomenclature

a	exponent
a_1	coefficient of proportionality
A	scour factor
A_s	momentum redistribution factor due to secondary current
A_1	value for the semi-transverse and semi-conjugate axis of the hyperbola
b	half width of the channel [L]
b_1	width of the overhang bank [L]
B	exponent
B'	calibration constant
C	curvature of the channel centerline [L^{-1}]
C_d	jet diffusion constant
C_f	coefficient of friction
C_L'	coefficient of lift force
C_s	suspended sediment concentration [$M L^{-3}$]
C_2''	empirical constant
C_*	equilibrium suspended sediment concentration [$M L^{-3}$]
d	depth of flow [L]
d_{cr}	diameter of particle whose motion is impending [L]
d_0	jet nozzle diameter [L]
D	average flow depth [L]
D_{50}	median particle size [L]
D_g	effective diameter of the bed materials [L]

D_r	dispersion ratio
E	erosion rate [$L T^{-3}$]
f	friction factor
F_d	particle densimetric Froude number
F_D	drag force of the fluid
F_s	seepage force [$M L^2 T^{-1}$]
F_{ss}, F_{sb}, F_{st}	factor of safety against shear, beam and tensile failures, respectively
g	gravitational acceleration [$L^2 T^{-1}$]
h_b	initial river stage [L]
h_f	maximum lift of the individual flood wave [L]
h_f^m	annual maximum flood lift [L]
h_s	seasonal lift of monsoonal response [L]
H'	overhang height of the bank [L]
I	hydraulic head gradient
I_w	plasticity index
J	distance from the nozzle to the centerline depth of scour [L]
J_e	equilibrium scour depth [L]
J_i	initial jet orifice height [L]
J_p	potential core length [L]
J^*	dimensionless scour term
k_d	erodibility coefficient
k_s, k_f^i	fitting parameter for seasonal and i th flood wave responses, respectively
l	length of the lower desiccation crack [L]

m	length of the upper crack [L]
m'	reciprocal of the velocity exponent for grain roughness
n	porosity
nun	nunner's number, equals to the inverse of the square root of friction factor
P_c	clay fraction by weight [%]
Q	discharge [$L^3 T^{-1}$]
q_n	sediment transport rate component in transverse direction [$M L^{-2} T^{-1}$]
q_s	sediment transport rate component in longitudinal direction [$M L^{-2} T^{-1}$]
q_s^*	dimensionless sediment discharge
r_s, r_f^i	fitting parameter for seasonal and i th flood wave responses, respectively
RB	relative backscatter [dB]
R_c	radius of curvature [L]
S	longitudinal slope of energy gradient line
S'	storativity of soil mass
SC	percentage of silt and clay [%]
S_g	specific gravity of soil
S_t	transverse bed slope
t	time [T]
T	return period of flood wave [T]
T'	transmissivity of soil mass
T_r	reference time equivalent
T^*	ratio measured time to the reference time
u	depth integrated velocity components in the stream-wise direction [$L T^{-1}$]
U	depth averaged longitudinal velocity [$L T^{-1}$]

U_0	velocity of the jet at the origin [L T ⁻¹]
v	depth integrated velocity components in the transverse direction [L T ⁻¹]
V_{mc}	mean velocity component for first displacement of bed material [L]
w_c	average width of bank mass collapsed [L]
α	projected area-volume ratio of the particle [L ⁻¹]
$\bar{\beta}$	average bank slope
η	bed elevation from datum [L]
ε_t	daily seepage bank erosion [L]
γ	unit weight of water [M L ⁻¹ T ⁻¹]
γ'_s	submerged unit weight of particle [M L ⁻¹ T ⁻¹]
λ	bed layer porosity
μ	coefficient of the kinetic friction between the particle
ρ	mass density of the water [M L ⁻³]
ρ_s	mass density of the particle [M L ⁻³]
t_b	time to bank collapse [T]
τ_a	developed shear stress at the soil boundary [M T ⁻¹]
τ_{bc}, τ_{b0}	critical and developed shear stress of the soils at bank toe [M T ⁻¹]
τ_c	critical shear stress [M T ⁻¹]
τ_i	initial peak boundary stress prior to scour [M T ⁻¹]
τ_{*c}	critical Shield's stress [M T ⁻¹]
τ_0	maximum shear stress due to jet velocity at the nozzle [M T ⁻¹]
τ^*	dimensionless shear stress

θ	deflection angle between the downstream and stream-wise direction
φ	angle between the bed shear stress and the longitudinal direction
ω	speed of flood wave [$L T^{-1}$]
ω'	total fluvial erosion [L]
ξ	water surface elevation from datum [L]
$\bar{\xi}$	depth averaged erosion rate of the deposited material at toe [$L T^{-1}$]



1.1 Overview

High river bank erosion in the morphologically dynamic alluvial rivers has been of great concern, causing serious problems to the habitats and others like loss of fertile land, valuables and life (Muramoto et al., 1992; Mosselman et al., 1995; and Tingsanchali et al., 1997). Better understanding of these mechanisms is primarily essential for designing a cost-effective river bank protection system(s). The bank erosion rate depends on several factors such as flow characteristics of the river, near bank hydraulics, composition and characteristics of the bank and bed materials, sediment load near the river bank and variation of the bed level at the toe of the bank etc. Besides this, at a river bend, the formation of secondary (helical) current can also accelerate the erosion rate significantly. Being associated with so many controlling variables with uncertainty in their measurement, hydrodynamic and morphodynamic processes involved in the bank erosion are always difficult to be completely understood/modelled in a large braided river (Karmaker and Dutta, 2009).

River bank erosion occurs mainly due to three processes: fluvial entrainment, sub-aerial erosion and mass failure due to poor strength of the bank materials (Lawler, 1995). One or the combination of these processes is responsible for the river bank erosion. Normally it is reported that, mass failure is dominated in the upper reach of the river, fluvial erosion dominated in the middle reach, while the sub-aerial weathering and weakening process dominates in the lower reaches. In this present study, the bank erosions due to fluvial entrainment and mass failure is addressed. Nevertheless the bank erosion due to sub-aerial process is dominated in many places and times (Couper and Maddock, 2001).

The erosion in a homogeneous river bank is quite simple and mainly dominated by a single process. Moreover, the nature of the erosion for a cohesive river bank is completely different from cohesionless river bank. In a cohesive river bank, the erosion takes place as crumbs of soil rather than individual particles, as they are bound tightly by the electromechanical cohesive forces (Lawler et al., 1997). The rotational type of bank failure is often reported for this case of river banks (Fig. 1.1). On the other hand the erosion in cohesionless river bank is due to loss of individual particle. For these non-cohesion river banks, planer failure is the main mechanism of bank erosion (Fig. 1.2). The river bank erosion process is a cyclic one, composed of toe erosion and under cut, failure of unstable bank mass and deposition at the toe and complete removal of the deposited material due to sediment transport. Moreover, the dense vegetation cover on the top soil has been found to reduce the effectiveness of fluvial erosion up to two orders of magnitude (Kirkby and Morgan, 1980).

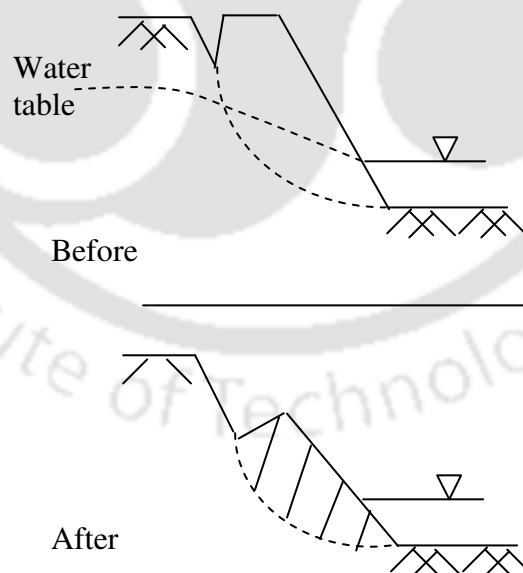


Fig. 1.1. Rotational failure in cohesive riverbanks (source: Hey et al., 1991)

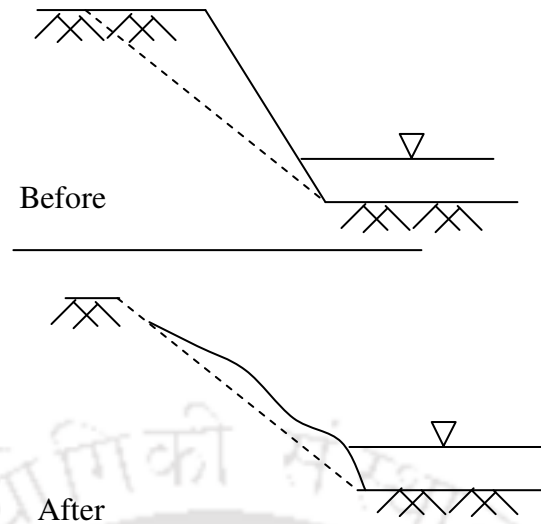


Fig. 1.2. Planar failure in non-cohesive riverbanks (source: Hey et al., 1991)

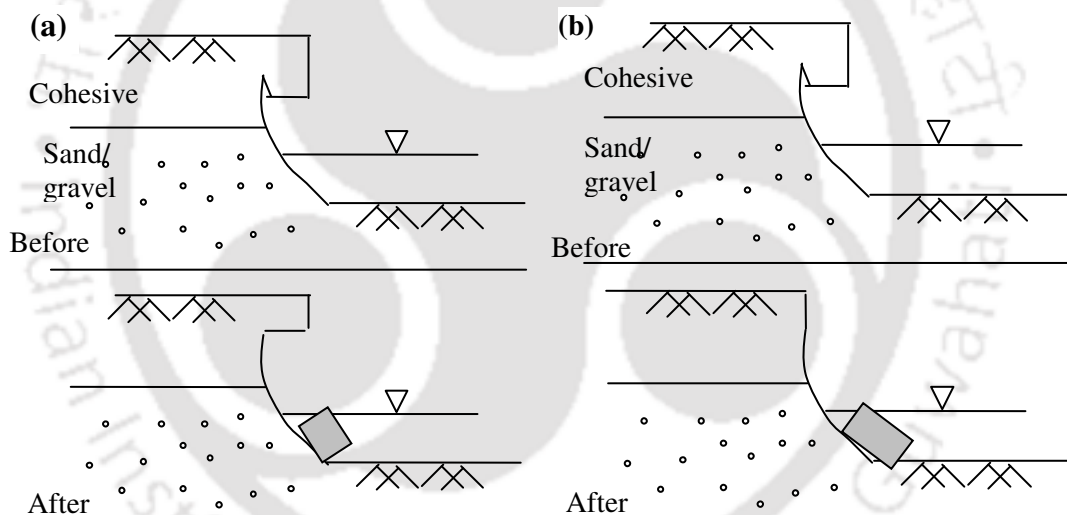


Fig. 1.3. Failures in a composite riverbank: (a) tension failure and (b) beam failure. (source: Hey et al., 1991)

The erosion in case of composite river bank is even more complex than the homogeneous one. River banks of the alluvial river reaches are mostly composed of stratified soil layers, grain sizes of which vary from fine to coarse one. Normally, the alluvial river bed and the lower layers of composite banks are composed of cohesionless

materials and the top layer of the bank is composed of clay soil. The main causes of the composite bank erosion are:

- a) Excess shear stress developed at the toe causing toe scour.
- b) High variation in the stage level changes the pore water pressure of the bank materials and due to excess pore water pressure bank materials collapse.
- c) During the recession phase of the hydrograph, seepage occurs through the silty and/or sandy soils. Due to high exit seepage gradient undercut develops and subsequently collapse of the upper part of the bank occurs due to tension or shear failure (Lawler et al., 1997).

Due to the excess shear stress developed at the toe of the bank, the bank erosion at the toe takes place; the eroded material subsequently removed by the flowing water having enough sediment transport capacity. Thus the top bank mass remains as a cantilever one. The collapse of this cantilever takes place either by beam, shear or tensile failure (Fig. 1.3).

Duan (2005) suggested that bank failure is actually a probabilistic phenomenon and its occurrences depend on the natural events that trigger bank erosion. Moreover, in a river bend, there exists secondary current. The existence of secondary current redistributes the shear stress and the flow pattern (Figs. 1.4 and 1.5). Secondary flow grows upon entering a river bend. In a sufficient long channel bend the secondary flow comes to equilibrium, thus the flow does not vary from one cross section to another.

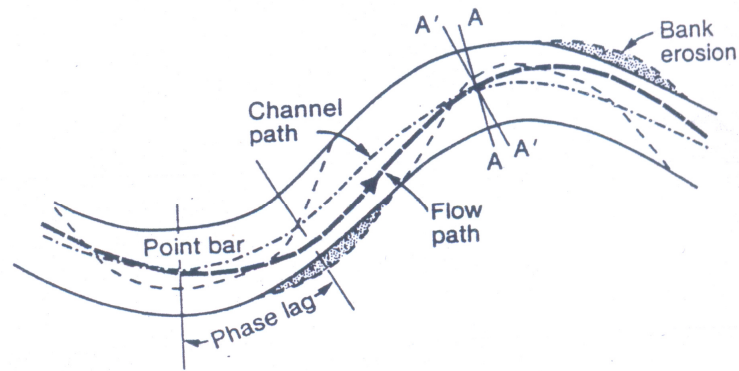


Fig. 1.4. Centerline and the streamline for flow in curved channel (source: Chang, 1988)

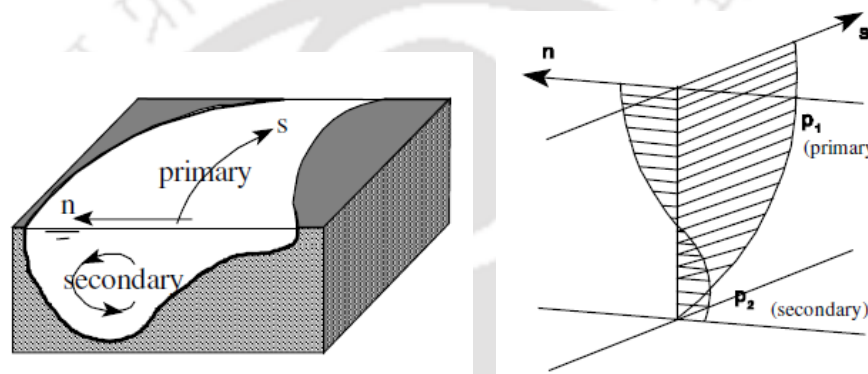


Fig. 1.5. Secondary flow in a curved channel (source: User manual MIKE 21C)

In this study attempts have been made to develop a stochastic bank erosion model for composite river banks to predict the probable bank erosion for Brahmaputra river.

1.2 Objectives of the present research

The main objectives of the present research are:

- To understand the bank erosion processes in a composite river bank at a selected channel bend of the Brahmaputra river and conduct *in situ* observations for detail hydrographic data with ADCP and echo-sounder aided with GPS; bank erosion data through repeated bankline survey with modern survey instruments like DGPS; and river stages records with automatic water level recorder.

- To characterize the seasonal (monsoonal) hydrograph for a large river basin based on the historical flow time series and then generate synthetic seasonal hydrographs of various flood wave return periods and basin-average seasonal rainfall.
- To conduct *in situ* observations on soil erodibility, its dependency on sites and soil layers and establish soil erodibility relationship of the bank of Brahmaputra river.
- To investigate the seepage erosion processes for a composite river bank under stage variation and bank stratification, and establish a functional relation for time of bank collapse with river stage.
- To developed a bank erosion model addressing the hydrodynamic, morphologic and bank erosion processes for predicting bank erosion rate at a river bend with composite bank.
- Formulate and compute stochastic seasonal bank erosion rate of river bends under flood waves of different return periods, monsoonal conditions and uncertainty in bank soil erodibility and bed material.
- To estimate the annual bank erosion rate from satellite imagery of major channel bends at the middle reach of the Brahmaputra river of about 500 km and validate the developed stochastic model results with these observation ranges.

1.3 Organization of the thesis

The thesis of the present study contains nine chapters. Chapter 1 contains a brief overview of the bank erosion problem in composite river bank and the specific objectives considered for the present study. Chapter 2 focuses towards understanding the relevant literatures to address the modeling concepts of bank erosion problems. Chapter 3 describes the details of the study area and the basic data collected from the sites. Chapter 4 addresses the procedure for generation of synthetic seasonal hydrograph for a large

river basin. The mathematical formulation of the synthetic hydrograph based on the random and deterministic hydrological parameters is addressed in this chapter. Chapter 5 explains the field experiments for determining the erodibility parameters of the fine soils in the composite riverbanks. Chapter 6 deals with seepage erosion estimation based on the laboratory experiments for various combinations of bank stratifications. Chapter 7 addresses the development of a mathematical river model to predict daily bank erosion rate considering the dominated bank erosion processes in a river bend. The developed model has been validated for a channel bend with detailed morphological data in this chapter. Chapter 8 contains the stochastic bank erosion estimation for different monsoon conditions, flood waves of various return periods, longitudinal bed slopes, and bed material sizes. This chapter also deals with satellite image analysis for determining the probabilistic distribution of annual bank erosion in the major river bends of Brahmaputra river in India. Finally, general validation of the developed model for predicting annual bank erosion rate in the river bends with various combinations of the parameters is addressed in this chapter. Chapter 9 finally summarizes the major findings on stochastic bank erosion from the present investigation and highlights some of the future studies in this field.

2.1 Introduction

In the last few decades several researchers carried out fundamental studies in order to understand the bank erosion processes. These studies include the rate of river bank erosion at various near bank hydraulic and flood characteristics, determining the critical shear stress and erodibility of the river bank materials, quantifying seepage erosion rate, etc. These studies were either carried out in the laboratory through physical river models or *in situ* monitoring at the erodible river banks. Some analytical models were also developed to describe the bank erosion processes. This chapter is organized, focusing on the previous research studies carried out on different aspects of river bank erosion and its controlling variables such as seepage erosion, soil erodibility, and fluvial river bank erosion. This chapter also includes some previous studies on the generation of synthetic seasonal hydrographs.

2.2 Erodibility of Bank Materials

The erodibility of the bank soil materials is the important factor for fluvial erosion. This erodibility is directly related with the critical shear stress of soil. Critical shear stress is the shear stress developed due to flowing water over a soil surface at the initiation of the scour. Several investigations were carried out for determining the critical shear stress of different soil materials. Some important investigations and their methods are discussed below.

Shields (1936) developed a diagram based on particle size to estimate the critical shear stress of the non-cohesive soils assuming no interaction between the sediment

particles. The study was carried out in uniform non-cohesive particle for which fluvial entrainment is a function of hydraulic lift and drag forces.

Smerdon and Beasley (1961) carried out the study in laboratory flume for determining the relationship of the critical shear stress with the basic soil properties. Eleven cohesive bank soils from the Missouri river were investigated in that study. The collected soil samples were placed in the flume without compaction. The soils were tested under increasing flow rates and the shear stress corresponding to the bed failure was considered as the critical shear stress. Different empirical relationships were developed and are shown through the following equations:

$$\tau_c = 0.16 \times (I_w)^{0.84} \quad (2.1)$$

$$\tau_c = 10.2 \times (D_r)^{-0.63} \quad (2.2)$$

$$\tau_c = 3.54 \times 10^{-28.1D_{50}} \quad (2.3)$$

$$\tau_c = 0.493 \times 10^{0.0182P_c} \quad (2.4)$$

where, τ_c critical shear stress (Pa), I_w plasticity index, D_r the dispersion ratio, D_{50} mean particle size (m), P_c the percent clay by weight (%). Clark and Wynn (2007) suggested that the relations with I_w and D_r are most relevant as they directly relate with cohesive properties of soil.

Neill (1967) documented the experimental data on the incipient motion for six sizes of graded gravels, two sizes of uniform glass balls, and cellulose acetate balls, diameters of which ranges between 6 and 30 mm. for a wide channel. The flow was uniform over the flat flume bed. After analyzing the experimental data he presented an equation with mean velocity, grain size, specific gravity and flow depth:

$$\frac{\rho V_{mc}^2}{\gamma_s D_g} = 2.50 \left(\frac{D_g}{d} \right)^{-0.20} \quad (2.5)$$

where, V_{mc} the mean velocity component for first displacement of bed material, D_g the effective diameter of the bed materials, d the depth of flow, γ_s the submerged unit weight equals to $g(\rho_s - \rho)$, g the acceleration due to gravity, ρ_s the mass density of the bed materials, ρ the mass density of the flowing fluid. Later the equation was modified to calculate the critical shear stress directly (Neill, 1973; Fairfax County, 2004):

$$\tau_c = 0.76090 \gamma (S_g - 1) D_{50}^{\frac{2}{3}} d^{\frac{1}{3}} \quad (2.6)$$

where, γ is the specific weight of water (N/m^3), S_g the specific gravity of soil, and d the depth of flow (m).

As the above critical shear stress estimation is dependent on the depth of flow so the estimated critical shear stress in this case can not be considered as an exclusive soil property.

Based on the experimental results of Dunn (1959) and Vanoni (1977), Julian and Torres (2006) estimated the critical shear stress from percentage of silt and clay (SC) contents. In this equation, silt-clay contents are defined as the particles less than 0.63 mm:

$$\tau_c = 0.1 + 0.1779(SC) + 0.0028(SC)^2 - 2.34 \times 10^{-5}(SC)^3 \quad (2.7)$$

In the same study, to account for the vegetation effect, a multiplying factor (range, 1-19.20) was used to increase the critical shear stress.

Although the flume studies allow much control over the experiments, but at the same time it involves lots of issues. The major issues are the disturbed soil structures and

the degree of compaction (Hanson et al., 1999; Hanson and Cook, 2004). Alternately, *in situ* experiments for determining the critical shear stress and the erodibility are advantageous. A submerged jet test device, designed by Hanson (1990a) allows better applicability for wide ranges of soils and environmental conditions. A water jet generated through a nozzle (diameter 6.4 mm) impacts on the soil surface. After impact, it diffuses radially and thus produces shear stress on the soil surface. Recent studies (Hanson and Cook, 1997; Hanson and Simon, 2001; Wynn, 2004) using this apparatus show the ranges of the critical shear stress and erodibility coefficients for different study sites. Only limitation is that, the test is only applicable to the cohesive soils only. Maximum scour depth and critical shear stress are estimated by fitting a hyperbolic logarithmic equation to the scour data using the method described by Blaisdell model (Blaisdell et al., 1981). The soil erodibility coefficient (k_d) is determined by fitting the jet scour data to the excess stress equation using the least square method. Hanson and Simon (2001) based on their 83 experimental results showed that this erodibility coefficient is related to the critical shear stress through a power relation:

$$k_d = 0.2\tau_c^{-0.5} \quad (2.8)$$

where, k_d is the erodibility coefficient ($\text{cm}^3/\text{N}\cdot\text{s}$). While, Wynn (2004) conducted 142 experiments in vegetated river bank at 25 field sites in southwest Virginia and found that these erodibility parameters are related through the following equation:

$$k_d = 3.1\tau_c^{-0.37} \quad (2.9)$$

The erosion rate for the fine grained soils in river bed or river bank is generally assumed to be proportional to the excess shear stress and can be expressed in the form of following equation (Hanson, 1990a, b; Hanson and Cook, 1997):

$$\varepsilon = k_d (\tau_a - \tau_c)^a \quad (2.10)$$

where, ε the rate of erosion (m/s), k_d the erodibility coefficient ($\text{m}^3/\text{N-s}$), τ_a the developed shear stress at the soil boundary (Pa), a the exponent generally considered to be 1.

No such study till now has been carried out to determine the erodibility parameters *in situ* and compare the results with the existing empirical methods for the banks of the Brahmaputra river. Therefore, it is important to measure the *in situ* erodibility parameters for the banks of the river Brahmaputra. This is one of the critical parameters in bank erosion processes, which involve the uncertainty in measurements and spatial distribution.

2.3 Seepage Erosion of River Bank

Loss of the river bank materials by exfiltrating seepage is called seepage erosion. This mechanism of erosion is widespread in occurrence and is very significant to the bank stability although it is rarely recognized (Hagerty, 1991a, b). Alternatively, high infiltration rates can cause the development of the perched water table above the less impermeable layers, thus large hydraulic gradient can initiate towards stream channels causing fairly rapid subsurface flow towards stream (Fox et. al., 2006). Some relevant researches are discussed in the following paragraphs.

Hagerty (1991a, b) discussed the different conditions for the seepage erosion. The most important conditions necessary for the seepage erosion is the presence of the open exfiltration face, a source of water which can create sufficient water head, removal of failed or displaced materials and a sufficient hydraulic gradient. River bank provides the potential open exfiltration face. During the rising phase, the water from the river

infiltrates into the bank face and stored there as bank storage (Fox et. al., 2006). When the water level in the river recedes, this stored water goes back to the river creating the high seepage erosion in the high conductivity layer. Another important factor is the removal of displaced materials (Kochel et al., 1985). Flume studies pointed out that surface erosion rates increase in magnitude when the unsaturated pore water pressure increases to near saturation and thus decreasing the soil strength (Rockwell, 2002; Owoputi and Stolte, 2001; Fox et. al., 2006).

Many researchers carried out their study to investigate the role of seepage on erosion for the hill-slope stability, pointed out that the seepage pressure as the body force acting on some representative sediment volume (Terzaghi, 1943; Zaslavsky and Kassiff, 1965; Howard and McLane, 1988). Howard and McLane (1988) clearly described that for cohesionless sediments on the bank face eroded by ground water are combination of the three forces: a tractive force defined as the sum of all the forces on the seepage face, gravity force, and a seepage force exerted by the groundwater seepage on the sediment grain. Howard and McLane (1988) defined the seepage force through the following equation:

$$F_s = C_2 \frac{3\pi}{n} \rho g i d^3 \quad (2.11)$$

where, ρ is the fluid density, C_2 an empirical constant, i the hydraulic head gradient, and n the porosity. Seepage force dominates in a narrow “sapping zone” at the flow discharge point and erosion occurs as a continuous outward sediment movement through this zone. The subsequent undercutting initiates failure of the “undermining zone” which is above the seeping zone (Howard and McLane, 1988; Periketi, 2005).

Seepage erosion or the soil-water pressure has been measured for the river bank erosion study by Burgi and Karaki (1971), Bradford and Priest (1977), Casagali et al. (1999), Rinaldi and Casagali (1999) and Wilson et al. (2007). But they have only considered the bank stability, mass failure and seepage erosion without deriving the seepage erosion rate model. Depending on the critical failure angle of the bank materials, Howard and McLane (1988) proposed a long term sediment delivery flux.

In the recent works, Periketi (2005), Fox et al. (2006), and Wilson et al. (2007) conducted studies to develop the sediment transport model specifically by describing the seepage erosion model for the stratified river bank. They designed an apparatus called “Lysimeter” for that study. The apparatus represents the two-dimensional river bank profile. It is a 1 meter × 1 meter rectangular box with 0.15 meter width. One side along the width is kept open to represent the river bank face and the opposite side there is a porous plate, through which water can seep into the lysimeter from the constant water head tank adjacent to the plate. They conducted the tests for variable water head and for variable bank slope conditions. The soils were collected from the river bank and same degree of compaction was maintained. They found the erosion rate versus discharge follows a power law relationship. The same hypothesis was also suggested by Howard and McLane (1988):

$$E = a_1 Q^b \quad (2.12)$$

where, E the erosion rate, Q the discharge and a_1 and b the regression parameters. As the correlation coefficient was very high in the relationship, a sediment transport model was developed based on the theory of Howard and McLane (1988). For normalizing the

slope, grain size and boundary conditions, they presented dimensionless sediment transport model. The dimensionless sediment discharge (q_s^*) and the dimensionless shear stress (τ^*) are related through the power law relationship:

$$q_s^* = 584 \times \tau^{*1.04} \quad (2.13)$$

The above equation is quite similar to the empirical bed load function for fluvial erosion as described by Meyer-Peter and Muller (1948), Nelson and Smith (1989) and Istanbuluoglu et al. (2003).

Using the same concept of seepage experiment, Chu-Agor et al. (2008) developed an apparatus to investigate the three dimensional seepage erosion mechanisms on the river face or hill facet. A power law relationship was found between the magnitude of erosion and height of undercut of the bank.

Tokaldany et al. (2010) attempted to develop an analytical method to estimate the failure plane angle and the bank retreat in a cohesive bank. In that research, by analysing the laboratory and field data, a set of curves was introduced to estimate the tension crack depth. Moreover, the seepage flow effect from the river bank is also considered in the flume experiments. However, the model is only useful in a homogeneous river bank.

The above mentioned studies do not include the relationship between the time of bank collapse and fall of stage level. It is also important to determine the relationship of time of bank collapse with uncertainty of the bank materials for various compositions of stratifications.

2.4 Hydraulic Erosion Measurements of the River Bank

Several studies had been carried out to understand the mechanisms and the root causes of the failure among various influencing factors. Two major factors are excess shear stress and the seepage erosion, which were already discussed in the previous sections. Here, some studies are discussed related to the real field study of the bank erosion for the time of erosion, and other factors related to the high flood discharges.

Lawler and Leeks (1992) designed a device to acquire the actual erosion rate of the river bank along with the time of erosion. They named the device as “Photo-Electronic Erosion Pin” (PEEP). It is a 0.4 meter long transparent waterproofed acrylic tube of 12 mm internal diameter. Photovoltaic cells are arranged inside the tube in a systematic fashion linearly. These cells are connected to a data logger for the continuous record. The cells generate the electricity under the light and the corresponding voltage is recorded into data logger. This PEEP is installed in the river bank normal to the bank face. Finally, the instrument was calibrated for the voltage generated to the exposed length of the bank. The investigation shows that the erosion is dominated during the recession stage of the hydrograph. Lawler (1994, 2005) found after studying various flood events that there exists a lag period between the peak flood and the maximum erosion which may vary from hours to even days and the bank erosion mostly occurs during the recession phase of the storm hydrograph (Duan, 2005).

Julian and Torres (2006) carried out their study to identify the major controlling variable for the bank erosion. They considered four factors which are thought to influence the bank erosion: magnitude of excess shear stress, duration of excess shear stress, event peak excess shear stress and the variability of the shear stress peaks. They concluded that

the event peak of excess shear stress is the best predictor for bank erosion in case of moderately cohesive river bank while, the variability of the excess shear stress is the best predictor for the river bank with minimally cohesive soil.

2.5 Analytical Model for River Bank Erosion Estimation

Using the understanding developed through various research studies on river bank erosion processes, several analytical approaches have been proposed to calculate the river bank erosion rate. Ikeda et al. (1981) carried out their study to estimate the rate of bank erosion. They related the rate of erosion to the excess near bank velocity linearly. The excess near bank velocity is the difference between the depth averaged velocity and the cross-sectional mean velocity. If the excess near bank velocity is greater than zero, then the bank retreats otherwise the bank advances. They included an erosion coefficient to account for variations in bend geometry and the bank material properties. Similar approach was adopted by Parker (1976), Johannesson (1985), Odgaard (1989a, b), Crosato (1990), Lan (1990) and Larsen (1995). But, Oddgard (1989a) pointed out that the bank erosion rate can be correlated to the near bank flow depth rather than excess near bank velocity. Crosato (1990) included both the fluvial erosion and bank failure in his study. The developed model is suitable for defining the qualitative behavior of the meandering rivers. However, the introduction of simplifications in that model limits the ability to provide reliable quantitative predictions of bank erosion (Piegay et al., 2005). Hasegawa (1989) developed a universal bank erosion coefficient relating the bank erosion rate to the cross-sectional mean velocity. His experiment was validated using the data from the alluvial channels in Japan. Hickin (1974), Hickin and Nanson (1975 and 1984), Osman and Thorne (1988) related the rate of bank erosion to the basic soil

properties of the bank materials. Osman and Thorne (1988) linked the basal bank erosion to the bank failure processes and described a planer bank failure model. Later, Darby and Thorne (1996a, b) extended this concept and incorporated pore water and hydrostatic confining pressure term. They pointed out that the probabilistic bank stability is the combination of the desired factor of safety and the expected soil property.

In the recent research studies, Duan (1998), Nagata et al. (2000), Duan et al. (2001), Darby et al. (2002) and Olsen (2003) developed the models which are capable of predicting the bed degradation or aggradation as well as the width adjustments. An empirical approach by Hasegawa (1981) was applied to the computational model by Nagata et al. (2000). The model was designed to predict the bank erosion process in the meandering river. However this model does not predict the geotechnical failure of the river bank. Duan et al. (2001) described an analytical approach to predict the rate of basal bank erosion and integrated to a two-dimensional depth averaged hydrodynamic model to estimate alluvial channel migration process. They described the rate of basal erosion as dependent on the longitudinal gradient of sediment transport, strength of secondary flow and the sediment eroded from the bank. Duan et al. (2001) and Duan (2001a, b) investigated the dependency of the basal erosion on the longitudinal gradient of sediment transport. They simulated the development, widening and the migration of the meandering channel. The simulated results were reasonably accurate to describe the field observation by Leopold and Langbein (1966). Darby et al. (2002) used a physically based bank erosion model to describe the bank erosion. They replaced the earlier bank erosion sub model in the 2-D depth averaged hydrodynamic and morphological model, RIPA, with the new model. Basal erosion of cohesive bank material and subsequent collapse of

the bank and then the transport and deposition of eroded materials were simulated in the model to predict the development of meandering channels. Although the probabilistic bank failure mechanism associated with the hydrologic events such as flooding was not incorporated in the model. Duan (2005) developed an analytical equation for predicting bank erosion rate based on theoretical analysis. He correlated the frequency of bank erosion with the frequency of flooding. The study indicated that the rate of bank erosion is directly related to the hydraulic forces, bank geometry, bank material cohesion and the frequency of flooding. Thus he correlated the geomorphologic processes to the hydrological processes. Jang and Shimizu (2005) used a model of a braided river with erodible banks to estimate channel evolution. Here, a moving boundary coordinate system was used to produce a naturally shaped boundary. The numerical scheme used the cubic interpolated pseudo-particle method in the flow field as the scheme introduced a little numerical diffusion. The secondary flow was also incorporated for calculating the sediment movement and the bed and bank evolution with time. However, they used a simple bank erosion model, i.e., it occurred when the bank gradient in transverse direction exceeds the angle of repose.

Recent attempts have been made to combine many important factors into the bank erosion model (Darby et al., 2007; Rinaldi et al., 2008). They focused two main aspects: to incorporate the positive and negative pore water pressure and confining pressure (Casagli et al., 1999; Simon et al., 2000; Rinaldi et al., 2004) and secondly, quantifying the effects of riparian vegetation on bank stability (Simon and Collison, 2002; Pollen and Simon, 2005; Pollen, 2006; Van de wael and Darby, 2007). Darby et al. (2007) developed an approach in which they fully coupled the hydraulic erosion, finite element seepage and

the limit equilibrium stability analysis in details. They also recognized that near bank shear stress governing fluvial erosion was gross idealization in their model. Rinaldi et al. (2008) further developed the model described by Darby et al. (2007) by incorporating more advanced hydrodynamic model to simulate reach-scale hydraulics in a river bend, in order to incorporate the spatial and temporal variability of the shear stress and resulting river bank erosion. They concluded that, in their investigation, fluvial erosion was less important than the mass failure being responsible for up to 30% of the total bank retreat.

Nasermoaddeli and Pasche (2010) emphasized that the undercutting of the riverbank, avalanche of the submerged zone on the bank, overhang bank failures are the dominant bank erosion processes in the non-cohesive river banks. They developed an approach to model those bank erosion processes by introducing a conjugate domain concept, in which, hydrodynamic, sediment transport and bed evolution equations were discretized on finite element conceptual domain, while bank evolution was computed on morphological domain. In the bank erosion estimations, avalanche and cantilever failure processes were included, while the pore water pressure distribution was approximated using an analytical function. They showed that their model prediction was better than the other available models.

These analytical bank erosion models consider mainly homogeneous bank materials, without stratifications until very recent studies. These studies also consider the planner failure mechanism or slip circle failure mechanism, which estimates the factor of safety at each simulation time step. But the failure of the composite banks is either due to tension or as beam of the overhanging soil blocks (Lawler et al., 1997). So an analytical bank erosion model is required to develop for the composite river banks.

2.6 Stochastic Hydrograph Generation

Several studies have been undertaken to determine the extreme flood magnitude and its frequency of occurrence based on the regional flood hydrograph analysis technique (Kumar et al., 1999; Kumar and Chatterjee, 2005; Fernandes and Naghettini, 2008) recently. Kumar et al. (1999) carried out a study for deriving regional flood frequency curves by using the L-moment approach coupled with relationship between annual maximum floods and catchment area for the seven sub-zones of India. The studies carried out by Indian CWC (1983) and Indian RDSO (1991) used the synthetic unit hydrograph and design storm event considering the physiographic and meteorological characteristics for estimation of the design floods. The regional flood frequency relationships study using L-moments approach for the gauged and un-gauged catchments of the North Brahmaputra region of India has been conducted by Kumar and Chatterjee (2005). In that study, various frequency distributions were employed and a regional flood frequency relationship was developed for the region.

A recent study carried out by Sirdas and Sen (2007) use the combination of isohyetal map, kinematic wave and rational methods for estimating flash flood synthetic hydrographs in arid regions. Sequential analysis of the time of concentration using empirical approach they related the flood peaks with the temporal intensities of the storm rainfall. Synthetic flood hydrograph generation method based on the flood frequency analysis over the sub-catchments for using in reservoir managements and hydraulic modelling has been demonstrated by Hickey et al. (2002). In that study, the synthetic hydrographs at the tributary levels were generated and combined to get the mainstream flood hydrograph and routed the flow to the downstream. Alternately, artificial neural

network (ANN) method has been used for the generation of synthetic flood hydrograph when the river basin and other meteorological complexities are present (Ahmad and Simonovic, 2005). The input parameters include antecedent precipitation index, ice melt index, winter precipitation, spring precipitation and time factor for generating flow hydrographs. Ahmad and Simonovic (2005) concluded that the ANN method can be used successfully in cases of deficit of historic flow data, which poses a difficulty to study the physical processes involved in the characteristics of flow hydrograph. Chau et al. (2005) used different models such as linear regression (LR), genetic algorithm based ANN (ANN-GA) and adaptive network based fuzzy interface system (ANFIS) for comparing flood forecasting methods. In that study, upstream stage level is used to forecast the downstream stage level. In other study by Chau (2006), particle swarm optimisation training had been used for ANN based model for river stage prediction.

Wang (2006) carried out a study on generating the flood hydrograph with return period concept based on nearly 70 years historic flood discharge data for two gauging station at Guadalupe river and Woodrow Wilson Bridge site. He generated the flood hydrograph combining the return period of extreme discharge and a fitted distribution for the flood peaks.

None of the above studies address the generalised behaviour of a flood hydrograph for a large river basin based on a return period of flood waves and the effect of the seasonal rainfall. The repeated flood waves with moderate return period may create more bank erosion compared to the annual extreme flood with design return period. So, a synthetic seasonal hydrograph generation technique, which can capture the nature of flood waves, for cumulative probability of achieving a particular stage and the relative

frequency of the daily stage change (rise/fall) is required for the stochastic bank erosion modeling.

2.7 Conclusions

From the reviews of the previous studies discussed above, the author has identified the following drawbacks as listed below:

- 1) The characteristics of the seasonal hydrographs for a large river basin are required to be determined by analyzing the historical records and then the synthetic seasonal hydrographs are required to be generated considering the same characteristics that of the observed one for different return periods of flood waves.
- 2) The erodibility of the fine soil layers in the composite banks of the alluvial river is required to be determined *in situ*. Literatures indicate very limited studies were carried out in this area. The uncertainty involved in the estimation of critical shear stress and soil erodibility coefficients leads to an unrealistic estimation of bank erosion. The probabilistic distribution of these erodibility parameters due to its spatial variability is required to be assessed considering its controlling factors.
- 3) Seepage erosion from a composite river bank and subsequent bank collapse is required to be characterized and functional relationships are required to be established between the seepage erosion with its controlling variables. Then a mathematical seepage erosion model is required to be developed for estimating the seepage erosion rate for a composite river bank.

- 4) A mathematical hydrodynamic and morphological model addressing the major bank erosion processes in the composite bank of a river bend is required to be developed. Previous studies clearly indicate the lack of such model to predict the bank erosion in a composite river bank and to understand the processes and their integrated effects on bank erosion prediction.
- 5) A stochastic nature of the bank erosion is required to be estimated, as the detailed hydrodynamic and morphological data are often not available for the river in the developing countries.



3.1 Introduction

This chapter addresses the details of the study area for different experiments, validation of the mathematical model and the gauging stations from where the historical stage hydrograph data has been collected. Moreover, this chapter describes the various data collected during the hydrographic survey and the analysis of those data for details understanding of morphological features of a selected channel bend. The close interrelationship between the river flow and channel formation governs many river channel processes such as meander planform, bed topography, bank erosion and lateral migration, which in turn provide the basis for analysis and hydrodynamic and sediment transport modeling in a curved channel (Chang, 1988). Under the influence of the centrifugal acceleration, the flow in the curved channel experiences secondary current as spiral motion of the flow directed normal to the main flow and the super-elevation of the water surface. The secondary current, which develops upon entering a channel bend, will eventually reach an equilibrium condition if sufficient bend length is available. In that state the flow is called as fully developed and the river profile does not change unless there is significant change in the discharge or other influential parameters such as sediment influx. The previous study by Richardson and Thorne (2001) indicate the existence of the multithread flow in a braided river. The hydrographic data were analysed here to identify the presence of multithread flow and its behavior, secondary current strength in a river bend and the range of sediment flux in the river channel during the high flood discharge during monsoon periods.

3.2 Study Area

The middle reach of the Brahmaputra river is considered for the present study. Within this river reach, different sites have been considered for different field observations and soil investigations. Fig. 3.1 shows the details of the study area considered along the Brahmaputra river. In the following paragraphs, the detail descriptions of different sites considered for the study is described.

3.2.1 River Gauging Stations

The historical stage hydrograph data analysis has been carried out for two river gauging stations of the Brahmaputra river. The river gauging sites at Guwahati city ($26^{\circ}10'N, 91^{\circ}44'E$) and at Tezpur town ($26^{\circ}37'N, 92^{\circ}48'E$) nearly 120 km upstream of Guwahati have been considered (Fig. 3.1). The Brahmaputra River is highly braided and complex in planform. However, due to presence of nodes (constrictions) at these two gauging sites, the river width is restricted and no major sand bars are present. At the northern bank up to Tezpur, Subansiri is the major tributary with drainage area of about 37,700 km², while Dhansiri is at the southern bank with drainage area of about 12,250 km². In between Tezpur and Guwahati, Kameng is the major tributary at the northern bank, having drainage area of nearly 14,450 km². Kapili, the major tributary at the southern bank within this reach, has the drainage area of about 15,900 km². The catchment area contributing flows up to Guwahati gauging station is about 424,100 km². The average annual rainfall depth spatially varies from about 400 mm to 6,000 mm in some parts of the Brahmaputra river basin. However, the annual rainfall is 1,844 mm at Tezpur and 1,561 mm at Guwahati (Datta and Singh 2004). Based on the climatic conditions the annual rainfall system can be divided into three major parts: a) pre-

monsoonal rainfall (February to April), b) monsoonal rainfall (May to October), and c) winter rainfall (November to January). Up to 80% of the annual rainfall occurs as monsoonal rainfall. The average rainy days are 130 and 115 for Tezpur and Guwahati, respectively.

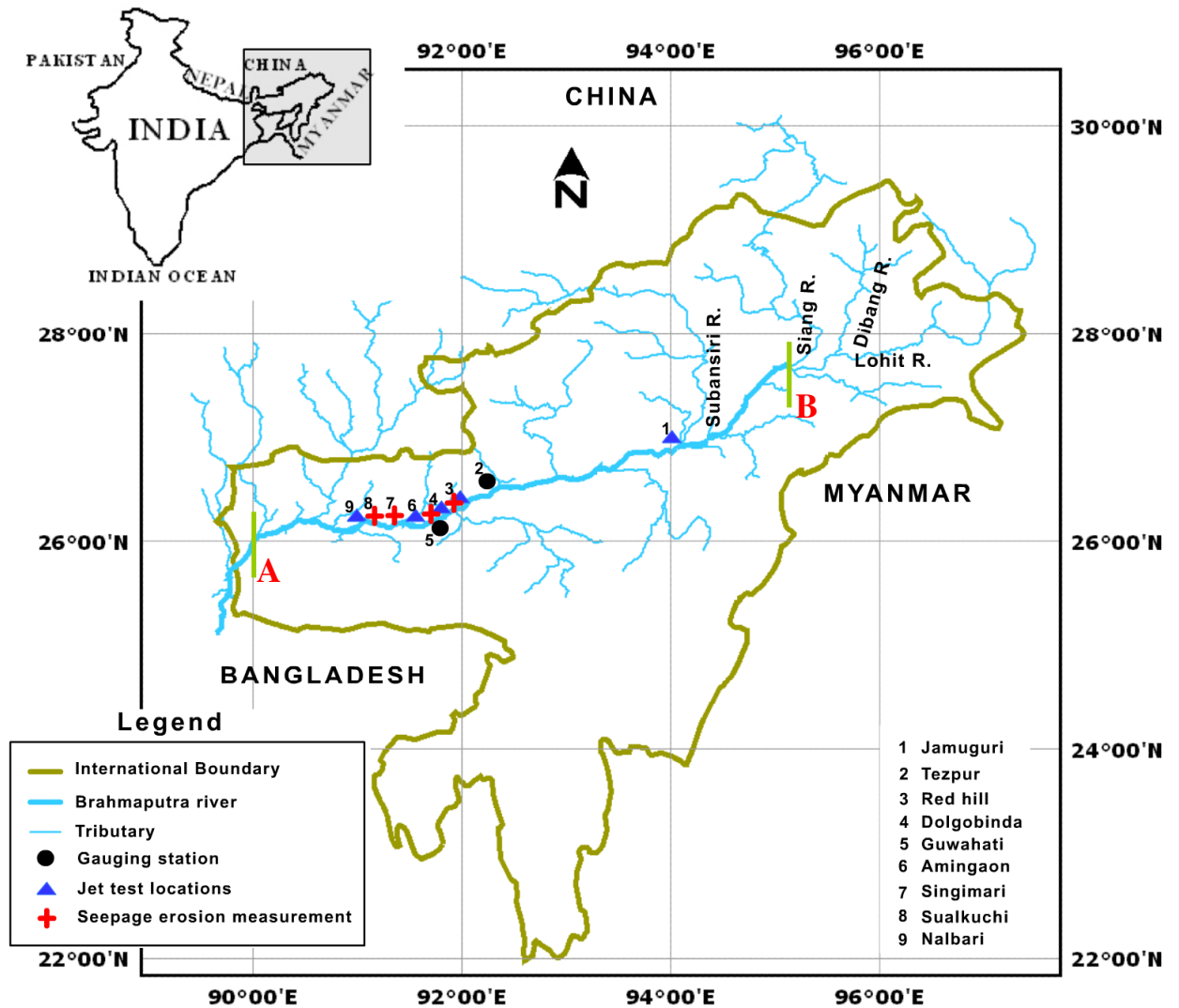


Fig. 3.1. Brahmaputra river at the North-East India showing the study sites (A-B is the river reach considered for satellite imagery analysis)

3.2.2 Submerged Jet Tests

Four field sites were selected along the banks of the Brahmaputra River (Fig. 3.2). One of the sites is located at the upper part of the Brahmaputra river, named Jamuguri ($26^{\circ}50'08''N, 93^{\circ}46'08''E$). The other two sites are near Guwahati in the middle section of the river, and the last one at Nalbari.

Almost all parts of the river experience bank erosion at a very high rate (Sarma, 2004). River bed materials do not vary significantly within the study reach, with an average D_{50} of 0.20 mm (Dutta et al., 2008). Except for the red hilly formation, the river bank is mostly composite with sand ($D_{50} = 0.2$ mm) and fine silt ($D_{50} = 0.06$ mm) soils. Sand layers are sandwiched between two fine silt layers. The hilly formations are formed with red coloured soil ($D_{50} = 55 \mu m$) which have a sticky nature. Fig. 3.2 depicts the particle size distribution of some soil samples collected from the bed and banks along the Brahmaputra river. Local surveys demonstrate that this layer is more resistant than the other soils normally found in the alluvial flood plain. Only the Jamuguri site was monitored for river stage records, bank line survey and bathymetric survey during the rainy season (June to September, 2007 and 2008).

3.2.3 Seepage Erosion Measurements

After investigating the river bank along the Brahmaputra river, four sites has been found appropriate to measure the seepage erosion. As most of the cases, the bank erosion is sum of the fluvial and seepage erosion. Therefore it is very important to identify the sites where, the seepage erosion is predominant. The four locations are: Dolgobinda ($26^{\circ}11'25''N, 91^{\circ}43'26''E$), Amingaon ($26^{\circ}10'55''N, 91^{\circ}41'24''E$), Sualkuchi

($26^{\circ}09'50''\text{N}$, $91^{\circ}34'26''\text{E}$) and Singimari ($26^{\circ}10'42''\text{N}$, $91^{\circ}35'15''\text{E}$). Fig. 3.3 shows the field photographs collected during the seepage erosion measurements at these sites.

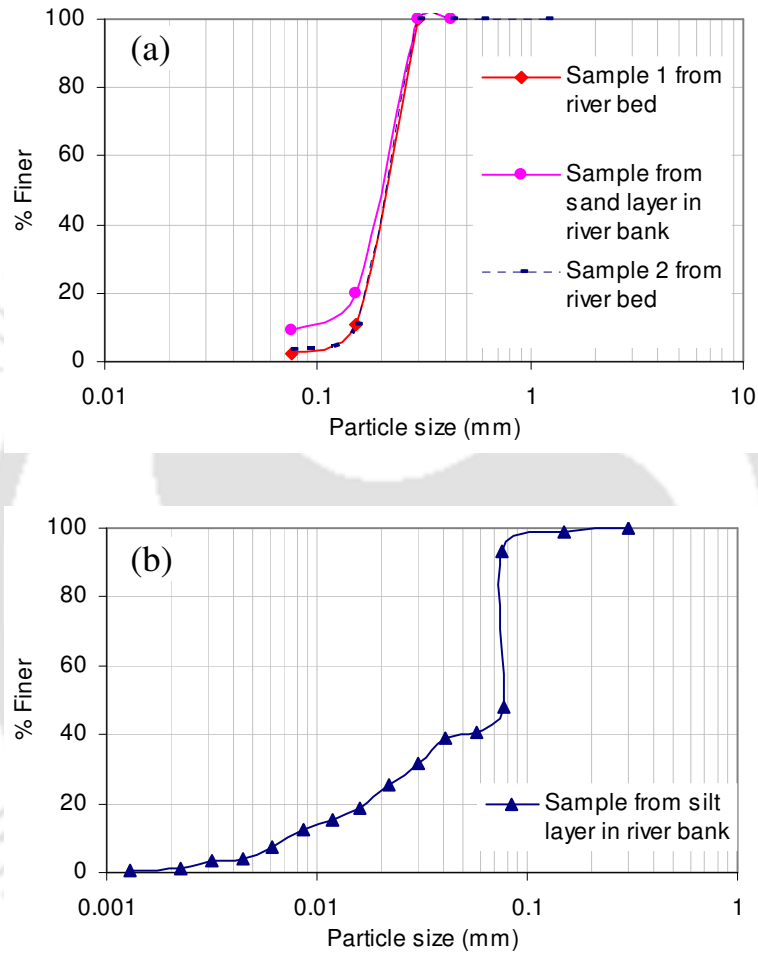


Fig. 3.2. Particle size distribution of the soil samples collected from: (a) the river bed and sand layer in river bank, (b) the silt layer in river bank.

3.2.4 Site for Model Validation

Detailed hydrographic and river bank line survey was conducted at the upper reach of the Brahmaputra river at Jamuguri, India. One braided loop of the river is in this site. The site

is about 70 km upstream of Tezpur town. Figure 3.4 shows the geographic location of the site. The river flows from east to west at this location. Morphological studies using multi-date satellite imagery show that after 2004 extreme flood event, a large river bend has been formed at the location. Till the year 2009 the river bank was under severe threat of erosion. It also has been found from the satellite imagery study that a bank area of 2.4 km² was eroded out during the flood season of 2004. The radius of the centerline of the bend is about 2460 m and the channel widths at the upstream and downstream are 630 m and 1126 m, respectively (Karmaker and Dutta, 2009).

The developed mathematical bank erosion model has been validated for this channel bend with detail morphological observations over a period for the years 2005 to 2008. Moreover, the 45 channel bends from the middle reach of the Brahmaputra river have been selected for further evaluation of stochastic prediction of the model. However, the details of these locations and observations are discussed in Chapter 8.



Fig. 3.3. Field photographs showing the seepage undercut and collapse along the Brahmaputra river bank: (a) collapse after seepage erosion (b) seepage undercut.

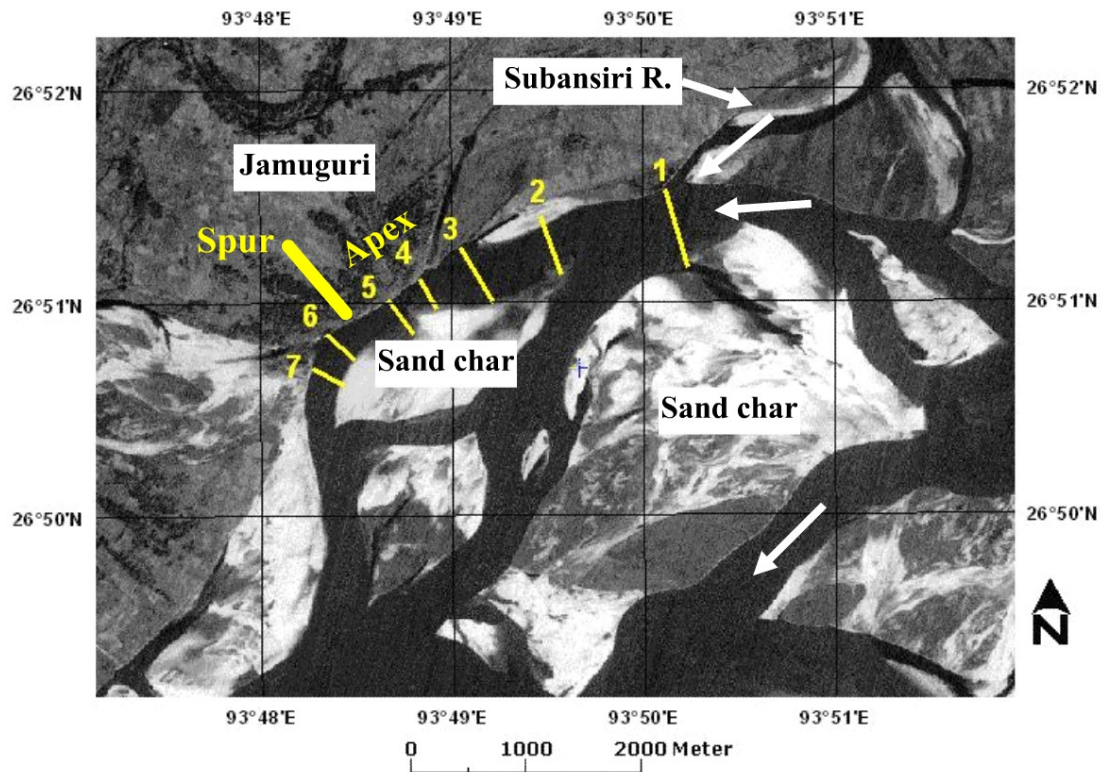


Fig. 3.4. Study river reach at Jamuguri (Arrows show the flow direction, numerals indicate the survey transects)

3.3 Field Observations

The detailed analysis of the stage records, soil characteristics will be discussed in the respective chapters. In this section, the basic data collected at the study sites has been discussed.

3.3.1 Characteristics of Braided Loops

In a braided river system, the main channel bifurcates in an irregular interval and meets again after crossing the deposited sand bars within the reach. After bifurcation, when the main channel attacks the bank, it becomes under the severe threat of erosion. Fig. 3.5 depicts the similar kind of the braided river stretch of the Brahmaputra. Here, the main

channel flows along the right hand side of the bank and form a bend, and thus attacking the bank. As shown in the figure, the main channel can be approximated as a sine generated curve, the half wave-length of which will represent the distance between the two deflection points: A-B.

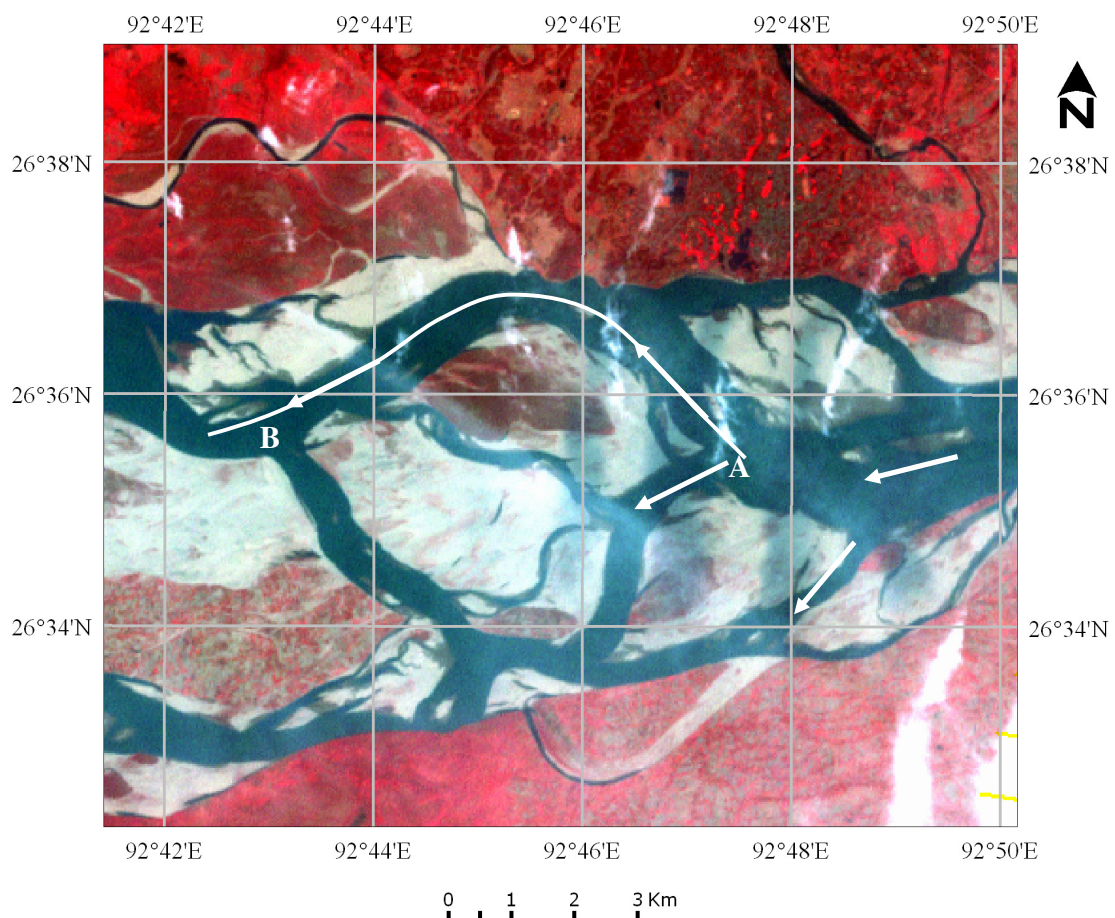


Fig. 3.5. Typical river channel bend approximated as a sine curve and attacking the bank.

(The arrows show the flow direction and the white line shows the approximated sine curve: A-B)

3.3.2 Hydrographic Survey

At Jamuguri detailed hydrographic survey was conducted twice: during the moderate flow condition and during high flood condition (water level nearly 1m higher than the

bank full discharge) for two years 2007 and 2008. The measured discharge during the high flow and moderate flow stages were about 4200 m³/sec and 2900 m³/sec, respectively. The survey includes detailed data collection like bathymetry using GPS aided Echo-sounder and velocity profile through ADCP (Acoustic Doppler Current Profiler). The measuring devices were mounted on an engine propelled vessel. The accuracy of the GPS instrument was +/-1.5 meters. The longitudinal bed slope is very gentle and about 1 in 7000 (Sarma, 2004).

The advanced river survey instrument like ADCP (Fig.3.6) provides the three dimensional velocity profiles on a continuous moving boat facility. The instrument also keeps the record of the backscatter data which can be used to obtain the suspended sediment concentration in the river (Gartner, 2004; Merkelbach, 2006). Gartner (2004) calibrated the back scatter data obtained from the ADCP instruments (manufacturer RD instruments) at two locations with sufficient point sample data. He found that the suspended sediment concentration (C_s) (in mg/lit) is related with the relative back scatter (RB) measured in decibell (dB) as follows:

$$C_s = 10^{(A+B' * RB)} \quad (3.1)$$

where, A and B' are the calibration constants for the instrument. The above formula is based on the sonar equation for sound scattering from the suspended particles in the flow. The calibrations constants are determined through the method of regression of concurrent back scatter with the known mass concentration measured using a semi-log plane. The relative backscatter (RB) is the sum of the echo level measured at transducer plus the two-way transmission losses (For details, Thevenot et al., 1992).

As in the present study, same model of ADCP instrument was used. However, to verify the calibration of the instrument with the one used by Gartner (2004), few water sample data were used. Water samples were collected from the location at the various depths and analyzed in the laboratory for the suspended sediment concentration. It has been found that Equation (3.1) holds good for the present case.



Fig.3.6. Photographs showing the ADCP instruments: (a) 4 transducers (b) installed in a survey vessel during data collection

Analysis of the ADCP data has been carried out using Matlab® software (ver. 7). A 3×3 median filter window has been used to eliminate the local noises in the raw ADCP data for analyzing velocity and sediment concentration.

3.3.2.1 Primary velocity variation

Primary velocity plots for each transect were examined for the changes in the velocity pattern. Velocities for two flow conditions as mentioned earlier were analysed. Just upstream of the transect-1, one tributary (lower branch of Subansiri river) meets to the river bend. During the moderate discharge condition two distinct flow patterns are visible (Fig. 3.7a): relatively low velocity zones upto 70 m from the bank and a very high velocity zone near 80 m. In transect-2 (Fig. 3.7b), two flows are partially mixed up, although multi-thread flow is visible. Richardson and Thorne (2001) also found the presence of similar multi-thread flow in the braided river system for Brahmaputra river reach in Bangladesh. The cores of the flow-threads are at 30 m, 70 m and 125 m. Due to the presence of sand bar near the bank (Fig. 3.4), the main flow has shifted towards the left bank (Fig. 3.7c). But at the apex of the bend (Fig. 3.7d), due to the centrifugal motion there is a formation of deep scour pool near the bank. The bank erosion is also active in this transect. Distinct transverse slope towards the bank ensures the presence of very high primary velocity near the bank. Similarly, in the transect-5, transverse slope is present and a very high velocity core is near the bank. However, multiple thread flow is also visible. In the next transect (Fig. 3.7f), due to the presence of land spur just upstream of the transect, the main flow has shifted towards the left side sand bar. But the low velocity zone near the bank is not exhibited in the previous transect as it is away from the existing land spur. However, little downstream of the structure, the multi-thread flow is again

prominent (Fig. 3.7g). During the high flood period, velocity distribution (Fig.3.8), shows the similar trend of moderate flow. The first two transects (Transect-1 and 2) data are absent due to instrumental data recording problem. For other transects, locations of high velocity zone fairly match with that of the moderate flow condition. In transects 4 and 5, near the apex of the bend, transverse bed slope is prominent whereas in other transects it is absent. Multi-thread flow pattern are found in all transects. However, at transect 5, two flow threads are closely spaced due to curvature effect, while they are well separated in other transects. Primary velocity at the thread core (1.9 m/sec) was found to be significantly higher than the moderate discharge condition (1.5 m/sec).

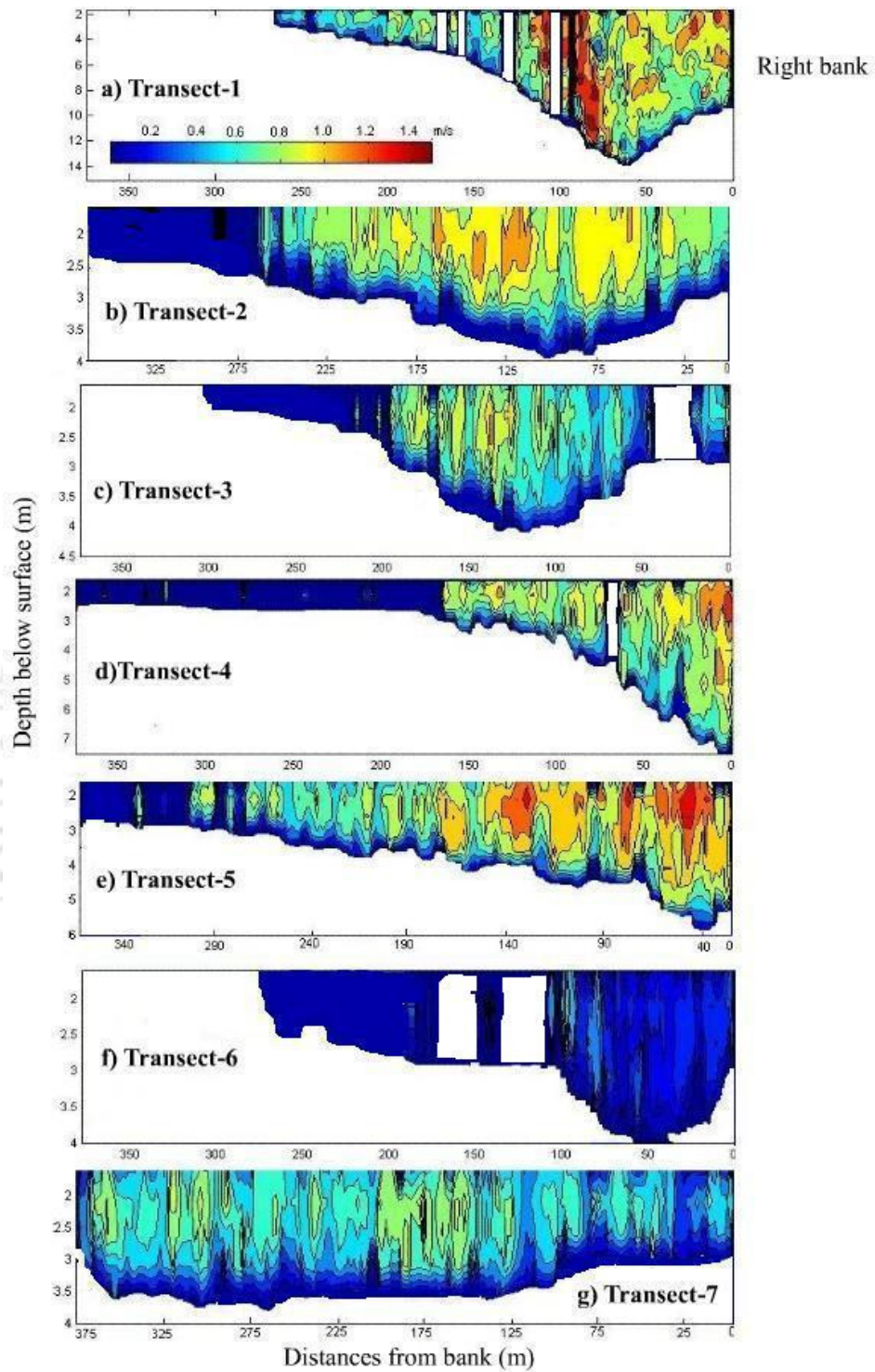


Fig. 3.7. Primary velocity contour for the survey transects during moderate discharge condition. a) transect-1, b) transect-2, c) transect-3, d) transect-4, e) transect-5, f) transect-6, g) transect-7 (White strips in the figures are due to data recording error)

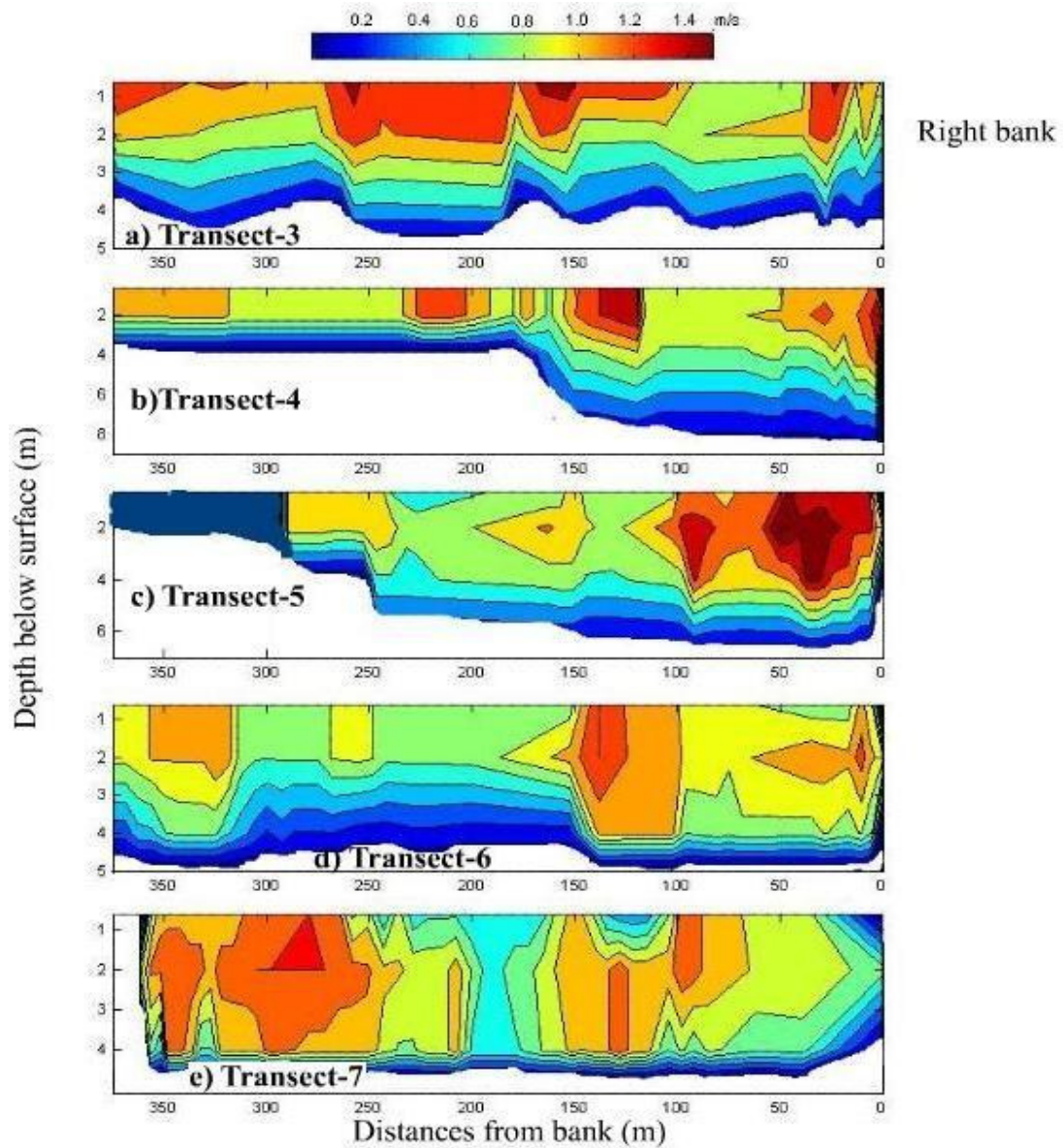


Fig. 3.8. Primary velocity contour for the survey transects during high discharge condition. a) transect-3, b) transect-4, c) transect-5, d) transect-6, e) transect-7

3.3.2.2 Secondary current variation

The transect-4 in the present study is located exactly at the apex of a channel bend. Moreover, this section is also under active bank erosion. It has been found from the secondary current plot along the survey transects; that the strength of helical motion is

negligible except transect-4. Fig. 3.9a shows the vector plot of the secondary current, while Fig. 3.9b for the velocity contour of secondary current. The circular motion of the secondary current in the river bend is found to be absent in this transect. For mildly sinuous river channels (sinuosity < 1.2), secondary current is negligible, as reported by Chen and Duan, 2006. As this channel bend lies in mildly sinuous category (sinuosity ~ 1.06), the present field studies reconfirms about the secondary current. The velocity vectors indicate that the secondary current directs towards the bank and almost horizontal. Due to bathymetric unevenness at 30 m from the bank, two cores of secondary flow are prominent at 20 m and 40 m. It can also be noted that the superelevation, which is very common in curved flow, is not evident in Fig. 3.9. As the channel radius is very large, the estimated superelevation is significantly small (~ 14 mm) in comparison to depth of flow (Chang, 1988). For this reason, during the survey, the channel water surface is considered as horizontal in the transverse direction.

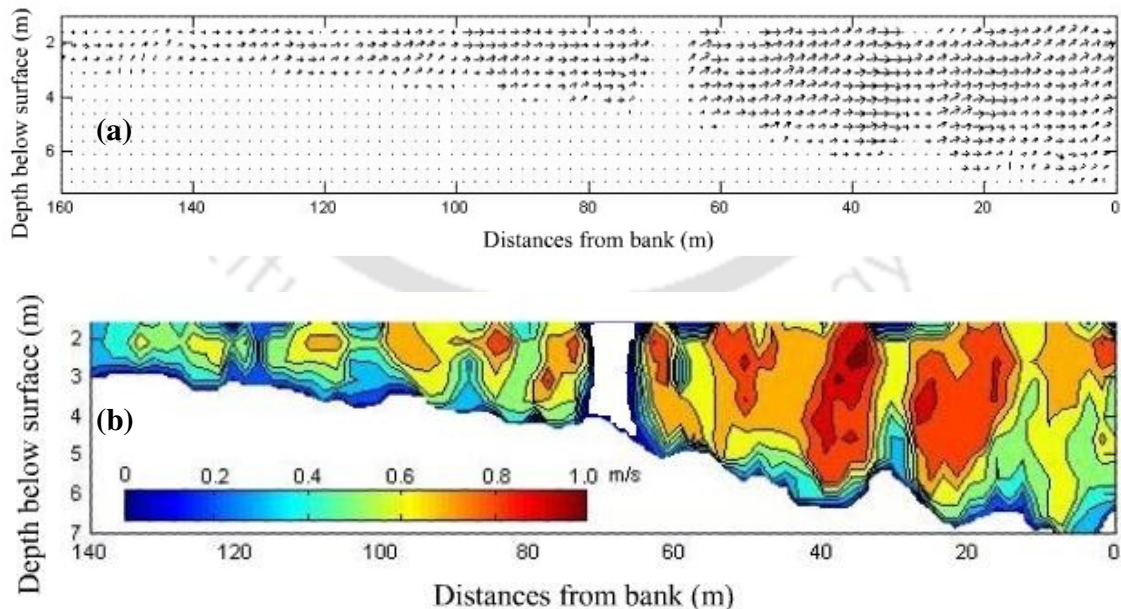


Fig. 3.9. Secondary current at transect-4 during moderate discharge condition. a) vector, b) magnitude (White strips in the figures are due to data recording error).

3.3.2.3 *Suspended sediment profile*

From the backscatter data of ADCP, the suspended sediment concentration has been estimated for each of the survey transects. Fig. 3.10a-g shows the suspended sediment concentrations in these transects. In transect-1, as a tributary (Subansiri river) joins the main river, two separate sediment zones are visible: zone-1 at the right hand side upto 75 m carries very less sediment load less than 200 mg/lit. Beyond 75 m, a zone of higher sediment load is found. In the next two transects, high sediment concentration is observed, which indicates the higher bed scouring process. Just downstream of these two transects, a bar formation resulting from the high sediment concentration has been found during the survey. Due to this, relatively less sediment concentration is found in the transect-4 (Fig. 3.10d). As this transect is under the severe bank erosion, huge sediment discharge from the bank supplied to this section. This phenomenon is reflected in the next transect-5 (Fig. 3.10e). Very high sediment concentration is observed near the bank and extended upto the top of the bank. But due to presence of the land spur at the downstream of this transect, and the formation of relatively still water pool enhances the deposition of these sediment and thus less sediment concentration is observed in transect-6 (Fig. 3.10f). Further downstream, the scouring is only at the left hand side, and a relatively less sediment concentration zone is observed near the bank.

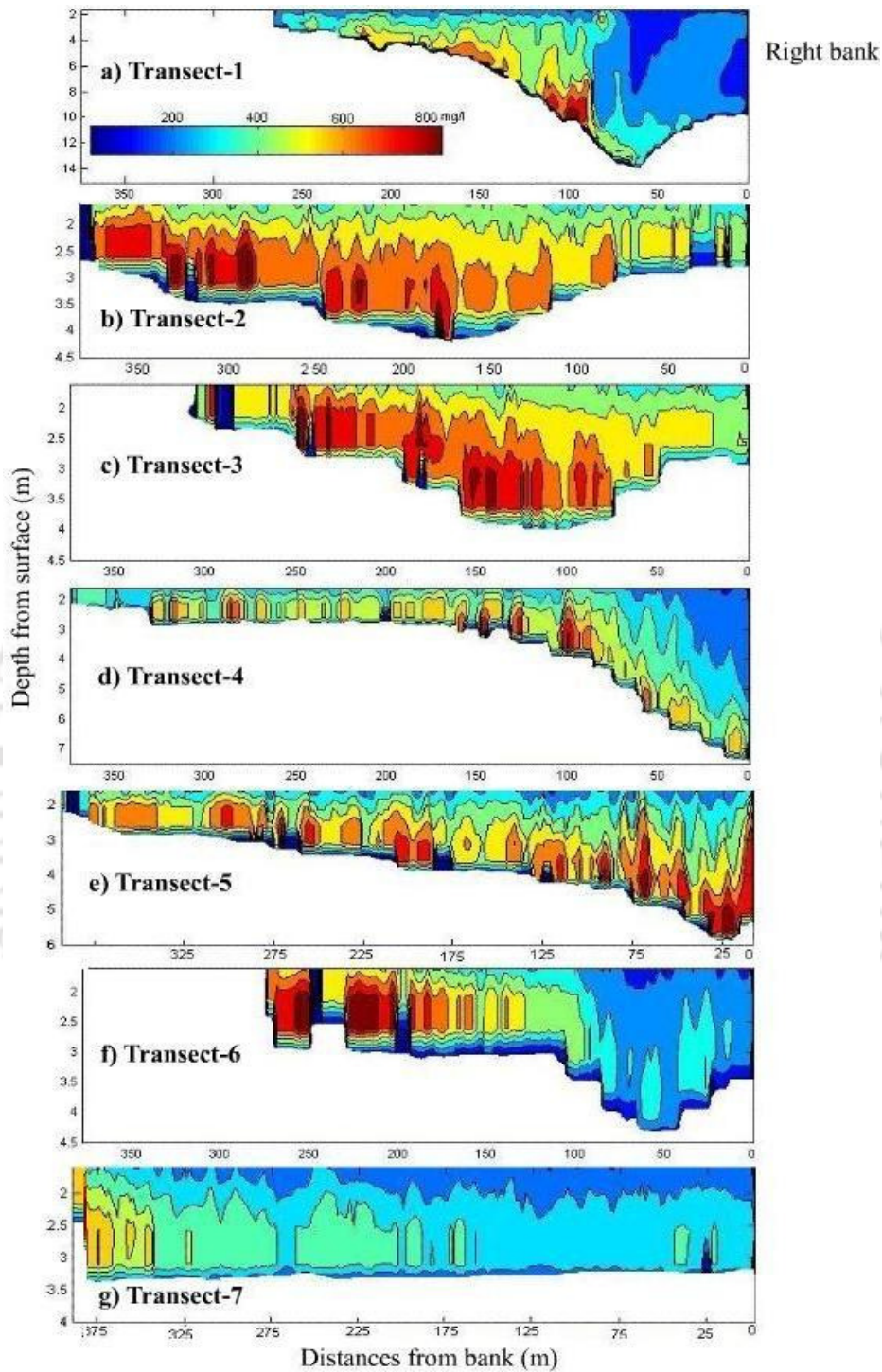


Fig. 3.10. Suspended sediment contour for the survey transects during moderate discharge conditions. a) transect-1, b) transect-2, c) transect-3, d) transect-4, e) transect-5, f) transect-6, g) transect-7

3.3.2.4 Transverse bed slope function

Several researchers have contributed towards determining the equilibrium sediment profile and bed-material concentration. These studies were mainly based on laboratory experiments and limited in-situ observed data at river scale. Zimmermann and Kennedy (1978), Falcon-Ascanio and Kennedy (1983), and Odgaard (1981) developed the bed slope function from flume studies and calibrated using the river data. Falcon-Ascanio and Kenedy (1983) assumed that the radial component of fluid force balances the submerged weight component down the transverse bed slope and thus defined the transverse bed slope as:

$$S_t = \frac{D}{R_c} F_d \frac{\sqrt{8\tau_{*c}}}{1-\lambda} \frac{1+f^{1/2}}{1+2f^{1/2}} \quad (3.2)$$

where, S_t is the transverse bed slope, D the average depth of flow, R_c the radius of curvature, τ_{*c} the critical Shields stress, λ the bed layer porosity, f the friction factor and F_d is the particle densimetric Froude number defined by:

$$F_d = \frac{U}{\sqrt{\frac{\rho_s - \rho}{\rho} g D_{50}}} \quad (3.3)$$

where, U is the depth averaged longitudinal velocity.

Odgaard (1981) developed a method for computing transverse bed profile based on incipient motion of sediment. It was assumed that, the sizes of the bed forming particles are just about to move in the longitudinal direction by rolling about their points of support. This is due to the fact that a particle starts moving only when the grain shear stress exceeds the critical shear stress τ_c .

$$S_t = \frac{3\alpha}{2} \frac{D}{R_c} \frac{U^2}{((\rho_s - 1)gd_{cr})^{1/2}} \left(\frac{1+m'}{m'(2+m')} \right) \quad (3.4)$$

where, α the projected area-volume ratio of the particle, d_{cr} the diameter of particle whose motion is impending, m' the reciprocal of the velocity exponent for grain roughness.

Zimmermann and Kennedy (1978) equated the radial component of the fluid drag force parallel to the bed with the particles submerged weight component. These are related to the stream flow by equating directly the moment of the radial boundary shear stress to the moment produced due to non uniformity of the centrifugal force about the channel axis. The proposed final form of the transverse bed slope can be given by:

$$S_t = 0.5\alpha \frac{D}{R_c} F_D^2 \left(\frac{1+nun}{nun(2+nun)} \right) \quad (3.5)$$

where, F_D the drag force of the fluid and nun is the nunner's number, equals to the inverse of the square root of friction factor.

Fig. 3.11 depicts the observed bed level at the two survey transects (1 and 4) for the two consecutive years: 2006 and 2007. The transverse slope shows a contrast between the transects: upon entering a river bend (transect-1) and near the apex of the river bend (transect- 4).

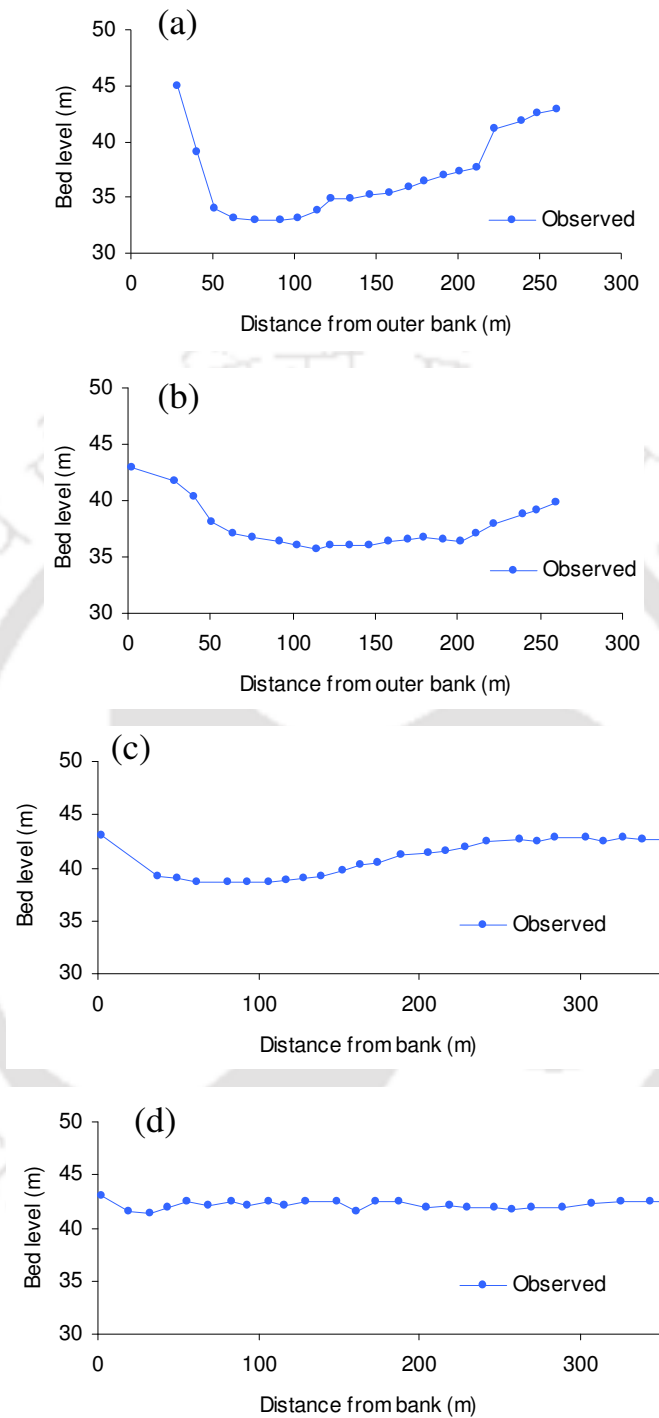


Fig. 3.11. River cross-section data collected during the bank-full discharge at (a) year 2006; transect-4 (b) year 2006; transect-1 (c) year 2007, transect-4 (d) year 2007; transect-1

Transverse bed-slopes predicted by Falcon and Kenedy (1983), Odgaard (1981), Zimmermann and Kennedy (1978) approaches are compared with that of in the measured cross-sections obtained from the survey. For this, four cross-sections (Transect 1, 2, 4 and 5), where the transverse bed slope is prominent, are considered. Fig. 3.12 shows the comparison of the Falcon and Kenedy, Odgaard and Zimmermann and Kennedy approach with the observed transverse bed-slopes. Result indicates at each of the four cross-sections that Odgaard's model gives better estimation of the transverse bed slope for the study reach. The main difference in Odgaard's model to that of others approaches is that the grain roughness was considered to compute the transverse bed slope. As the sorting process of the bed particles in a river bend takes place, it controls the distribution of grain roughness along the transverse direction.

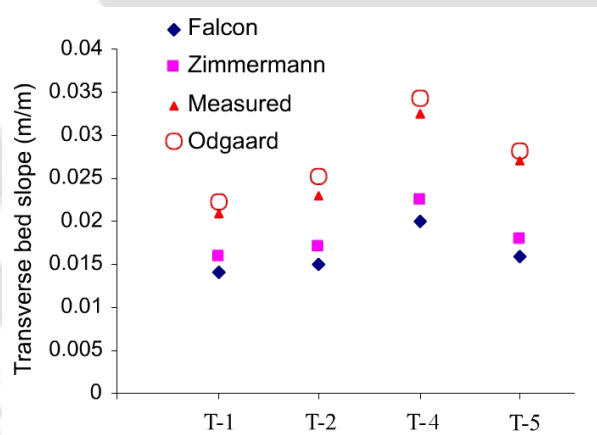


Fig. 3.12. Comparison of the transverse bed slope functions with the observed data (T-1: transect-1; T-2: transect-2; T-4: transect-4; T-5: transect-5)

3.3.3 Stage Level Recording

Automatic stage level recorder was installed *in situ* to get the stage during the monsoon period (May to September) for the years 2007 and 2008. For this, two pressure

type water level recorders were used (Virtual electronics, Roorkee, India). One is kept as a standby in case of failure of one sensor. Fig. 3.13 shows the details of the recorder and the sensor installed *in situ*. The sensor is connected to data logger to record with one hour temporal resolution. Stage data were downloaded through a data shuttle at one month interval. Fig.3.14 shows the sample stage records collected with the automatic data recorder.



Fig. 3.13. *In situ* installation of the automatic stage recorder: (a) installation of pressor sensor inside the outer casing, (b) automatic data logger and the solar panel, (c) the pressor type sensor for stage recording.

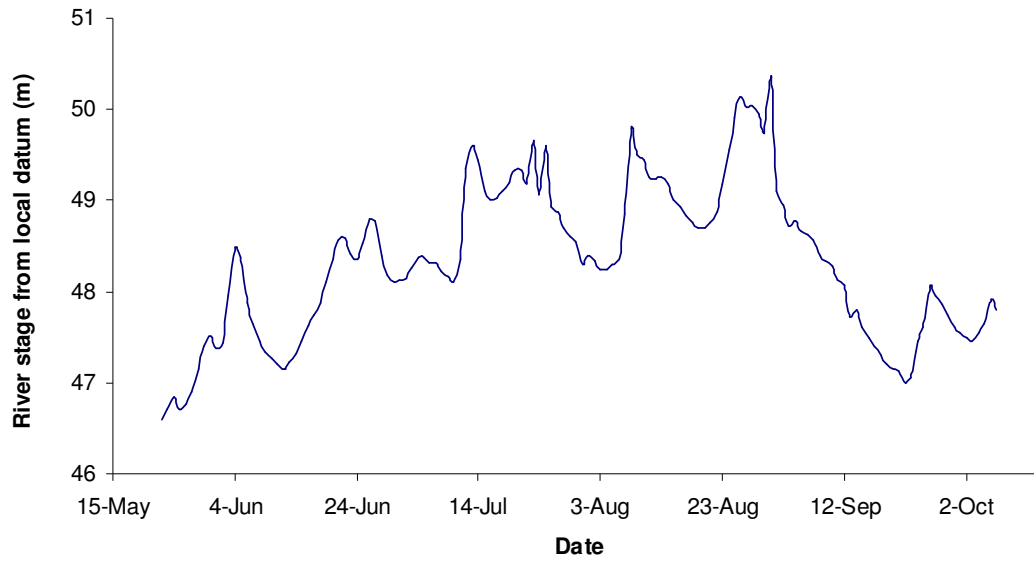


Fig. 3.14. Typical river stage records at Jamuguri during the monsoon season

3.3.4 River Bank Soil Profile

The analysis of the soil samples reveals that the soil near the ground surface is fine textured (silt) of about 1 m deep. In between the sand layers fine textured soil layers are interlaced alternately. This is a common composite bank soil layering often found in alluvial rivers where frequent inundations of the flood plain occur. Fig. 3.15 shows the field photograph of the bank stratification at Jamuguri. Median bulk density found for the fine soils is 17.62 kN m^{-3} with an average natural moisture content of 33%. In comparison the median bulk density for the sand soil is 14.5 kN m^{-3} with the average moisture content of 34.5%. Fig. 3.16 shows the detailed soil layers at two sites (Jamuguri and Dolgobinda) along the river bank.

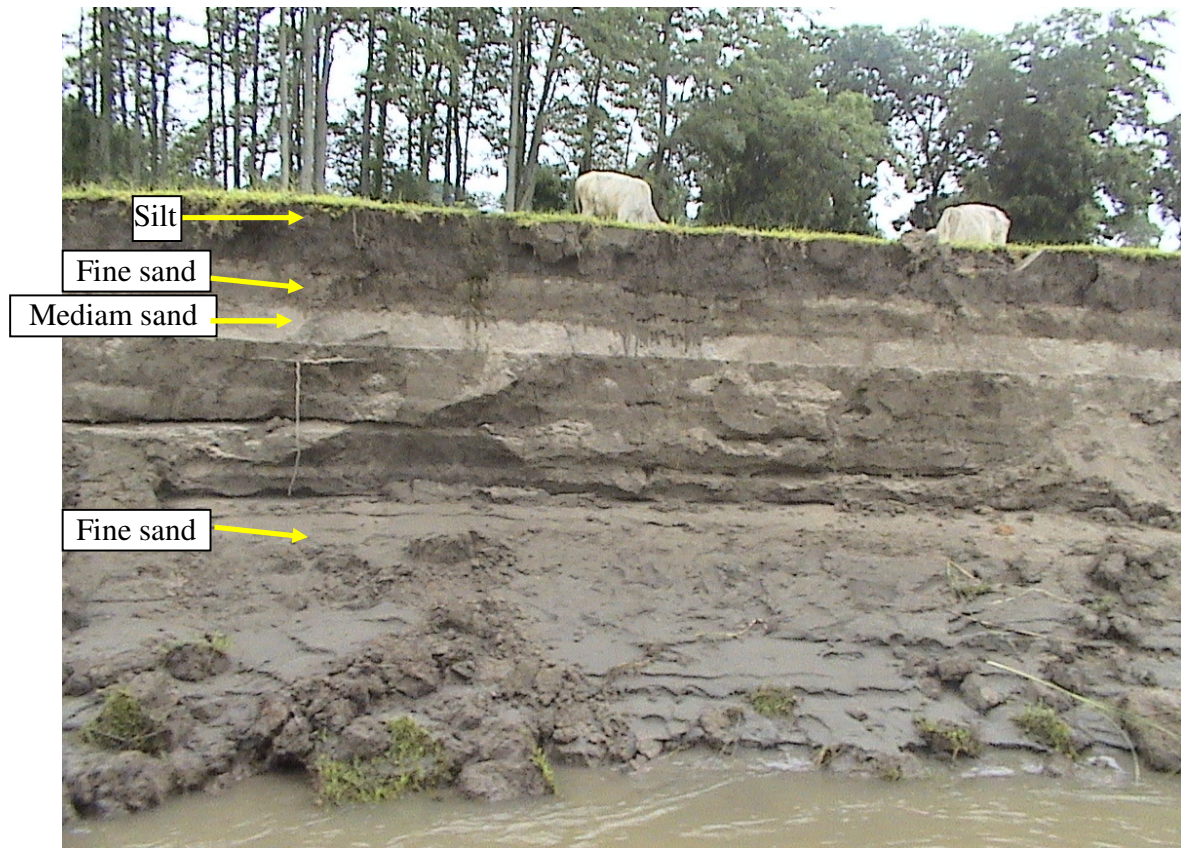


Fig. 3.15. Field photograph showing the bank stratifications in the composite river bank of Brahmaputra river

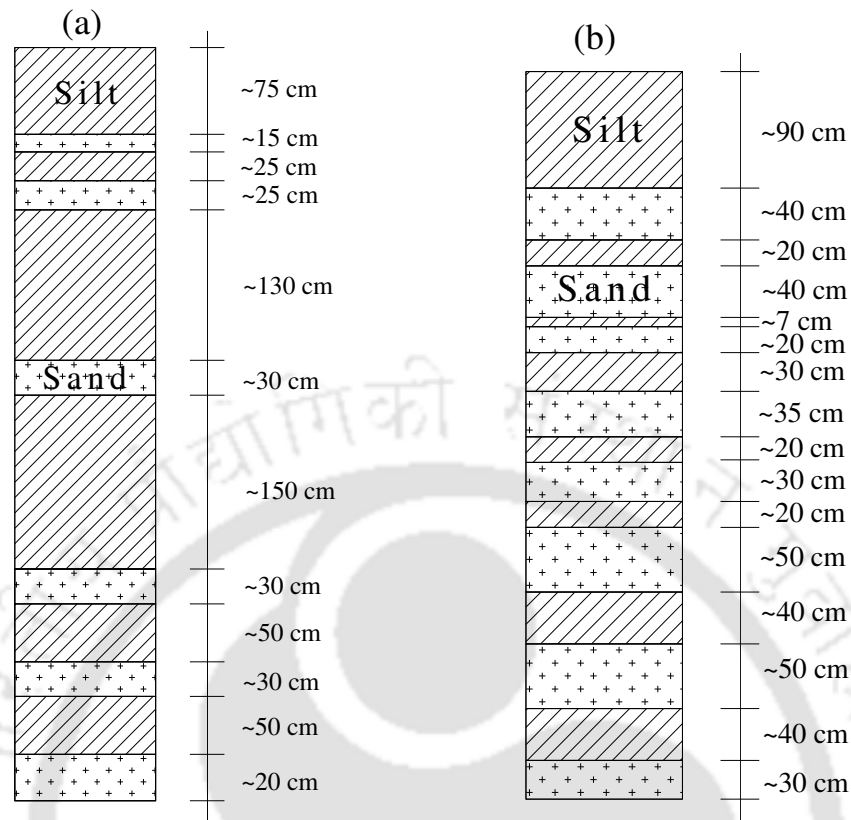


Fig. 3.16. Typical soil stratification along the Brahmaputra river at: (a) Jamuguri and (b) Dolgobinda.

3.3.5 In Situ Total Bank Erosion Measurement

In situ bank erosion and its spatial pattern can be measured with the help of erosion pins (Lawler, 1993). Initially, the bank erosion of the Brahmaputra river at Jamuguri was also decided to measure with the use of erosion pins. These erosion pins are of 16 mm diameter and 4 meter long. For measurements of the erosion rate, these pins were marked with smallest divisions of 5 cm. The marks were alternately painted with red and yellow colour for its easy measurements. Total 36 erosion pins were used in 12 sections: at each section 3 pins were used in 3 silt layers (topmost, middle and bottommost). The pins were droved normally into the bank with the help of a hammer (Fig. 3.17). Initially, while

driving the pins into the riverbank, 10 cm (2 smallest divisions) were kept exposed for its easy finding during the measurement (Fig. 3.18a). The daily erosion measurements were scheduled throughout the monsoon periods.

However, two problems were faced for the erosion measurement with erosion pins in the Brahmaputra river. First, the erosion rate is very high during the flood periods. Due to this reason, many of the erosion pins were lost with the flow, as the erosion in a high discharge periods was higher than 4 m. Secondly, during the monsoon period, the river was almost at its bankfull condition; thus it is difficult to measure the daily bank erosion. The presence of very high velocity near the bank also poses the difficulties to stand near the bank and measure the readings of erosion pins. Only few sections, where the seasonal bank erosions were minimum, it was possible to measure the seasonal erosion (Fig. 3.18a and b). For this reason, it was found that the repeated bankline survey was more useful and used for model validation.



Fig. 3.17. Insertion of the erosion pins into the bank



Fig. 3.18. Bank erosion measurement through erosion pins: (a) initial exposure of the pin, (b) After partial erosion of bank

3.4 Conclusions

This chapter discusses the details of study area along the river Brahmaputra. The detail hydrodynamic, morphological and bank erosion measurement was carried out at a river bend at the upper reach of the Brahmaputra river in India. However, the soil samples collection, *in situ* soil erodibility measurement, historical river stage records analysis and seepage erosion measurements were carried out at the locations from the upper to middle reach of the Brahmaputra river.

The analysis of the hydrographic survey confirms the presence of the multi-thread flow in the river channel of the Brahmaputra as earlier reported by Richardson and Thorne (2001). The major flow thread was found at the outer bank near the apex of the river bend. This indicates the presence of the higher shear stress zone near the bank.

However, due to the mildly sinuous bend ($\text{sinuosity} < 1.2$) the secondary current strength was negligible at the bend apex. So, for the modeling of the bank erosion in a mildly sinuous river bend, for simplification, the secondary current component can be ignored. The analysis of the survey data also indicate that the range of the sediment concentration in the river channel is 200-600 mg/lit (median concentration ~ 450 mg/lit), while at few locations are having sediment concentration of 800 mg/lit or more. The analysis also indicates that the transverse bed slope at the river bend can be approximately estimated using Odgaard's approach.

CHAPTER 4

GENERATION OF STOCHASTIC HYDROGRAPH

4.1 Introduction

Frequency analysis of historical hydrographs of a river basin is commonly carried out for assessing the risk involved in flood inundation of the important land, associated economic losses, and designing river training works in and around rivers. For the extreme flood frequency analysis with return period up to 100 years, the standard regional flood frequency analysis methods give satisfactory results for small to medium size river basins (Kumar and Chatterjee, 2005; Fernandes and Naghettini, 2008). However, the seasonal flood hydrograph of a large river basin comprises of multiple peaks within a rainy season every year. The characteristics of the hydrographs become more complex due to the interaction of different geographic features, land use and land cover, spatio-temporal variation of the precipitation distribution and due to presence of a complex drainage system.

However, the hydraulic effectiveness of some flood mitigation system as polders, short-term change in the river morphology and environmental conditions depends not only on the extreme flood event, but also on the characteristics of seasonal flood flow. For example, once the storage capacity of the retention area (polder, wetland) is at its limit, subsequent flood of lower magnitude can damage more severe than one with the extreme flood. The river bank erosion in alluvial flood plain depends on the number of flood events and their characteristics. The bank erosion for these cases are mainly due to liquefaction of the bank materials and mostly found during the recession part of the flood waves due to water level variation of the ground water and river stage (Kokoty et al., 2005).

In this chapter, a synthetic seasonal hydrograph generation technique is proposed for the Brahmaputra river basin in India. The characteristics of the flood hydrographs are dependent on several variables e.g., time to peak, base period, time of occurrence of the flood waves, etc. To find out the distribution of the variables, probability distribution functions namely normal, lognormal, exponential, gamma, Pearson type-III, extreme value type-I have been employed. Based on the best distribution function and relation to total seasonal rainfall, flood characteristics and monsoonal response are generated. A superimposed approach is finally followed for the synthetic hydrograph generation.

4.2 Methodology

4.2.1 Historic Stage Record Analysis

Typical stage hydrograph of a river with large catchment size during a monsoon period (rainy season) is depicted in Fig. 4.1. During the monsoon period, number of flood waves can be noticed in the hydrograph due to several clustered storm events or antecedent wet condition. Among them, a few flood waves with distinct characteristics can be identified. The hydrograph characteristics are found to be similar over the years and can be separated into two components, namely flood waves and monsoonal (seasonal) response. The flood waves are the fast response due to occurrence of the severe and clustered storm event in the river basin. The other component is the part of the average response of monsoonal rainfall system. As the monsoon progresses part of the river basin becomes saturated and its runoff response then become faster (Chattopadhyay and Dutta, 2006). This approximation helps to identify the duration of the individual flood events. The period in between the monsoon onset to the end of monsoon can be identified as shown in Fig. 4.1.

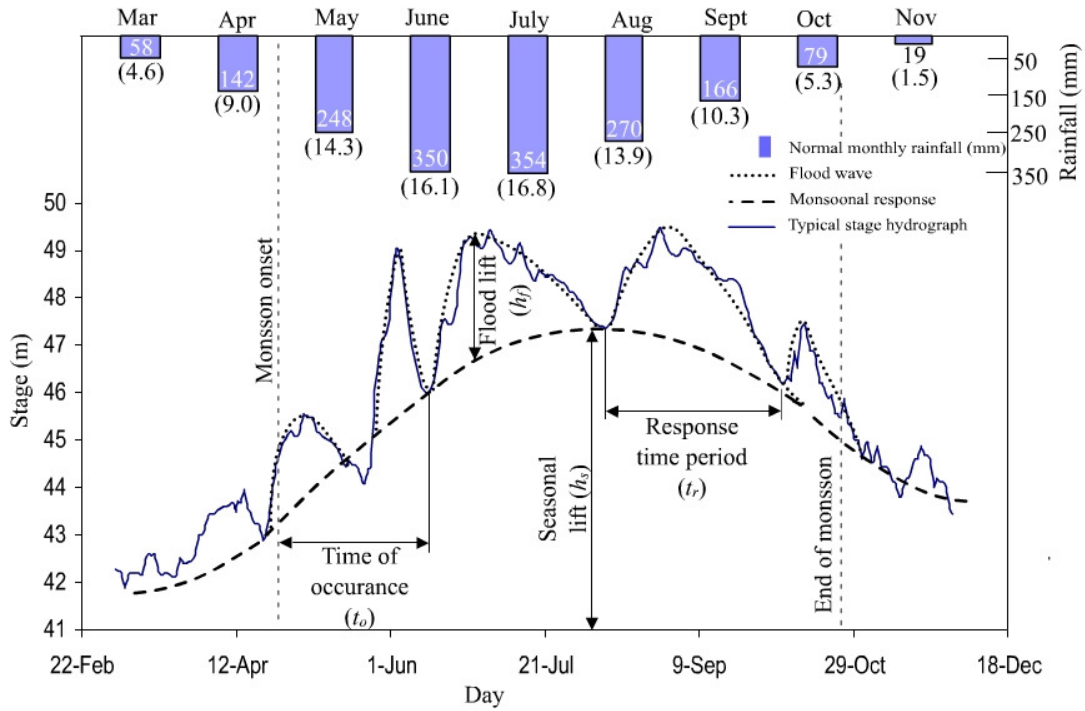


Fig. 4.1. Conceptual diagram of the different components of the synthetic hydrograph

(Numerals within the brackets indicate monthly mean rainy days)

Next, the monsoonal response is approximated using Maxwell distribution (Spiegel, 1992). The distribution suggested by Maxwell (1960) gives better approximation of the flood wave as it considers the correlations with peak flows with length ratios (Howard, 1990). The generalised Maxwell distribution function used for this approximation is given by the following expression:

$$h(t) = h_b + h_s \left[\frac{t}{k_s} \exp \left(1 - \frac{t}{k_s} \right) \right]^{r_s} \quad (4.1)$$

where, h_b the initial stage, h_s the seasonal lift of monsoonal response, t the base time, k_s and r_s are the fitting parameters depending on the time to peak and base period of the seasonal response, respectively.

The individual flood wave characteristics can be approximated by using Maxwell distribution:

$$h^i(t) = h_f^i \left[\frac{t}{k_f^i} \exp \left(1 - \frac{t}{k_f^i} \right) \right]^{r_f^i} \quad \text{when, } t_o^i \leq t \leq (t_o^i + t_r) \quad (4.2)$$

where, $h^i(t)$ the height of lift of the i^{th} flood wave at time t , h_f the maximum lift of the i^{th} flood wave, k_f^i and r_f^i are the fitting parameters, t_o the time of occurrence and t_r the response time period

For n numbers of flood waves and superimposing them with the monsoonal response, we get:

$$h(t) = h_b + h_s \left[\frac{t}{k_s} \exp \left(1 - \frac{t}{k_s} \right) \right]^{r_s} + \sum_{i=1}^n h_f^i \left[\frac{t}{k_f^i} \exp \left(1 - \frac{t}{k_f^i} \right) \right]^{r_f^i} \quad (4.3)$$

The lift of monsoonal response can be computed from its relation with total seasonal rainfall (R_s) at the basin level. Then we can write:

$$h_s = f_1(R_s) \quad (4.4)$$

Similarly, annual maximum flood lift, (h_f^m) can be computed using frequency analysis. For a given return period (T), it becomes:

$$h_f^m = f_2(T) \quad (4.5)$$

For simplification of equation (4.3), the fitting parameters, k_f^i and r_f^i can be substituted with the most frequently occurring values, k_f and r_f . Finally, the equation (4.6) can be written after substitution of h_s and h_f^m :

$$h(t) = h_b + f_1(R_s) \left[\frac{t}{k_s} \exp \left(1 - \frac{t}{k_s} \right) \right]^{r_s} + f_2(T) \left[\frac{t}{k_f} \exp \left(1 - \frac{t}{k_f} \right) \right]^{r_f} + \sum_{i=1}^{n-1} h_f^i \left[\frac{t}{k_f} \exp \left(1 - \frac{t}{k_f} \right) \right]^{r_f} \quad (4.6)$$

A brief description of the different parameters used in this equation is presented in Table 4.1.

Two random variables listed in Table 4.1 are computed by random number generation techniques which require the mean and standard deviation of a random variable and its best probability distribution. Random number generation for different probability distribution is well described by Wang (2006). The required parameters in the random number generation technique are computed from characteristics of flood waves of the historical stage records. For this, the individual flood waves are fitted with the Maxwell distribution function by minimising the summation of absolute errors using Microsoft excel optimisation routine, which follows generalized reduced gradient method. Some minor fluctuations if required are filtered out from the hydrograph to reduce the absolute error. Fig. 4.2 shows a flow chart for the stepwise analyses and generation technique followed in the present study.

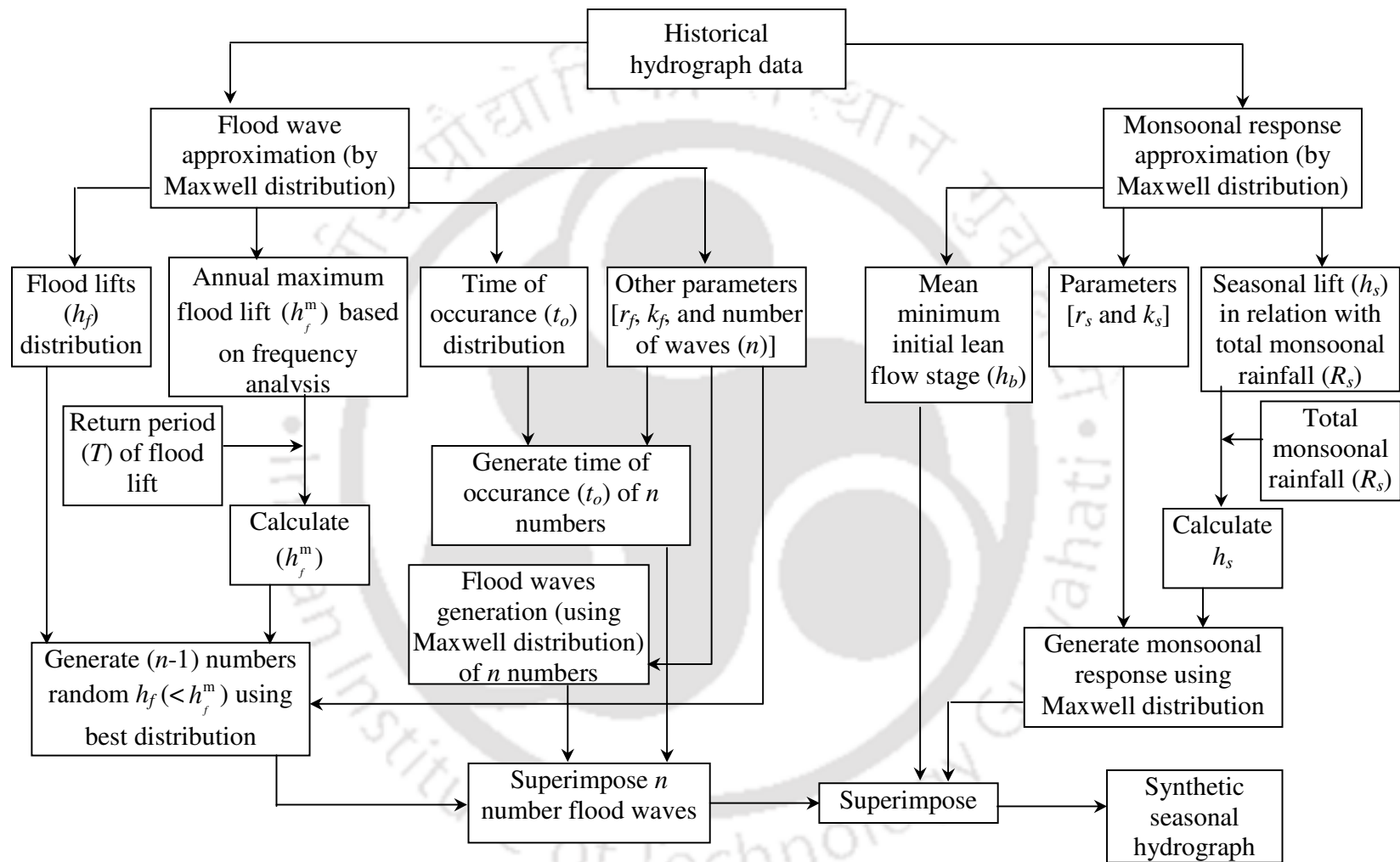


Fig. 4.2 Conceptual flow chart for synthetic hydrograph generation

Table 4.1 Details of different parameters used in the synthetic generation

Variables	Computation procedure	Nature
Initial stage (h_b)	Determined from mean minimum stage just before monsoon onset	Deterministic
Seasonal lift (h_s)	Determined from the relationship with the total monsoonal rainfall (R_s) over the entire basin	Deterministic
r_s and k_s	Used the median value from the historical data	Deterministic
Time period for monsoonal lift (t)	Considered the total period of monsoon	Deterministic
Annual maximum flood lift (h_f^m)	Computed using frequency analysis for a given return period (T)	Deterministic
Subsequent lifts of flood waves (h_i)	Generated based on their distribution function	Probabilistic
Time of occurrence of flood waves (t_o)	Generated based on their distribution function	Probabilistic
r_f and k_f	Used the median value from the historical data	Deterministic

4.3 Data Used

Daily stage records during monsoon period (May to October) for 20 years from 1978 to 2004, with one or two year data gap in between, were available from the river gauging station at Guwahati. However, only 13 year stage records were obtained for Tezpur within the same period. Data scarcity is one of the biggest challenges in the developing country and due to this problem no gauge records are available before 1978. A typical stage hydrograph obtained from the gauging station at Guwahati is shown in Figs. 3.14 and 4.1. The study of the stage hydrograph clearly shows average 9 flood waves every year.

Monthly rainfall data at 0.5 degree grid were collected from an open source of NASA (trmm.gsfc.nasa.gov/) for the whole Brahmaputra river basin for the study period. The

average rainfall over the basin for each monsoon season is estimated to find their relationship with the monsoonal lift (h_s).

4.4 Results and Discussion

The monsoonal stage hydrograph records from the two gauging stations is analysed to estimate the required parameters for the hydrograph generation. The flood waves and the monsoonal response have been separated, each fitted with Maxwell distribution and based on the analysis, Table 4.2a and 4.2b show the ranges of the variables for both gauging stations. It has been found from the stage hydrograph analysis, average nine flood waves occur every year. Further analyses of the parameters including the generation of the synthetic hydrograph have been carried out using MATLAB (ver. 7.0) programming. Different probability distribution functions were tested to find the best for the cumulative probability distribution of the flood lift values (h_f^m) for the two gauging sites (Chow et al., 1988). The Chi-square test shows that h_f^m closely follows the lognormal distribution for both the gauging stations (Fig. 4.3a and b). Similar results were also reported by Bora (2004). However, the time of occurrence of the flood events closely fits with the normal distribution (Fig. 4.4) except very low value (less than 12 days). The lift of monsoonal response (h_s) mainly depends on the total monsoonal rainfall as already discussed. Result indicates a logarithmic relationship ($r^2 = 0.84$) between the monsoonal lift and the total rainfall (Fig. 4.5). Interestingly, this figure indicates that with same average rainfall the monsoonal lift at Guwahati gauging station is higher than Tezpur gauging station. This is because Guwahati is situated at downstream of Tezpur and thus the contributing catchment area is more than that of Tezpur.

Using these analyses, the annual maximum flood lifts with a particular return period for the flood waves has been generated using the MATLAB programme. Table 4.3 shows the

maximum flood lift for different return periods at the gauging stations, based on the frequency analysis of historic data. Fig. 4.6 shows the stage-cumulative probability curves with 95% confidence limits for both the gauging stations.

The analysis of the data presented in Table 4.2a indicates that there is very less difference between the median values of the magnitude of flood wave. This represents that, the attenuation of the flood wave is negligible from Tezpur station to Guwahati station. However, generated flood lifts based on the return period indicates that flood lifts at downstream gauging station (Guwahati) is higher than the upstream gauging station until 100 year return period flood. At 100 year return period flood, prediction shows the same flood lift at both the station. This is due to the error of approximation in the fitting function and as the error in estimation is less compared to the total lift (maximum error 4.8%), so this estimation can be accepted. Table 4.2b indicate that stage rise due to monsoonal response is more in Guwahati than that of Tezpur station.

The developed MATLAB programme finally generates the synthetic stage hydrograph (Fig. 4.7a) based on a total monsoonal rainfall of 2707 mm and a particular return period of 6 year for maximum flood wave with total nine number of flood events. Similar relationship between the total rainfall and flood lift is found in Guwahati station in 1988 (Fig. 4.7b). It clearly indicates that there is difference in the time of occurrence of flood waves. But the generated hydrograph accurately represents the monsoonal lift and the flood wave nature. However, minor fluctuations that can be naturally observed in the stage record, which have not been captured in the present approach.

Table 4.2a Range of parameters for flood waves

Parameters		Guwahati			Tezpur		
		range	avg. (std dev)	median	range	avg. (std dev)	median
Flood wave	h_f^m (m)	0.47-3.13	1.52 (0.63)	1.33	0.65-3.35	1.63 (0.62)	1.55
	r_f	0.10-14.0	3.86 (2.64)	3.2	0.45-25	4.71 (4.35)	3.3
	k_f	0.25-21.5	6.17 (4.15)	5.6	1-21	8.00 (5.28)	7
	Time of occurrence (T_i)(days)	15-165	82.73 (42.03)	--	15-160	79.12 (41.96)	--
	Response time period (t) (days)	5-50	15.7 (8.88)	20.1	7-58	19.75 (11.71)	20.1

Table 4.2b Range of parameters for monsoonal response

Parameters		Guwahati			Tezpur		
		range	avg. (std dev)	median	range	avg. (std dev)	median
Monsoonal response	h_s (m)	2.39-3.60	3.08 (0.33)	3.05	1.6-2.80	2.26 (0.31)	2.25
	r_s	1.00-5.00	2.27 (1.16)	2.1	0.65-10.0	2.29 (2.44)	1.38
	k_s	58.0-100.0	76.10 (11.44)	75	33.5-97.0	68.08 (17.12)	67

4.5 Evaluation of Synthetic Hydrograph

A number of hydrographs can be generated based on the method discussed above. The evaluation method of the generated hydrograph depends on the purpose of its usage. For river bank erosion dominated by the liquefaction and seepage of the river bank materials (Fox et al. 2006), daily change of the river water level (stage) plays the major role. In this present study, the generated stage hydrographs are evaluated in two ways. First, the synthetic hydrographs are compared with the observed stage data for their cumulative frequency function. Secondly, the relative frequency of the daily stage change (rise/fall) is evaluated with the observed data. For this, three stage hydrographs of three different years (1998, 1988 and 1978) have been chosen, one with lower return period (2 year) and the other with medium return period (6 year) and the last with higher return period (10 year). From that, the return period for annual maximum lift of flood wave is calculated. Twenty numbers of stage hydrographs have been generated for each station and for three years. Chi-square test has been applied to estimate the limit of error for cumulative distribution of stages and relative frequency of the rate of change of stages.

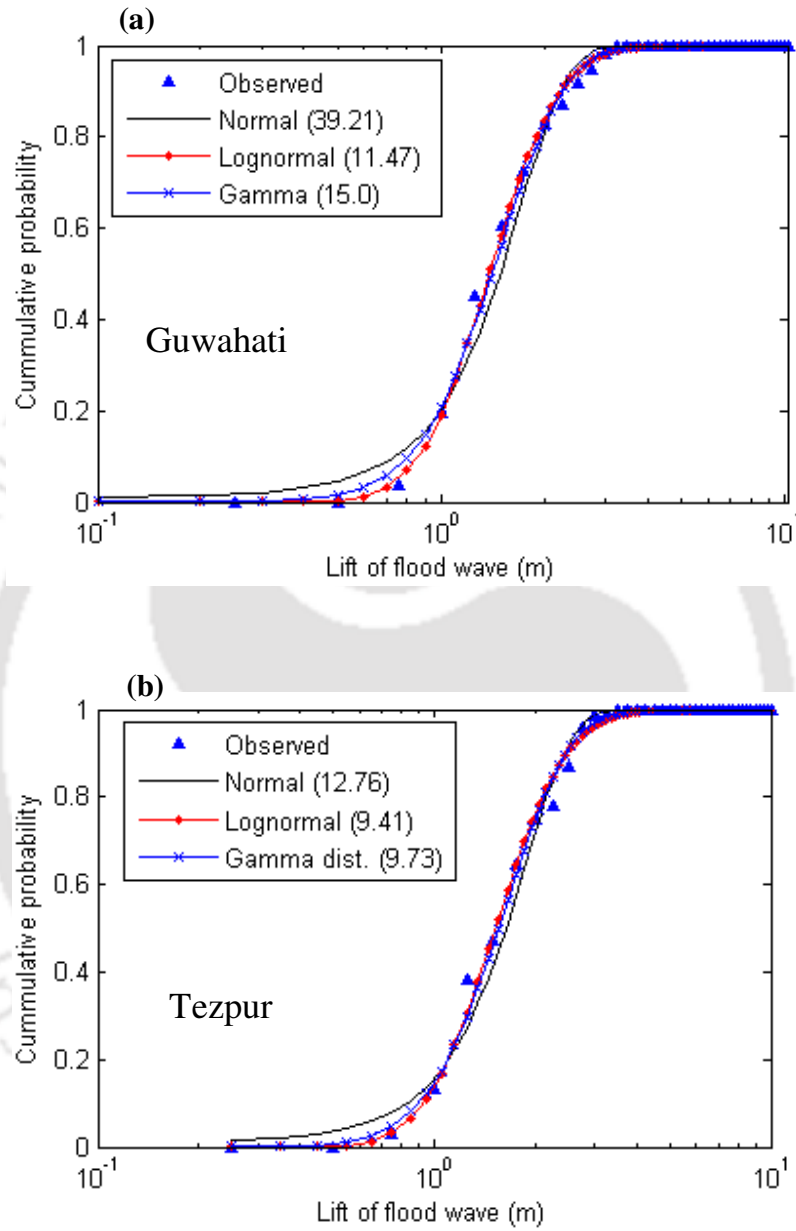


Fig. 4.3 Probability distribution functions for event-wise flood lift, a) Guwahati station and b) Tezpur station (Numerals within the brackets indicate the respective Chi-square value)

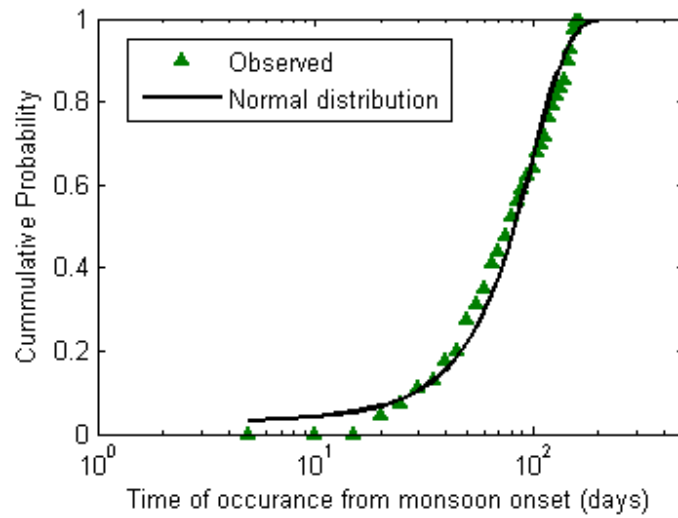


Fig. 4.4 Probability distribution function for the time of occurrence of flood wave for Guwahati gauging station

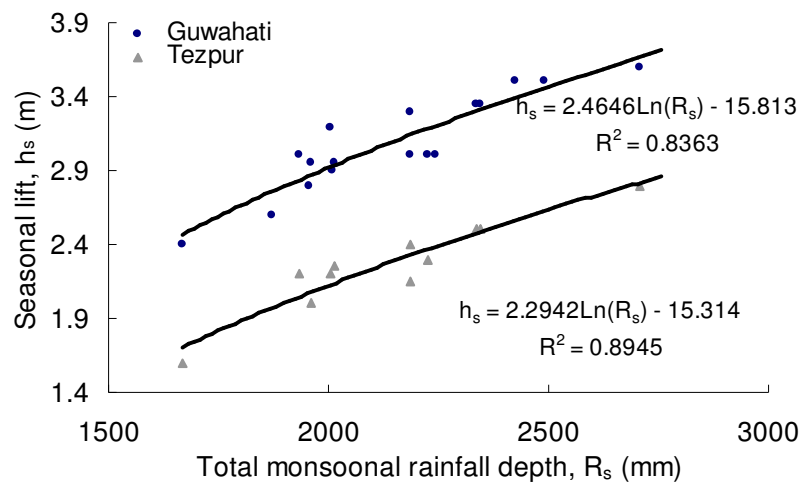


Fig. 4.5 Relationship between the total monsoonal rainfall and monsoonal lift for both the gauging stations

Table 4.3 Probable lift for flood waves

Return period (yrs)	Gauging station	
	Guwahati	Tezpur
2	2.39	2.28
5	2.87	2.80
10	3.11	3.07
15	3.24	3.20
20	3.32	3.29
30	3.42	3.40
50	3.54	3.53
100	3.68	3.68

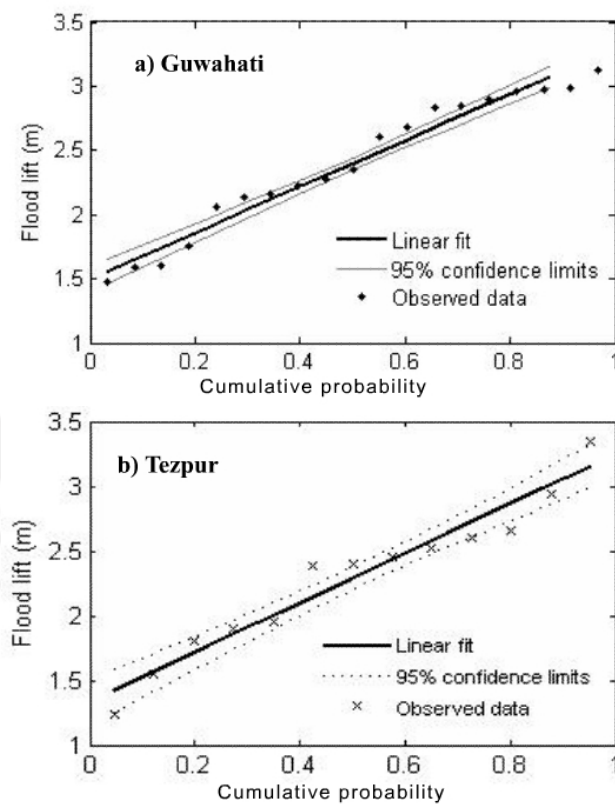


Fig. 4.6. Stage-cumulative probability curves with confidence limits (a) Guwahati and (b) Tezpur station.

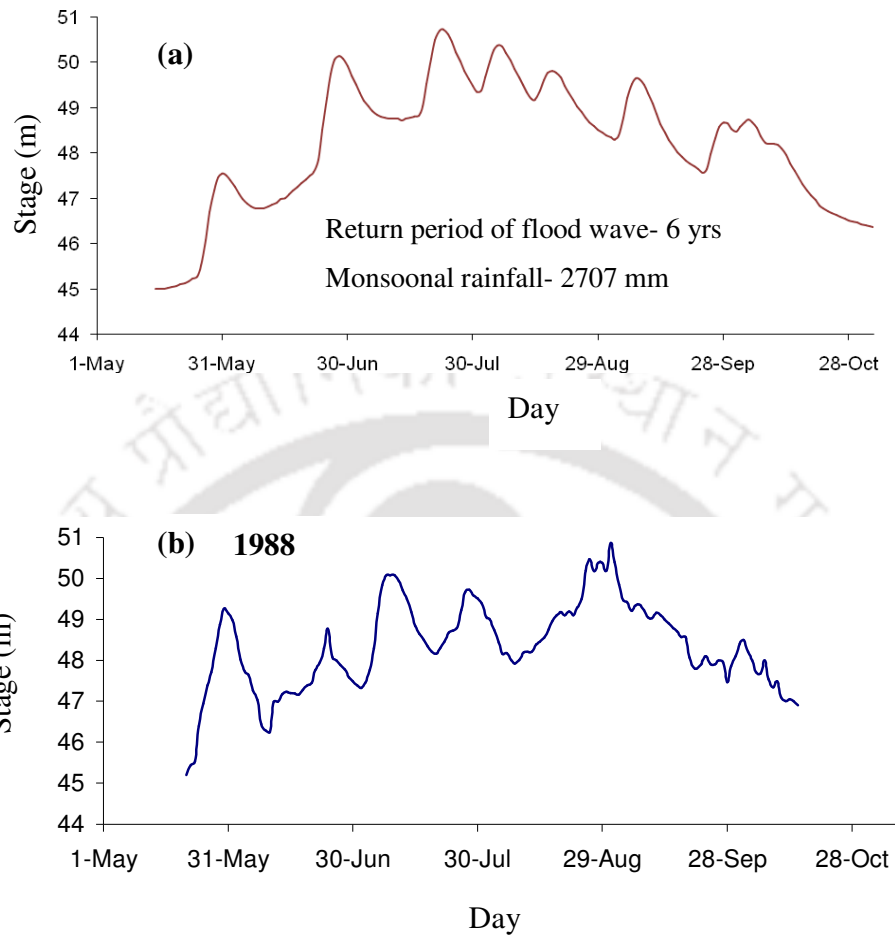


Fig. 4.7. a) Typical synthetically generated stage hydrograph for Guwahati station in 1988
(b) same year observed daily stage hydrograph

Table 4.4 Flood lift and return period of flood waves considered for the evaluation

Flood year	Average total rainfall (mm)	Approximate return period of flood wave (yrs)	Guwahati	Tezpur
			Flood lift (m)	Flood lift (m)
1998	2005	2	2.13	2.45
1988	2707	6	2.85	2.95
1978	1667	10	3.13	3.10

Table 4.4 shows the variables for the two gauging stations for selected flood years. Form the fitted relation of the stage with return period, there is minor difference in the return periods, which has been solved by considering approximate return period.

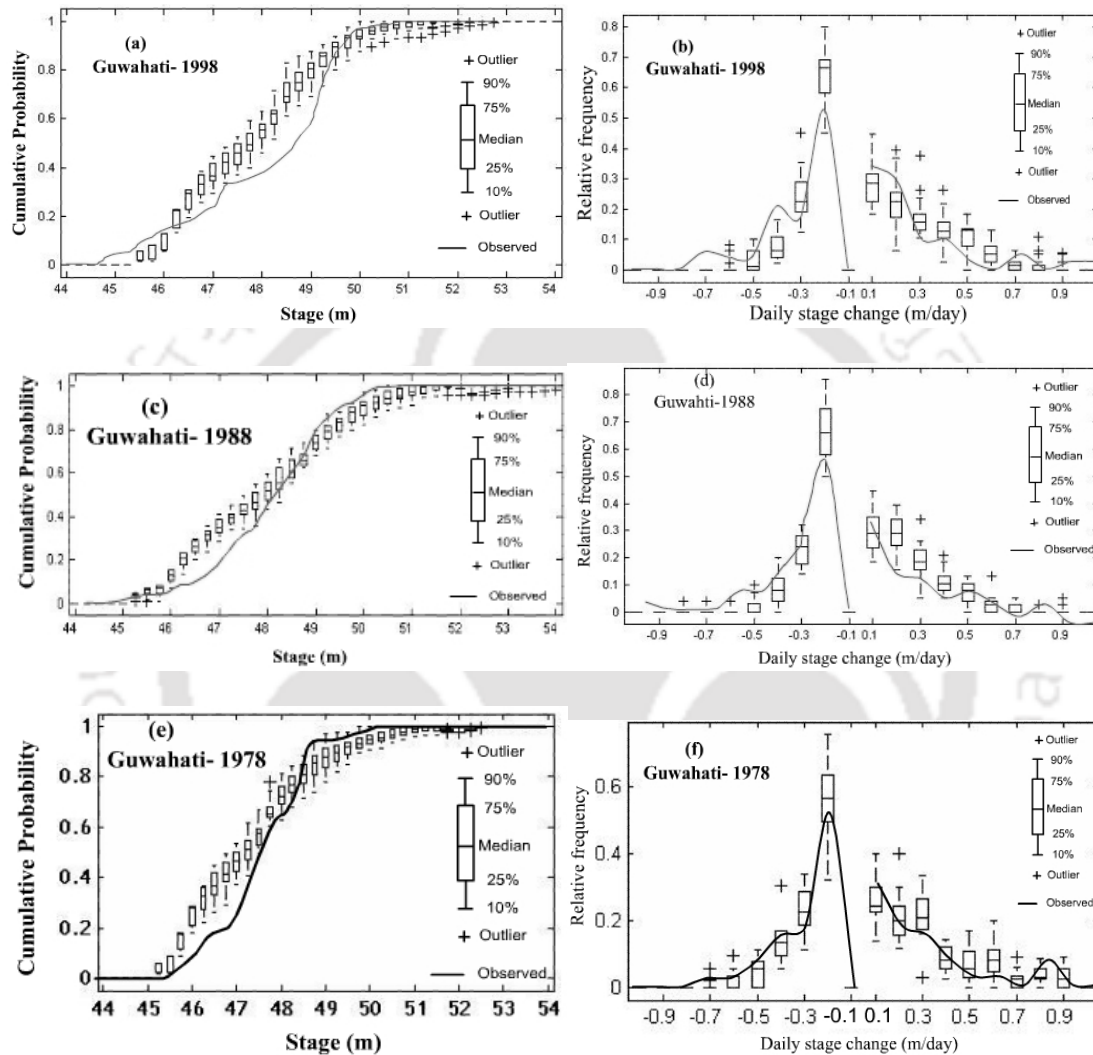


Fig. 4.8. Evaluation of the proposed method for Guwahati station. a) Cumulative frequency function for daily stage level in 1998, b) Relative frequency function for daily stage change in 1998, c) Cumulative frequency function for stage level in 1988, d) Relative frequency function for daily stage change in 1988. Positive sign in daily stage change indicates rise of stage, else fall

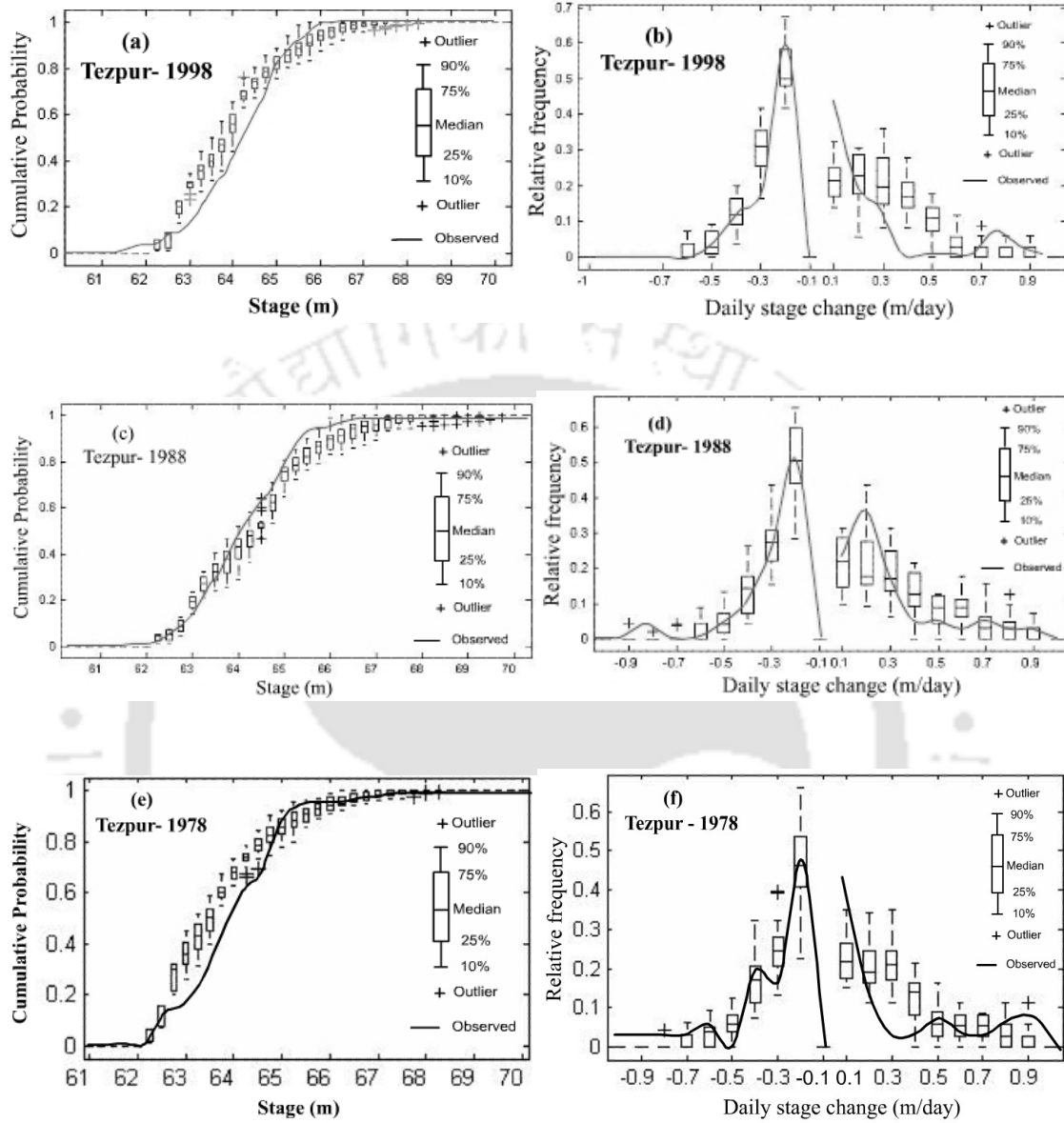


Fig. 4.9. Evaluation of the proposed method for Tezpur gauging station. a) Cumulative frequency function for stage level in 1998, b) Relative frequency function for daily stage change in 1998, c) Cumulative frequency function for stage level in 1988 d) Relative frequency function for daily stage change in 1988. Positive sign in daily stage change indicates rise of stage, else fall

Using the same return periods with random nature of time of occurrence of flood wave and subsequent lifts, 20 stage hydrographs for Guwahati station (Fig. 4.8) and Tezpur station (Fig. 4.9), are generated. The chi-square value is estimated to be 40.96, 34.31 and 44.28, respectively for year 1998, 1988 and 1978 for Guwahati station, which is well below the limiting value (112.3) for acceptance with 78 degree of freedom and 99% confidence limit (Kreyszig, 2004).

Similarly, for Tezpur, the chi-square value is 31.90 and 14.02, respectively while the limiting value with 99% confidence limit and 38 degree of freedom is 63.7. Also acceptable visual comparison between the ranges of cumulative probability of the stages with the observed one can be found in Figs. 4.8a, c, e and 4.9a, c, e.

Daily stage variations at rising and receding phases are also evaluated in this study. In the observed stage record for the large river basin during the non-flood event period also minor fluctuation can be observed. These minor fluctuations in the river stages are due to various hydrological, meteorological and geological factors. But the approximated distribution is a smooth and nearly flat during the non-flood event period. So, considerably higher relative frequency at near zero value of the rate of change of stage can be obtained. However, the observed fluctuations are small and do not have significant effect on hydraulic and morphologic changes for a river. To overcome this problem, a threshold limit (± 0.1 m/day) is selected, beyond which the rate of change of stage will cause a significant impact on the hydraulic and morphologic features of river. Figs. 4.8b and 4.8d and 4.8f show the relative frequency distribution of the simulated and observed hydrograph at Guwahati gauging station for the year 1998 and 1988 and 1978, respectively. A box plot diagram has been used to provide the ranges of the relative frequency of simulated twenty hydrographs.

Similarly, Fig. 4.9b and 4.9d and 4.9f show the relative distribution for Tezpur station for the year 1998, 1988 and 1978, respectively. The relative frequency of the rising and the receding phase are estimated separately for the synthetic and observed hydrographs. This indicates that daily stage change for rising and receding phase match well beyond a minimum threshold of ± 0.1 m/day.

4.6 Conclusions

Analysis of stage records during the monsoon period has been carried out for two gauging stations in the middle reach of the Brahmaputra River, India. This analysis emphasises on flood wave characteristics as lift of the flood wave (h), time of occurrence (T), response time period (t), number of flood waves (n), monsoonal lift (H) etc. Each seasonal stage hydrograph has been separated into two components: flood waves and the monsoonal response. Using Maxwell distribution function, individual flood waves and monsoonal response have been approximated and the lift of flood, their time of occurrences were determined and fitted with suitable distribution functions. Particular monsoonal lift in a year was determined based on the relationship between the total monsoonal rainfall and the monsoonal lift during the study period. Other parameters have been analysed to find out their range and median values. Based on this analysis, the synthetic stage hydrograph has been generated for a given return period of the flood wave. Lifts of the selected events of flood waves and their time of occurrences were generated randomly following the best fitted distribution. In this study, mean minimum river stage just before the monsoon has been considered as the initial stage. Finally, the initial river stage, the flood waves, their time of occurrences and the monsoonal lift are superimposed to generate the synthetic stage hydrograph.

The present method of generation of synthetic hydrograph can capture both maximum stage and the rate of change of river stage. The generated hydrographs has been evaluated for both the gauging stations for three years 1998, 1988 and 1978. The proposed method gives satisfactory results not only for three years which comprises low, medium and high flood lift, but also for two gauging the stations. The relative frequency of receding phase of daily stage change is computed more accurately than that of rising phase. However, minor fluctuations of stage can not be captured accurately by the present method, so up to a minimum value of ± 0.1 m/day has been neglected for this present study. The present approach can be refined for obtaining more accurate synthetic stage hydrograph if stage records for longer period are available. In the rivers with lesser number of gauging stations and insufficient stage records, this synthetic seasonal hydrograph will be very helpful for various river engineering purposes.

CHAPTER 5 ERODIBILITY OF THE FINE BANK SOILS

5.1 Introduction

The erosion of a composite river bank critically depends on the erodibility of its fine soils, as the fine soil has the higher resistance against erosion. So, for estimation of the bank erosion in a composite river bank, it is important to determine the critical shear stress and erodibility coefficients of the bank soil and their spatial distribution. Assessing erosion by flowing water for cohesive fine soils in a composite river bank is complex (Hanson and Simon, 2001). Nevertheless, significant bank erosion can be saved by assessing spatio-temporal variation of erodibility of the river bank. In this chapter, erodibility parameters of the river bank of Brahmaputra in India are estimated through *in situ* submerged jet tests. The different empirical methods for determine the soil erodibility parameters are discussed here and the results are compared with the *in situ* measurements. The apparatus used for the tests and the experimental procedures are explained in details. The spatial variations of the soil erodibility and their controlling variables are thoroughly discussed here. The significance of spatial and layer wise distribution of the erodibility parameters was tested through analysis of variance (ANOVA).

5.2 Erodibility of Fine Soil

Critical shear stress (τ_c) is the limiting shear stress developed by flowing water, above which, soil erosion will be initiated. If the developed shear stress is less than the critical shear stress, the erosion rate is assumed to be negligible (Osman and Thorne, 1988; Hanson, 1990a; Hanson et al., 2002). The erosion rate for the fine soils in a river

bed or river bank is generally assumed to be proportional to the excess shear stress as given below (Hanson and Cook, 1997; Hanson, 1990a, 1990b):

$$\varepsilon = k_d (\tau_a - \tau_c)^a \quad (5.1)$$

where, ε is the rate of erosion (m s^{-1}), k_d the erodibility coefficient ($\text{m}^3(\text{N-s})^{-1}$), τ_a the developed shear stress at the soil boundary (Pa), τ_c the critical shear stress (Pa), and a the exponent generally considered to be 1.

Many research studies have been conducted on erodibility of the cohesive soils after investigating the various soil properties such as soil moisture content, soil structures, clay content, clay mineralogy and chemical composition of river and pore water (Grissinger, 1982). Due to the influence of several factors, the erodibility parameters are difficult to quantify (Grissinger, 1982). Different approaches for estimating critical shear stress include flume studies (circular and straight), submerged jet test, rotating cylinder test, water tunnel, estimation based on particle sizes of soil: all have produce important results (Hanson and Simon, 2001; Clark and Wynn, 2007). Shields' diagram is commonly used to estimate the critical shear stress for uniform, non-cohesive soils based on the particle size (Shields, 1936; Hann et al., 1994). Later Shields' diagram had been modified to estimate the cohesive soils also (Temple and Hanson, 1994). However, this modification of the diagram is not based on laboratory or field data.

Several studies for understanding the soil erodibility have been carried out by using open channel flow (Dunn, 1959; Smerdon and Beasley, 1961; Kamphuis and Hall, 1983). The critical shear stress was determined by visual inspection for different magnitudes of developed shear stress. However, the point of failure at critical shear stress is difficult to assess and therefore, partly subjective (Hanson et al., 1999; Clark and

Wynn, 2007). Hanson and Cook (1997) successfully provided the solution graphically, by plotting the erosion rate versus shear stress.

The study by Julian and Torres (2006) concluded that four possible flow properties control the hydraulic erosion rates of cohesive river banks: magnitude (Arulanandan et al., 1980), duration (Wolman, 1959; Knighton, 1973), event peak (Hooke, 1979), and variability (Knighton, 1973).

Moreover, in a composite river bank, sizes of the soil particles vary to a great extent. The variation of the soil particle size, especially the %silt-clay content has been identified as the key factor to fluvial erosion and mass failure (Schumm, 1960a,b). However, the major limitation of the submerged jet tests is that it does not assess the total bank erosion; however, this test can be used to assess the soil erodibility of fine soils from the composite banks of the alluvial rivers. The bank erosion processes with measured soil erodibility can be modeled for estimating the total bank erosion.

There is lack of study to quantify the variation of the *in-situ* erodibility for fine soils from composite banks of an alluvial river. Moreover, the knowledge of variations of erodibility parameters at various layers at a specific site is essential for understanding the bank erosion processes. The objective of this chapter is to obtain the variation of erodibility parameters (τ_c and k_d) of the river, using *in-situ* submerged jet test methods and analyze their dependency on locations and various stratified layers along the composite banks of the Brahmaputra river, India.

5.3. Methodology

5.3.1 Measurement of Erodibility Parameters

Critical shear stress (τ_c) and the erodibility coefficients (k_d) are measured *in situ* using submerged jet test apparatus (Hanson, 1990b). The detail of the apparatus (Fig. 5.1) described by Hanson and Cook (2004) is given in the Appendix-A, and hence are not discussed here. Before testing, it should be ensured that the layer is composed of silt or clay soils; because the tests cannot be performed for sand. The top of the test layer should be cleaned before the placing of the jet test device.

A total number of fifty-eight submerged jet tests were conducted along the river Brahmaputra. Sixteen submerged jet tests were performed in Jamuguri (876 km from the Bay of Bengal), eighteen tests near Dolgobinda (657 km from the Bay of Bengal), seven tests at Amingaon (652 km from the Bay of Bengal), twelve tests at Nalbari (613 km from the Bay of Bengal) and the remaining tests at the hilly facet (red soil) near north bank of Guwahati (656 km from the Bay of Bengal) [refer Fig. 3.1].

5.3.2 Test Procedure

The detail procedure of the submerged jet test as described by Hanson and Cook (2004) are discussed here in brief.

1. Prior to the test, the site has to be cleaned and leveled suitable for the test. The preferred surface is near flat, although at the slope surface measures are to be taken to avoid overtopping.
2. The steel submergence tank is to be inserted 76 mm in to the ground to avoid the piping with the filled water. Jet tube and the point gage assembly are then attached along with the lid through the side clamps.

3. Head tank and the pipes are connected to the jet tubes and arrangements are made for assuring the constant and uniform supply of the river water.
4. A deflector plate is used at the bottom of the jet nozzle to fill the submergence tank with water, without causing scour at the soil surface.
5. Initial readings at a reference, at bottom of the jet tube and the soil surface have been taken using the point gage. Pressure gage reading at dial gage are also noted. Minimum 4 cm initial gap between the bottom of the jet tube and the soil surface is recommended by Hanson and Cook (2004).
6. The deflector plate is then moved out of the way of the nozzle and the time of test initiation is noted. After every 5 minutes the scour reading is taken using the point gage. The test is to be run normally for 45 minutes (Wynn, 2004). But in case of highly erodible soil, readings every 1 minutes interval is noted. In the later case, test may run for as low as 15 minutes.

Tractive stress distribution along the scour surface is not uniform, but it is zero exactly at the center. Then it increases radially from the center reaching to a peak value and then again decreases radially (Fig. 5.2) (Hanson et al., 1990).

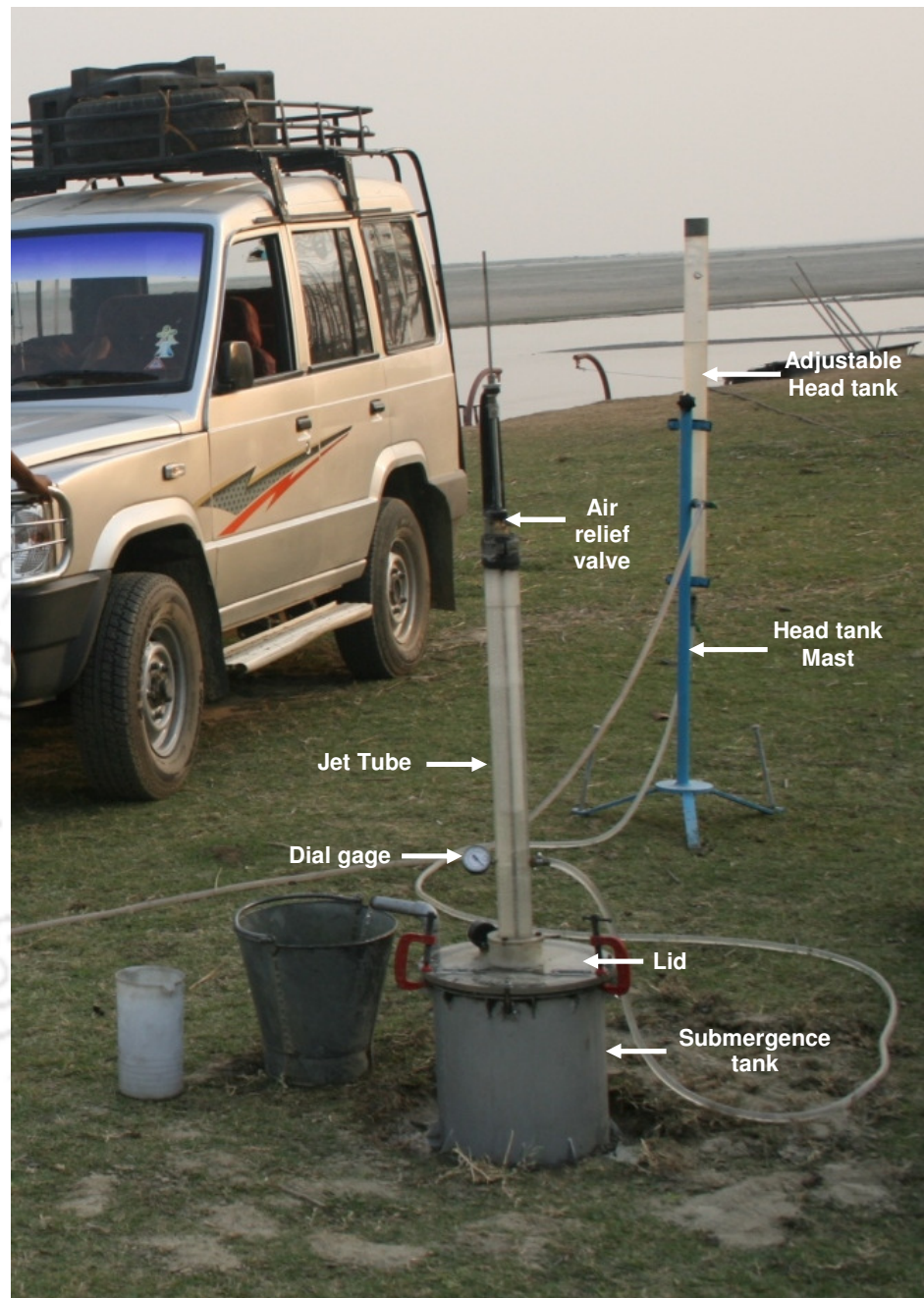


Fig.5.1. Submerged jet test apparatus with head water tank.

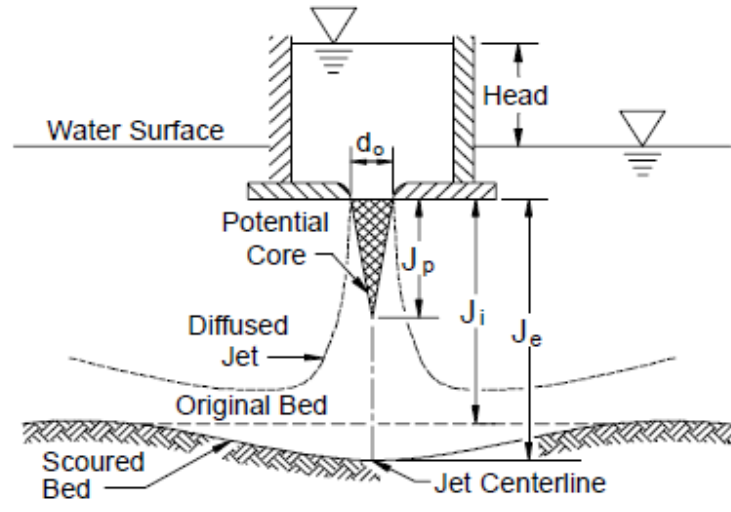


Fig. 5.2. Schematic diagram for submerged jet test defining the parameters (after Hanson and Cook, 2004)

It has been assumed that the peak stress causes the maximum scour beneath the jet. The initial shear stress (τ_i), in the jet impingement zone for a particular test set-up can be estimated using the following equations (Hanson and Cook, 2004):

$$\tau_i = \tau_0 \left(\frac{J_p}{J_i} \right)^2 \quad (5.2)$$

$$J_p = C_d \times d_0 \quad (5.3)$$

$$\tau_0 = C_f \rho U_0^2 \quad (5.4)$$

$$U_0 = \sqrt{2gh} \quad (5.5)$$

where, τ_i is the initial peak boundary stress prior to scour, τ_0 the maximum shear stress due to jet velocity at the nozzle, J_p the potential core length, J_i the initial jet orifice height, C_d the diffusion constant taken as 6.3, d_0 the nozzle diameter, C_f the coefficient

of friction taken as 0.00416, U_0 the velocity at the nozzle, h the differential water head measurement.

Critical shear stress (τ_c) is estimated based on the equilibrium scour depth (J_e) as described by (Blaisdell et al., 1981):

$$\log\left(\frac{U_0 t}{d_0}\right) = \left[\left(\log\left(\frac{J}{d_0}\right) - \log\left(\frac{U_0 t}{d_0}\right) \right) - \log\left(\frac{J_e}{d_0}\right) \right]^2 - A_1^2 \right]^{0.5} \quad (5.6)$$

or, in simple it can be written as:

$$x = [(f - f_0)^2 - A_1^2]^{0.5} \quad (5.6a)$$

where, A_1 is the value for the semi-transverse and semi-conjugate axis of the hyperbola, U_0 the velocity of the jet at the origin, t the time of data reading. A spreadsheet program can be developed to minimize the sum of the error between the observed and the functionally fitting values. Once J_e is determined then the τ_c can be determined by applying the following equation:

$$\tau_c = \tau_0 \left(\frac{J_p}{J_e} \right)^2 \quad (5.7)$$

The erodibility coefficient (k_d) is estimated based on the scour data, time required, already estimated τ_c and the dimensionless time function:

$$t_m = T_r \left[0.5 \ln \left(\frac{1+J^*}{1-J^*} \right) - J^* - 0.5 \ln \left(\frac{1+J_i^*}{1-J_i^*} \right) + J_i^* \right] \quad (5.8)$$

where, t_m is the measured time, T_r the reference time equivalent to $J_e / (k_d \tau_c)$, J^* is the dimensionless scour term equals to J/J_e , J_i^* the dimensionless scour term at J_i/J_e , J the distance from the nozzle to the centerline depth of scour, J_i the initial distance from the

nozzle to the soil surface. Dimensionless time (T^*) can be defined as the ratio measured time to the reference time ($T^* = t_m / T_r$). A spreadsheet routine program minimizes the sum of the errors of the observed value of t_m from the functionally determined values. Fig. 5.3 shows the hyperbolic fitting of the *in situ* jet test results for determining the critical shear stress as described through Equation 5.6a. Fitting of the scour function of dimensionless time and depth to determined the erodibility coefficient as described by Blaisdell (1981) is depicted in Fig. 5.4. The experimental set up of the submerged jet test and various steps during the experiments are shown in Fig. 5.5.

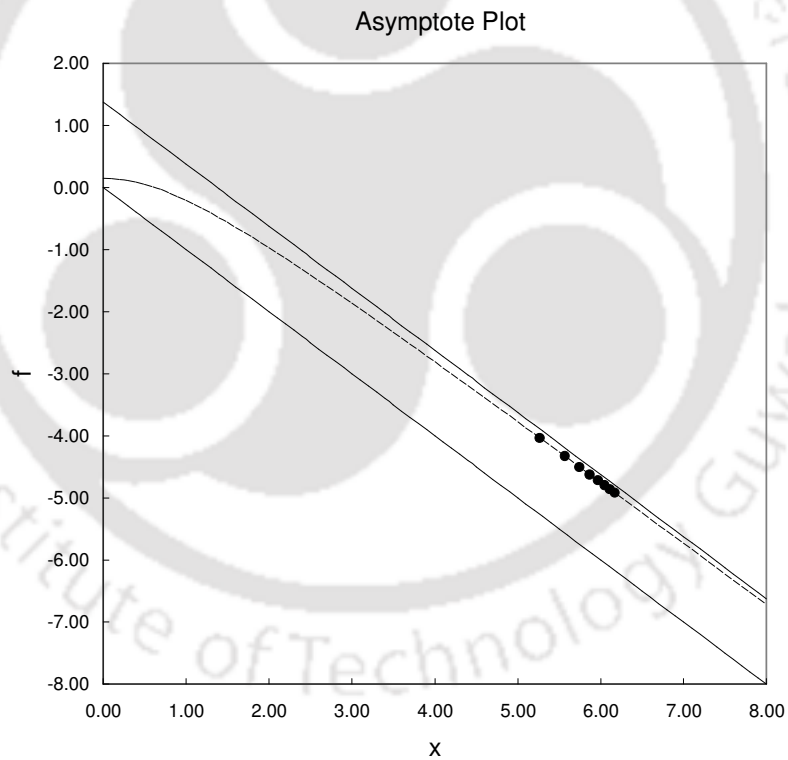


Fig. 5.3. Hyperbolic curve fitting of jet test results for determining the critical shear stress.

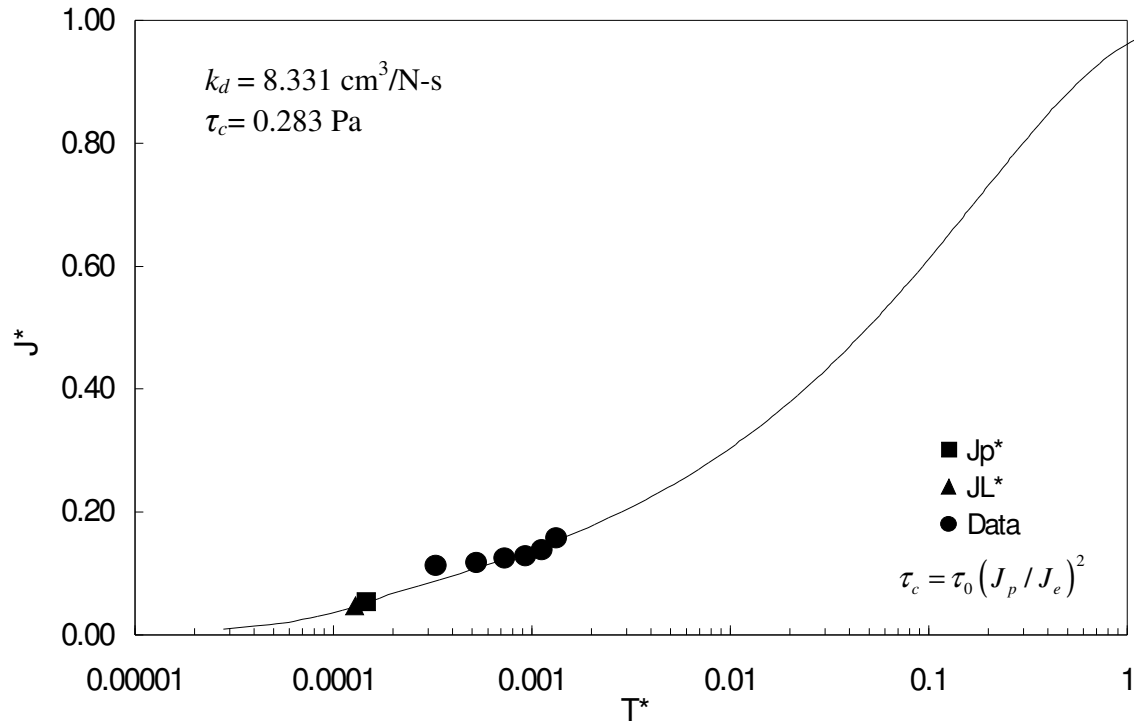


Fig. 5.4. Fitting scour function of dimensionless time and depth for determining the erodibility coefficient (k_d) through Blaisdell method (1981).

5.3.3 Soil Testing

In the present chapter, the critical shear stress results with four other approaches given through Equations (2.3), (2.4) and (2.7) have been compared. These include Shields' diagram and the empirical equations using the percentage clay content, mean particle size (Smerdon and Beasley, 1961) and percentage silt clay contents (Julian and Torres, 2006). Moreover, the estimated value of k_d was compared by using two empirical relationships (Hanson and Cook, 1997; Hanson and Simon, 2001; Wynn, 2004).

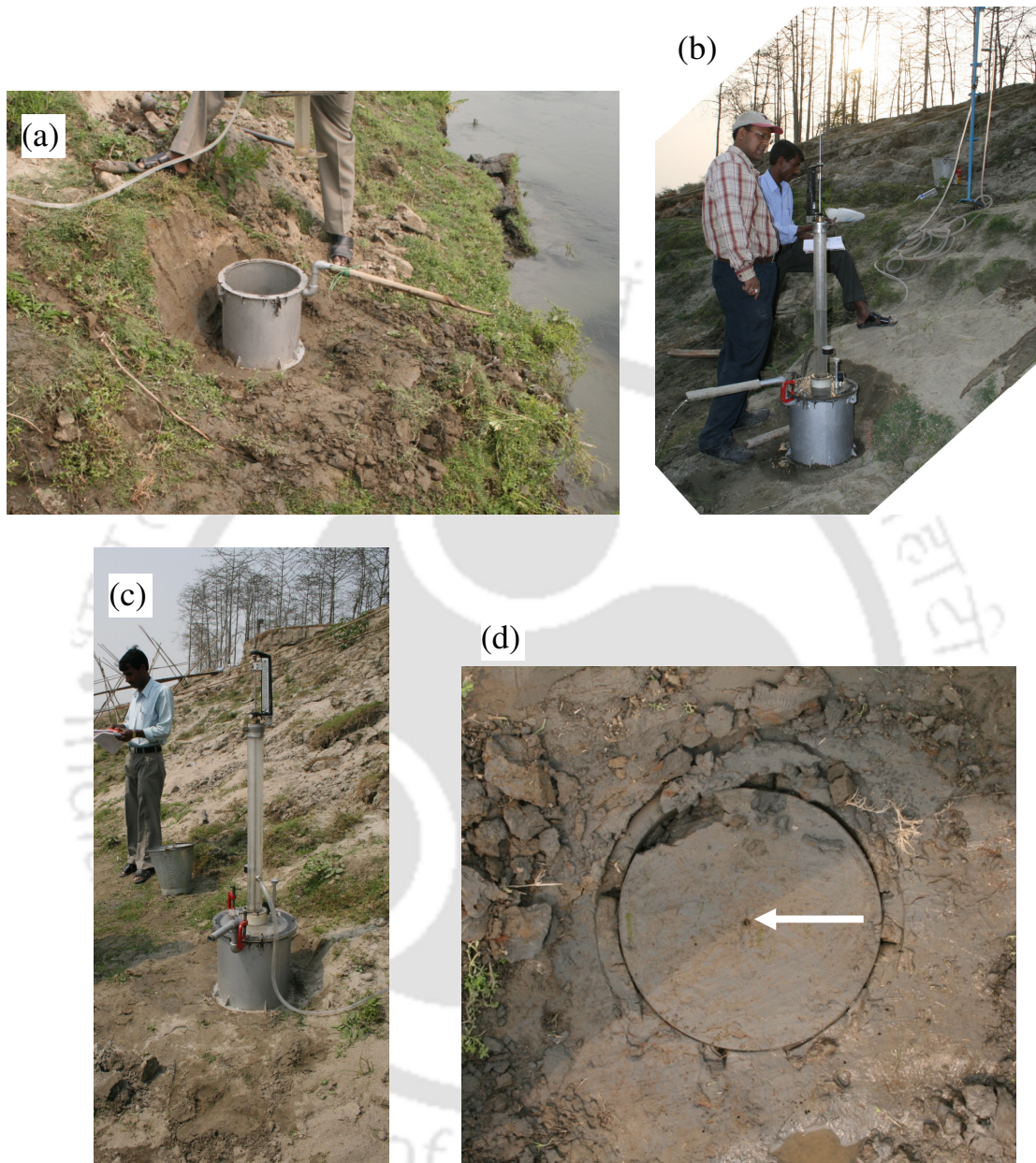


Fig. 5.5. Field photographs of the jet test experiment. (a) Site preparation (b) Installation of Jet column (c) jet experiments (d) scour hole (arrow mark) formed due to the jet.

Soil samples were collected adjacent to the sites where jet tests were conducted. In total ten samples were collected from the first site out of sixteen experimental locations where the bank erosion rates were monitored. The samples were collected as close to the previous test or when the tests were conducted in the different layers in the stratified river banks. Soil samples were oven dried for 24 hours prior to analysis for the particle size distribution using sieve analysis. Finer fractions (less than 75 microns) of the samples were analyzed using hydrometer method following guidelines given in IS standard 2720-part 4 (BIS, 1985).

5.3.4 *Shear Stress Estimation*

Using the excess shear stress equation (5.1), the bank erosion of the monitored bank can be estimated. The developed shear stress at any level of the bank was estimated by the equation (5.16) (Leutheusser, 1963):

$$\tau_a = 0.76 \rho g d S \quad (5.9)$$

where, τ_a is the developed shear stress (Pa) at a depth d (m) from top the free water surface, and S the longitudinal slope of energy gradient line (m m^{-1}). In equation (5.1), the negative value of the excess shear stress gives the negative value of bank erosion, which has no physical meaning. Therefore, only the portion of the bank where the developed shear stress is more than the critical shear stress is considered for erosion estimation. It is assumed that the bank materials where the developed shear stress exceeds the critical shear stress are homogeneous and composed of similar fine soil as tested in that vicinity.

5.4 Results and Discussion

The details of the bank stratifications observed along the banks of the Brahmaputra river has already discussed in Chapter 3. The experimental results are discussed in this section.

5.4.1 *In Situ Jet Test*

The relative frequency measured for τ_c and k_d at each site along the Brahmaputra river is shown in Figures 5.6a and 5.6b respectively. Results for the critical shear stress show that the soils tested along the bank fall in between 0.1 Pa and 100 Pa. The majority of the critical shear stress values are between 1.0 Pa and 10 Pa. In the case of the Nalbari location, 100% of the critical shear stress values are within this range. However, for the red soil, more than 70% critical shear stress values are greater than 10 Pa, which shows its erosion resistant nature. In the case of k_d , all the data fall in the range from 0.1 to 10 $\text{cm}^3(\text{N-s})^{-1}$.

Analysis also indicates that there is a wide range of variation in the erosion resistance of the river banks, even at a particular site. As observed at the Jamuguri location, the critical shear stress varies from 0.01 Pa to as high as 23.34 Pa. Similarly, the ranges of the critical shear stress are 0.26 – 13.29 Pa, 0.22 - 12.74 Pa and 3.43 - 20.71 Pa at Amingaon, near Dolgobinda bank site and for the red soil hill, respectively. The critical shear stress at the most resistant bank site (red soil hill site) was found to be greater than 15 Pa. Whilst the critical shear stresses vary over five orders of magnitude, the erodibility coefficients (k_d) vary over only three orders. The erodibility coefficient (k_d) ranges from 0.62-11.28 $\text{cm}^3(\text{N-s})^{-1}$, 1.76-4.02 $\text{cm}^3(\text{N-s})^{-1}$, 0.52-7.14 $\text{cm}^3(\text{N-s})^{-1}$, and

$0.53\text{-}1.95 \text{ cm}^3(\text{N-s})^{-1}$ for Jamuguri, Dolgobinda bank site, Amingaon and red hilly soils, respectively.

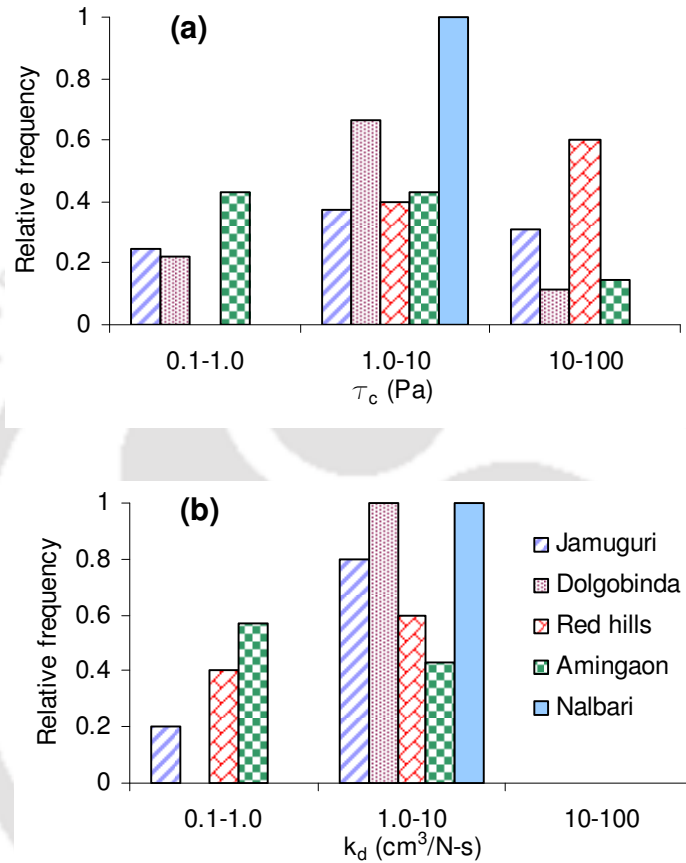


Figure 5.6. Relative frequency of soil erodibility parameters. For (a) critical shear stress and (b) erodibility coefficient.

The test results for river bank locations demonstrate that the critical shear stress and the erodibility coefficient are inversely related. Hanson and Simon (2001) and Wynn (2004) both found a similar power relationship between these two parameters. It has been found from the jet test analysis; they follow the relationship described as:

$$k_d = 3.16\tau_c^{-0.185} \quad (5.10)$$

where, k_d is the erodibility coefficient in $\text{cm}^3(\text{N-s})^{-1}$ and τ_c is the critical shear stress in Pa. Initially, the equation was derived from the simple least square fitting method, which generates $k_d = 2.447\tau_c^{-0.04}$. The correlation between them was very poor ($r^2 = 0.0076$). However, it was found that there are a few outlier data points for which it is not possible to fit the data with simple regression. In this case robust regression technique gives a better fitting. This actually uses an iteratively re-weighted least square fitting method. The weights at each iteration are calculated by applying the bi-square function to the residuals from the previous iteration. Thus this method gives less weight to the outliers and so the results are less sensitive to those outliers. As the robust regression is used for linear fitting equation, in this case, logs of both data has been taken and a higher correlation ($p = 0.002$) has been found. A similar trend of relationship was also reported by Wynn (2004), who found relationship as $k_d = 3.1\tau_c^{-0.37}$. Hanson and Simon (2001) also reported a similar relationship between them although the coefficients are significantly different. A comparison between the relationship derived by Hanson and Simon (2001), Wynn (2004) and the present investigation is depicted in Figure 5.7, which shows a wide variation between the regression coefficients. This may be due to the fact that Hanson and Simon carried out their tests in the river bed, while the present study was carried out for the river banks. The vegetation cover in the river bank imparts some resistance to erodibility due to root spreading and thus ultimately influences the relationship between the erodibility parameters (Wynn, 2004).

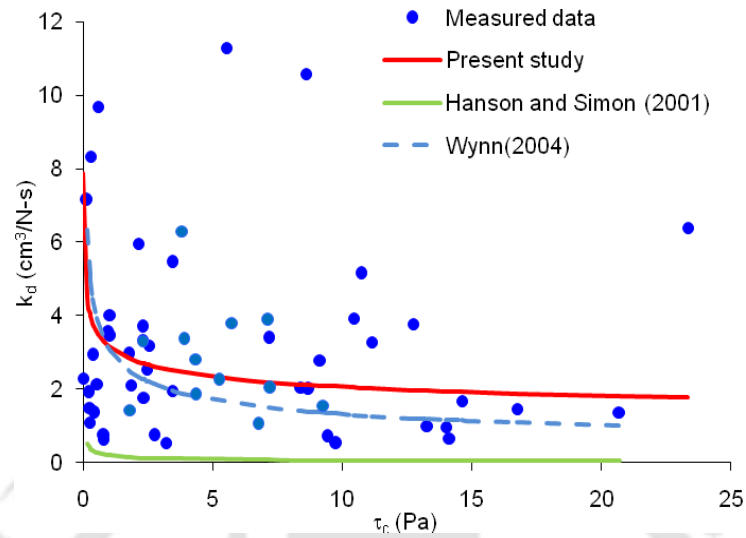


Figure 5.7. Relationship between critical shear stress and erodibility coefficient.

To relate these erodibility parameters with the resistant or erodible nature of the river bank, a scatter plot between the critical shear stress and the erodibility coefficients is illustrated in Figure 5.8. This indicates that at Jamuguri, some bank locations are very erodible while other places are only moderately erodible. Similarly in Amingaon the banks also range from very erodible to moderately resistant. On the other hand, near Dolgobinda, the river banks are all found to be very erodible. However, the average trend of the red hilly soils indicates that they lie on the upper limit of the moderately resistant zone.

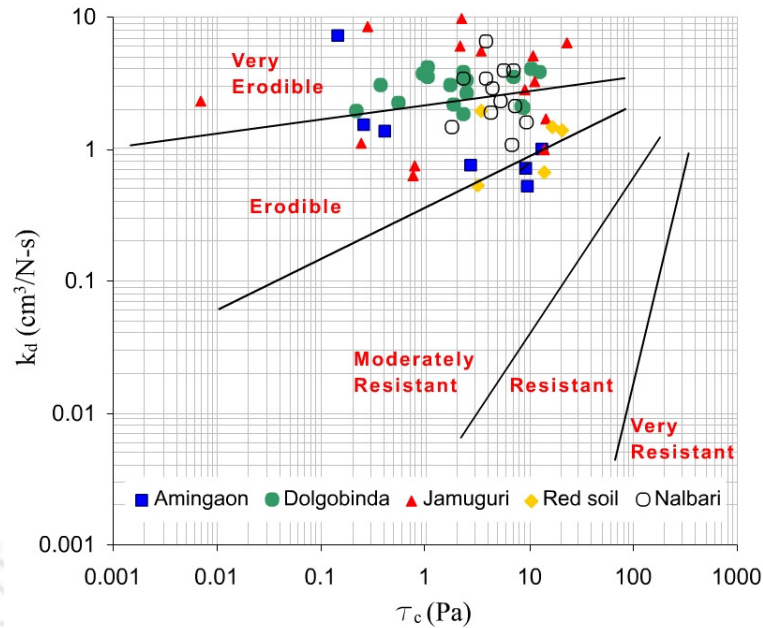


Figure 5.8. Classification of the river bank soil based on erodibility parameters (after Hanson and Simon, 2001).

Moreover, one dimensional ANOVA analysis was carried out for the erodibility parameters to test the locations and layer-wise dependency. Results show that the critical shear stress (Figure 5.9a) and erodibility coefficients (Figure 5.9b) are highly site specific (p value for critical shear stress and erodibility coefficients are 0.06 and 0.01, respectively). It means that the erodibility parameters vary significantly from one site to another. Figure 5.9a shows that critical shear stress values are quite similar for Dolgobinda and Amingaon, and also for Jamuguri and Nalbari. However, the ranges of critical shear stress for Jamuguri and Amingaon are wide. It can be observed from Figure 5.9b that the erodibility coefficient for the soil tested at Amingaon and the red soils are quite similar. So, for the estimation of the fluvial bank erosion it is important to estimate the local erodibility parameters of the bank soil.

ANOVA analysis is also carried out to test the dependency of the erodible parameters on the different layers of cohesive soils found at the site, (Figure 5.9c and 5.9d). However, the analysis shows that the erodibility parameters do not depend on the soil layers in a particular site. The p values for both the cases are very high (for τ_c , p value = 0.967 and for k_d , p value = 0.43), which indicates a poor relationship between the layers and the erodibility parameters. So for estimation of the fluvial erosion, layer-wise average erodibility parameters can be considered at a particular site.

Table 5.1 Estimates of the critical shear stress, median, and the ranges for different methods: Shields' diagram (SD), median particle size (D_{50}), percentage silt-clay content (SC) and percentage clay content

Method	τ_c (Pa)		
	Median	Minimum	Maximum
Jet test	2.14	0.007	23.34
SD (Shields' 1936)	0.17	0.15	0.17
D_{50} (Smerdon and Beasley 1961)	3.52	3.50	3.53
SC (Julian and Torres 2006)	2.70	1.47	5.93
%Clay (Smerdon and Beasley 1961)	0.50	0.49	0.51

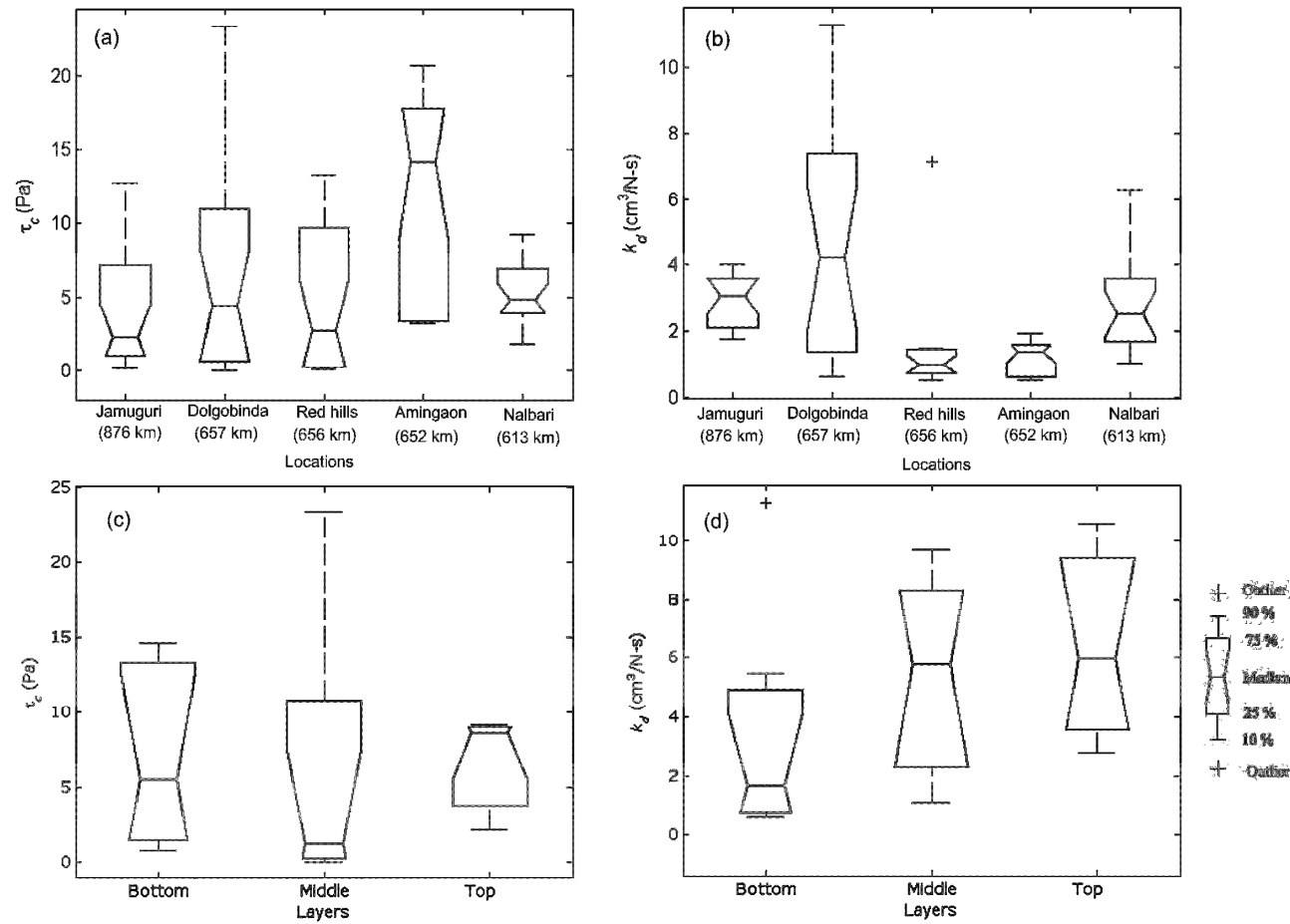


Figure 5.9. Spatial variation of erodibility parameters. For (a) critical shear stress on locations (p-value 0.060), (b) erodibility coefficient on locations (p-value 0.010), (c) critical shear stress on layers (p-value 0.967) and (d) erodibility coefficient on layers (p-value 0.437). (Numerals in bracket indicate the distance from the river mouth at Bay of Bengal)

Critical shear stresses measured *in situ* using the jet testing, and the same estimated using Shields' diagram (SD), median particle size (D_{50}), percentage silt-clay content (SC) and the percentage clay content (%Clay) are presented in Table 5.1. Comparison of the data indicates the poor estimations using Shields' diagram and % clay content. The one of the reasons is that Shields' diagram was developed for non-cohesive particles with no consideration of the interaction between particles (Shields, 1936; Vanonni, 1977; Hann et al., 1994). On the other hand, because the clay content was found to be very small for the soils tested, the % clay content method also gives poor estimation of critical shear stress. Estimation from percentage silt clay content method provides almost the same median value (2.70 Pa) as measured by the jet tests (2.14 Pa), while the median particle size estimation provides a slightly higher median value. However, the percentage silt clay content method can not assess the actual critical shear stress, as the water quality influences the soil erodibility (Clark and Wynn, 2007). For this reason submerged jet test method provides better estimate of critical shear stress under field conditions, while percentage silt clay content may be used as an alternative method for its approximate estimation.

5.5. Conclusions

The soil erodibility parameters of cohesive fine soils from composite banks along the Brahmaputra river in India were measured using the *in situ* submerged jet test apparatus. From the upper reach of the Brahmaputra river up to Nalbari, Assam were considered for the study. Results indicate that the critical shear stress ranges from very low (0.01 Pa) to as high as 23 Pa. The erodibility coefficient k_d , varies from $0.52 \text{ cm}^3(\text{N-s})^{-1}$ to more than

$11 \text{ cm}^3(\text{N-s})^{-1}$. The relationship between τ_c and k_d indicate that they do not follow the relation suggested by Hanson and Simon (2001), rather it closely follows the relations established by Wynn (2004) for this type of soil. Analysis of the critical shear stress and the erodibility coefficients indicate that these parameters vary significantly from one site to another, and thus for erosion estimation *in-situ* erodibility parameters need to be measured locally. Layer-wise analysis of the erodibility parameters in a particular location indicate that the variation of the erodibility parameters is not significant, and thus average erodibility parameters of the soils from various layers can be taken for a particular bank. These *in situ* erodible parameters can be considered for qualitative estimate of yearly bank erosion for these types of composite river bank.

CHAPTER 6

BANK SEEPAGE EROSION: EXPERIMENT AND MODELING

6.1 Introduction

Although the tractive force erosion is common and considered to be the major bank erosion processes. However, several studies reported that the bank failure may occur long after the high stage flows and the places are not anticipated to the fluvial erosion (Hagerty, 1991a, b). This essentially indicates the involvement of other major bank erosion processes and seepage flow from the river bank is considered to be one of the important mechanisms (Darby and Throne, 1996a; Crosta and di Prisco, 1999; Rinaldi and Casagli, 1999). The soil particles come out with exfiltration water from the river bank and form a zone of seepage undercut, and consequently failure of the overhang bank mass. For a composite river bank, this phenomenon is common as the presence of sand layers in between fine soils is responsible for higher rate of seepage flow. High rainfall infiltration rates can cause the development of the perched water table above the less impermeable layers, thus large hydraulic gradient can initiate towards the stream channels causing fairly rapid subsurface flow towards stream (Fox et al., 2006).

It is important to establish the experimental relationships between the available seepage gradient and the bank erosion and to apply these relations to the river scale. But till date, no research study has been carried out to characterize the seepage behavior for a composite river bank and modeling the seepage erosion and subsequent bank retreat. With limited studies, Fox et al. (2006), Wilson et al. (2007), Chu-Agor et al. (2008) were able to

establish the relationship with the seepage head and the seepage sediment discharge from the composite river bank. This chapter investigates the functional relationships between the river stage fluctuation and the development of seepage gradient followed by subsequent seepage erosion for different river bank stratifications. The development of a mathematical model describing the groundwater fluctuation due to multiple flood waves is discussed here, thoroughly. The functional relations obtained from the lysimeter experiments were incorporated into the model to estimate the daily seepage erosion. *In situ* seepage erosion measurements were carried out at the end of the monsoon period and used to validate the model prediction for estimating annual seepage erosion for the composite riverbanks.

6.2 Methodology

6.2.1 Experimental Setup

An experimental setup was fabricated using Perspex sheet of 12 mm thick (Fig. 6.1). The name *lysimeter* was given by the first developer for the set up (Fox et al. 2006). The details of the apparatus are shown separately in Appendix-B. It is a 1 meter long, 1 meter high and 15 cm wide channel. One side along the width is kept open to represent the river bank face and the opposite side there is a water head tank, which can provide constant seepage head during the experiment. In between the water tank and the main body of the setup, perforated perspex sheet of 12 mm was inserted through which water can seep into the model. The porous sheet has the holes of 3 mm diameter and measures were taken to prevent the soil movement from the experimental compartment to the seepage tank either by using fine mesh or by putting cotton in the holes. Before packing the lysimeter with the soil, a metal plate was placed to protect the porous plate from clogging with the packed soil. At the front face also, a metal plate was used for facilitate compacting. At one side, four piezometers are fixed to the

lysimeter at 20 cm spacing and 5 cm above the bottom plate. For providing different seepage head, openings are located at 20, 40, 60 and 80 cm from the bottom of the lysimeter at the back side of the seepage water tank. For easy collection of the sediment and discharge sample during experiments, an outlet is also provided at the front face.

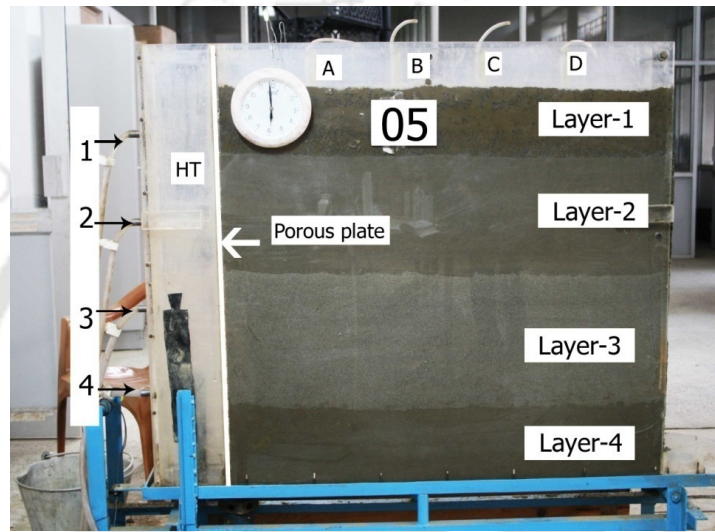


Fig. 6.1. Experimental set up of lysimeter (Layer-1: silt; Layer-2: fine sand; Layer-3: medium sand; Layer-4: fine sand)

Table 6.1. Soil properties of the stratified riverbank of Brahmaputra

Texture	Composition		Bulk density (kN/m ³)	Hydraulic conductivity (cm/day)	Moisture content		
	Sand (%)	Silt (%)			Residual, (cm ³ /cm ³)	Saturated, (cm ³ /cm ³)	α (cm ⁻¹)
Silt	6	91	17.6	56.7	0.04	0.54	0.01
Medium sand	98	1.9	14.5	1128.0	0.05	0.38	0.04
Fine sand	82	17	15.5	147.2	0.03	0.40	0.05

Soils are packed in the lysimeter at 2.5 cm lifts to bulk densities measured at the field. Two different bank heights are investigated: 70 cm and 90 cm. Four different soils are collected and used as layers for the packing of the lysimeter. Details of the layers are described in Table 6.1.

Detail experimental procedure has been discussed by Fox et al. (2006). Before starting of the experiments, packed soils are saturated with a solution of 0.005 (M) CaCl_2 in order to prevent the dispersion of soil particles. Experience suggests keeping the packed soil saturated for no more than two hours in order to prevent liquefaction of soil. Another two hours are allowed to drain out the extra water to achieve the antecedent moisture condition.

Two digital cameras are used to monitor the experiments: one captures from the side and the second one from the front focusing the bottom two layers. Sampling cylinders are also arranged to collect the water and soil discharge from the lysimeter during the experiment. Photographs are taken at regular interval and specially to capture the seepage erosion and the final collapse. During the experiments, piezometers readings are also recorded till the final collapse at regular interval. The experiments were conducted by varying seepage head (40, 50, 60, 70, 80, 90 or 95 cm), bank slope (0, 5 or 10%), bank height (70 or 90 cm) and arrangement soil stratifications (number of semipermeable layers). For inclined lysimeter tests, a vertical bank face is prepared by cutting the extra soil mass after packing. Experiments were conducted for various soil stratifications with numbers of silt/cohesive and sand layers commonly observed at the composite river bank of Brahmaputra. Captured images are processed through a digital image processing software to scale the image. The width and the volume of bank collapsed are measured from the processed images. Each experiment was repeated to check the consistency of result.

In the experiments, four basic soil layers were used with three soil samples: silt, fine sand and medium sand (Fig. 6.2). Bottom layer with fine sand (13 cm or 17 cm), underlain by medium sand (30 cm or 36 cm), underlain by fine sand (15 cm or 25 cm) and the top silt layer (12 cm) were repacked in lysimeter for thirty-two basic experiments. Along with these stratifications, thirty experiments were conducted with additional two silt layers (3 cm) introduced in between fine and medium sand layers. Additional one silt layer (3 cm) was introduced within the medium sand layer by equally dividing the height of this layer for another five experiments. The dimensions of the silt and sand layers were scaled as per the dimensions observed in the river bank. Total 67 experiments were carried out to investigate the significance of different parameters on the seepage erosion. The details of these experiments are discussed in Section 6.3.2.

Soil sample collected from the field was tested for the particle size distribution, cohesion, and field bulk densities. During the low stage periods, river bank survey was conducted and seepage erosion width was collected from the locations where seepage erosion had been prominent.

6.2.2 Mathematical Bank Seepage Model

A mathematical model is formulated considering the subsurface flow system in a confined aquifer, (Fig. 6.2). The initial stage at the river and as well as groundwater is h_0 , at $t = 0$. The origin of the horizontal axis is at the intersection of the bank and the river, while the positive direction towards the bank. It is assumed that the storage in the semipermeable layer is negligible (Hantush and Jacob, 1955).

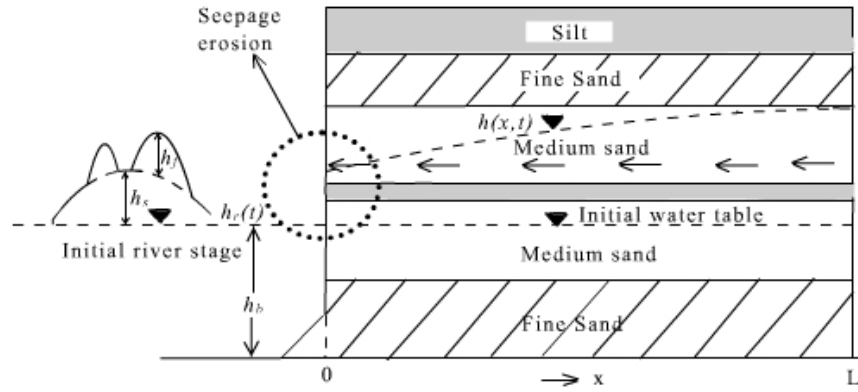


Fig. 6.2. Schematic diagram for seepage erosion due to groundwater exfiltration

Thus the governing equation for one-dimensional ground water flow in the confined aquifer can be given as:

$$S' \frac{\partial h}{\partial t} = T' \frac{\partial^2 h}{\partial x^2} \quad (6.1)$$

$$h(x, 0) = h_0 \quad (6.2)$$

$$h(0, t) = h_0 + h_f \sin(\omega t + c) \quad (6.3)$$

$$\lim_{x \rightarrow \infty} h(x, t) = h_0 \quad (6.4)$$

where, S' is the storativity, T' the transmissivity, h_f the amplitude of the flood wave, ω the speed of flood wave. Jiao and Tang (1999) gave the approximate solution to the above equations and can be written as within the domain between 0 and L:

$$h(x, t) = h_0 + h_f e^{-px} \sin\left(\omega t + c - \frac{\omega S'}{2pT'} x\right), \text{ for } 0 < x < L \quad (6.5)$$

$$\text{where, } p = \sqrt{\frac{\omega S'}{2T'}} \quad (6.6)$$

The seasonal stage hydrograph for a large river basin can be separated into two components, namely flood waves and monsoonal response (Chapter 4). Monsoonal response

is the part of the average response of monsoonal rainfall system. A relationship was established (Eq. 4.6) between the total monsoonal rainfall (R_s) at basin level and monsoonal response (h_s), which is the average lift of the ground water level.

After fitting the individual flood wave and the monsoonal response with Maxwell distribution (1960), the synthetic stage hydrograph can be generated (Chapter 4, Eq. 4.6).

In order to apply the solutions provided by Jiao and Tang (1999) it is necessary to decompose the flood waves developed in Eq. 4.6 into the summation of sine functions. Thus Equation (6.5) can be re-written as:

$$h_r(t) = h_b + f_1(R_s) \left[\frac{t}{k_s} \exp \left(1 - \frac{t}{k_s} \right) \right]^{r_s} + \sum_{i=1}^n \sum_{k=1}^m A_k \sin(\omega_k t_k + c_k) \quad (6.7)$$

where, A_k , ω_k , c_k and t_k are the amplitude, frequency, phase shift and time period of the k th sinusoidal component of i th flood wave, respectively. The difference between the river stage [$h_r(t)$] and the ground water level $h(x,t)$ produces the seepage head. To implement the equation (6.7) with equation (6.3), h_0 is substituted as follows:

$$h_0 = h_b + f_1(R_s) \left[\frac{t}{k_s} \exp \left(1 - \frac{t}{k_s} \right) \right]^{r_s} \quad (6.8)$$

The positive difference between groundwater level [$h(x,t)$] and the river stage [$h_r(t)$] at any time t gives the available seepage head. This head over the length (L) of the bank soil develops the seepage gradient (i). At the point, when the available seepage gradient is higher than the critical seepage gradient (i_c), seepage flow causes the bank erosion. This can be given as follows:

$$i = \left(\frac{h(L,t) - h_r(t)}{L} \right) \quad \text{for, } h(L,t) > h_r(t) \quad (6.9)$$

Time required to bank collapse (t_b) is a function of the seepage gradient and can be given as:

$$t_b = f_c(i) \quad \text{for, } i > i_c \quad (6.10)$$

Daily seepage bank erosion (ε_t) can be estimated as:

$$\varepsilon_t = \frac{24 \times 60}{f(i)} \times w_c \quad (6.11)$$

where, w_c is the average width of bank mass collapsed. The above equation has been developed considering the time to collapse (t_b) in minutes. When the river stage is higher than the ground water, the water will seep towards bank; otherwise seepage flow will occur from the river bank.

6.3 Results and Discussion

6.3.1 Soil characterization

Soil samples collected from the river banks were tested in the laboratory for its particle size distribution, field bulk densities and moisture content. Sand, silt and clay fractions of the soil were used to get the hydraulic conductivity of the soil using the Rosetta model (Schaap, 1999). This model by regression gives the most probable saturated hydraulic conductivity of the field soil sample and can be considered for all practical studies when detailed soil data are not available (Sarkar et al., 2008). Results indicate significant variation in saturated hydraulic conductivity between the layers (Table 6.1). Significant difference is also found in moisture retention characteristics between silt and medium sand.

6.3.2 Lysimeter experiments

Experiments were carried out from the soils collected from the river bank of Brahmaputra from different strata. Soils were re-compacted to its field bulk densities for each experiment.

A soil moisture profile probe (Delta-T, PR-2) was inserted into the packed soil during the experiment to monitor the moisture content through a meter (Delta-T, HH2) as found in the field [for details see Appendix-C]. Sixty-seven lysimeter experiments were conducted to investigate the significance of bank slope, seepage gradient, bank height and stratifications on the seepage erosion. The typical formation of the headcut due to seepage, its growth and the final collapse is shown in Fig. 6.3.

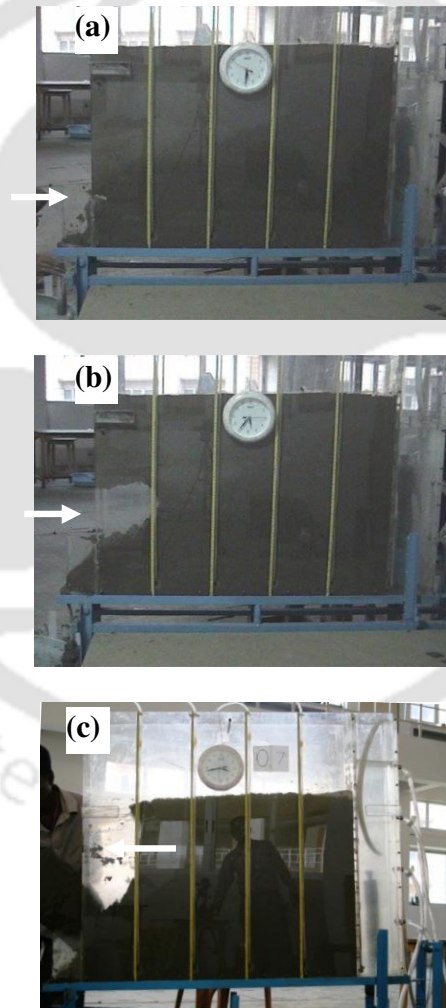


Fig. 6.3. Experimental photographs showing (a) seepage undercut development, (b) its growth and (c) the final bank collapse

Discharges from the lysimeter were recorded during the experiments. The average discharge was found to be 172.8 L/day, which is within the range as reported by Fox et al. (2006) and Wilson et al. (2007). Average soil erosion along with seepage flow was found to be 1.3 kg/L, which falls within the range as reported by Fox et al. (2006).

The influence of seepage gradient on the time of bank collapse was investigated through twelve experiments. Apart from that some experiments were also carried out at lower head and found that no seepage erosion occurred. As expected the rate of advancement of the wetting front increases with the increase of seepage head. The seepage gradient and the time of collapse indicate an inverse power relationship between them for two different bank heights (Fig. 6.4).

The impact of bank slope on the erosion is also shown in Fig. 6.4. As the bank slope increases, it is expected that the time to bank collapse will be less, but, the result shows the inconsistency for both the bank heights. Similar inconsistency in the time of bank collapse was also reported by Fox et al. (2006). However, the time of collapse was strongly correlated with the seepage gradient for both the bank height experiments.

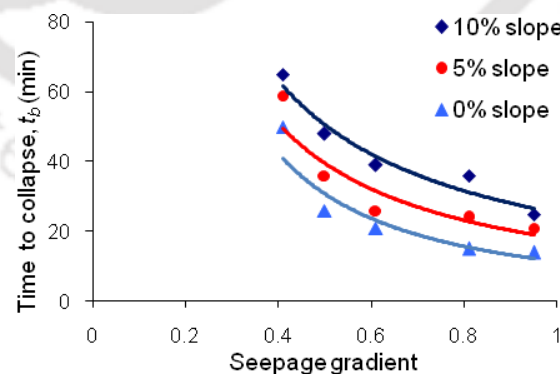


Fig. 6.4. Relationship between the time of collapse and seepage gradient for different slopes

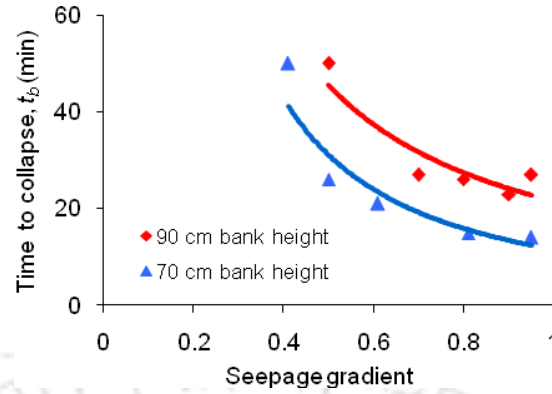


Fig. 6.5. Relationship between time of collapse and bank height for 0% slope

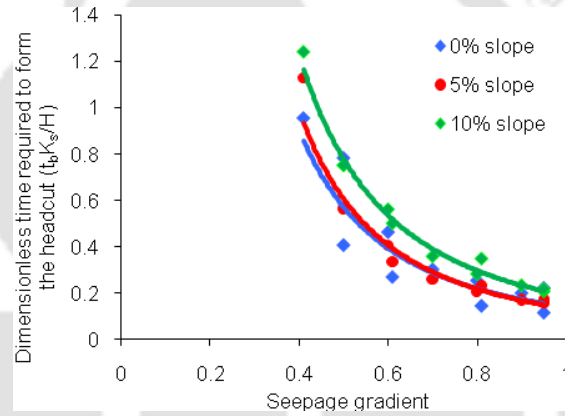


Fig. 6.6. Time required for seepage undercut and collapse, non-dimensionalized by the saturated hydraulic conductivity, K_s and the seepage head, H

The effect of bank height on the time of collapse was investigated for two bank heights (Fig. 6.5). The results are similar as found by Fox et al. (2006). For a higher bank height (90 cm), the time to collapse is more than that with 70 cm bank height. However, it is expected that with a higher bank height, the bank collapse time will be less. Therefore, to normalize the trend of bank collapse for different bank heights, the time required for developing headcut and collapse non-dimensional analysis for various seepage gradients and

bank heights were carried out. For this, time to collapse (t_b) is multiplied by $\frac{K_s}{H}$, where, H is the seepage head. Both the bank heights are considered for this analysis and grouped under three slope variations (0%, 5% and 10%). Fig. 6.6 shows the relations between the normalized time of collapse and seepage gradient. Analysis indicates the time to collapse remain unaffected by the bank height variation. For mild slopes (upto 5%) of the bank, dimensionless time required to collapse is not influenced by the slope variations. But for steeper slopes (i.e., 10%) dimensionless time required to collapse is significant.

The impact of stratification on the time of collapse was investigated through fifteen experiments. Three sets of bank stratification with semipermeable layer were considered: (1) Only top semipermeable layer, (2) Top + 2 semipermeable layers and (3) Top + 3 semipermeable layers. Analysis shows that as the number of semipermeable layer increases, the time of collapse increases significantly (Fig. 6.7). The respective relational functions for case (1), case (2) and case (3) obtained from the experiments as follows, respectively:

$$t_b = 81.87 \times i^{-1.426} \quad (6.12)$$

$$t_b = 150.57 \times i^{-1.356} \quad (6.13)$$

$$t_b = 249.28 \times i^{-1.416} \quad (6.14)$$

where, the unit of t_b is minutes.

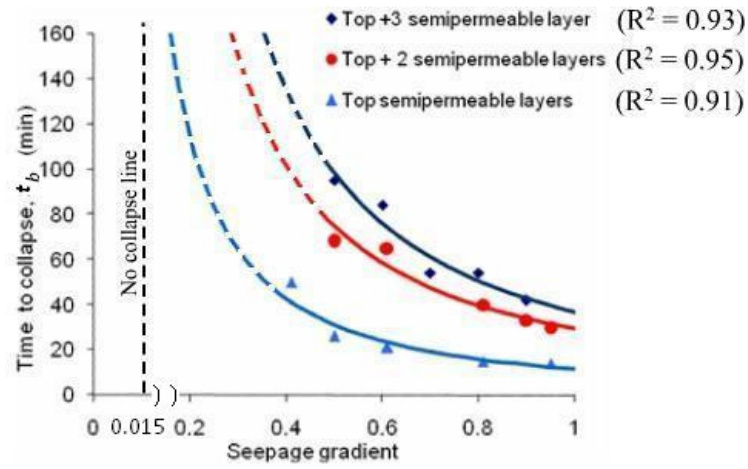


Fig. 6.7. Relationship between time to collapse and seepage gradient for different stratifications

6.3.3 Width of Bank Failure

Bank failure dimensions through the digital image processing were analysed to investigate the relationship of the collapse width of the photographs with the height of bank, and soil layer compositions. For that total experiments were divided into four cases: (1) Top semipermeable layer with 70 cm bank height, (2) Top +2 semipermeable layer with 70 cm bank height, (3) Top semipermeable layer with 90 cm bank height and (4) Top +2 semipermeable layer with 90 cm bank height. Analysis show the range and median are almost same for failure width (Fig. 6.8). This justifies to use the average collapse width w_c in the seepage model.

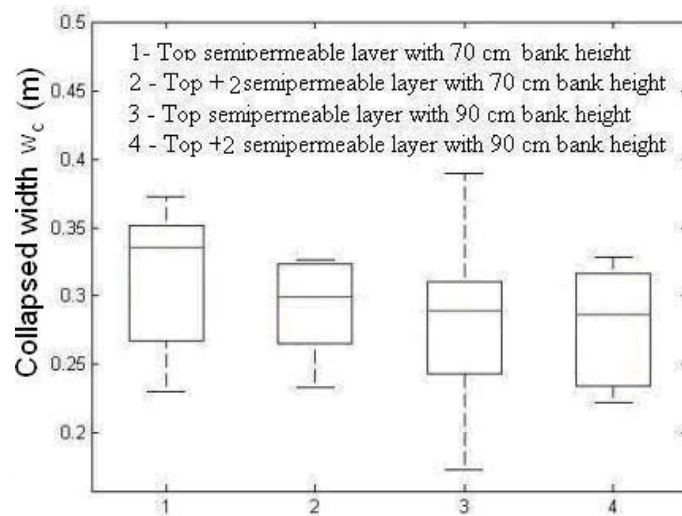


Fig. 6.8. Range and median of the collapse bank width for four different soil compositions

6.3.4 Seepage Erosion Prediction

The seepage function developed for various arrangements of number of silt and sand layers are used for the seepage modeling of natural riverbanks. As in the present model, the monsoonal response of the stage hydrograph is a slow hydrological process which is considered as the integrated response of the groundwater aquifer and can be assumed to follow the similar response; thus causing no seepage flow. Therefore, the seepage gradient will only be created due to the flood waves. Seepage gradient is calculated at a distance 1 m from the bank face as similar to the length adopted in the lysimeter experiments.

For mathematical modeling of seepage erosion, three types of soil stratifications with slope 0% were used as discussed in the previous section. It has been already mentioned that the lysimeter experiments were carried out for the maximum collapse time of 2 hours. But in a river, the seepage erosion can occur through 24 hours if we assume the river stage in one day is either constant or its diurnal fluctuation is negligible for all practical purposes. So the functions developed through the lysimeter experiments were extended backward upto the

collapse time of 24 hours. It was found that after this time the functions are parallel to the y-axis. That means if the seepage gradient is lower than a threshold, practically there would be no seepage flow. For different arrangements of bank stratifications, the threshold seepage gradient is found to be the same and for all practical purpose, can be considered as a constant (the present case, 0.015 m/m). The seepage head is mainly created due to delay in the peak between the river stage and the groundwater aquifer.

Seepage erosion model simulations were carried out for flood wave return period of 1, 2, 5, 10, 15 and 20 years (Figs. 6.9 and 6.10). Fig. 6.9 shows the typical time series of river stage with the estimated seepage gradient developed and the estimated cumulative seepage erosion during the monsoon period. Results from the different return period of flood waves indicate that for a particular soil stratification and low return period flood waves, annual seepage erosion vary significantly, whereas for higher return periods of flood wave (more than 10 year), annual seepage erosion is nearly constant (Fig. 6.10). Interestingly, as the number of semipermeable layer increases, the total seepage erosion decreases significantly. With the presence of two semipermeable layers, annual seepage erosion decreased nearly 50%. With further introduction of another semipermeable layer, annual seepage erosion was further reduced about 36% (Fig. 6.10). Therefore the model predicts the ranges of annual seepage erosion between 1 and 6 m for various combinations of bank stratification.

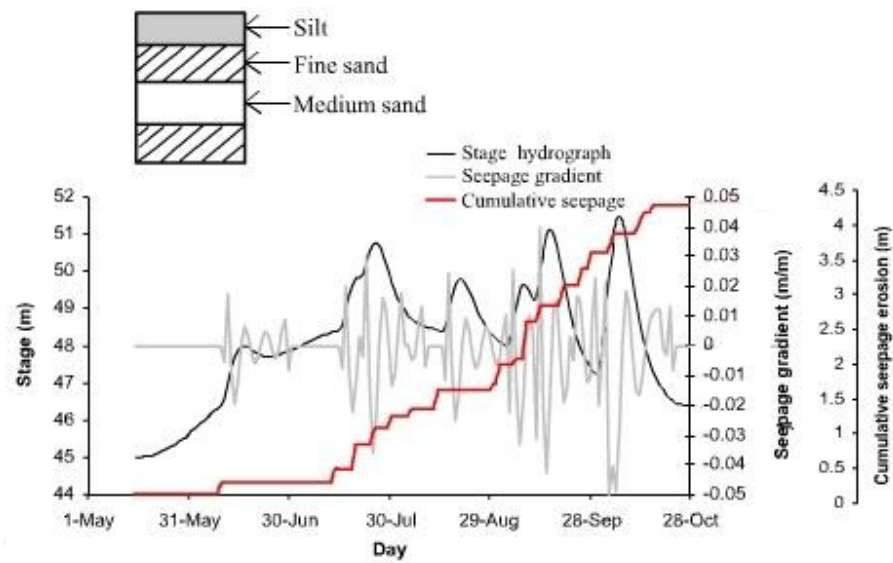


Fig. 6.9. Time series seepage analysis with a 10 year return period of flood wave, developed maximum seepage gradient and cumulative seepage erosion

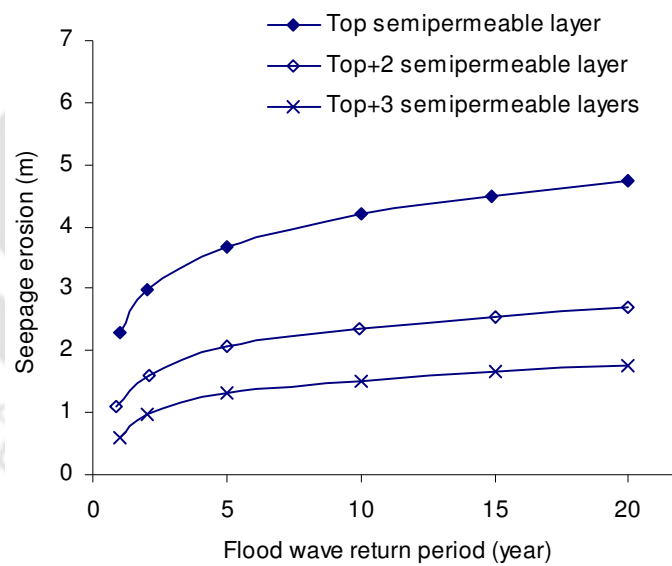


Fig. 6.10. Predicted annual seepage erosion for different arrangement of semipermeable layers

6.4 Field Verification of the Seepage Erosion

No *in situ* data have been reported so far for the seepage erosion from the bank of Brahmaputra river. For the present study, the *in situ* seepage erosion was measured after the end of monsoon period, as during this time seepage erosion will be maximum. However, the seepage erosion measurement during the monsoon was not possible due to practical difficulties during this period; difficulties in differentiating between seepage erosion and fluvial erosion. Thus the erosion measurements collected from the field obviously do not represent the total annual seepage erosion. Although, from the model estimation, it was found that for most of the cases, major seepage erosion occurs at the end of the monsoon period. The range of seepage erosion measured is 2.0 m to 7.1 m with an average of about 4.4 m (Table 6.2). This range is quite similar as found from the model prediction.

Table 6.2. Field measurements of seepage erosion

Location	Seepage erosion (m)	Average erosion (m)
Dolgobinda (Guwahati)	4.3 m	5.5 m
	6.7 m	
	5.5 m	
Amingaon	7.1 m	4.5 m
	2.0 m	
	5.3 m	
	3.7 m	
Sualkuchi	2.5 m	3.4 m
	3.2 m	
	4.6 m	
Singimari	5.7 m	4.3 m
	5.2 m	
	3.6 m	
	3.0 m	

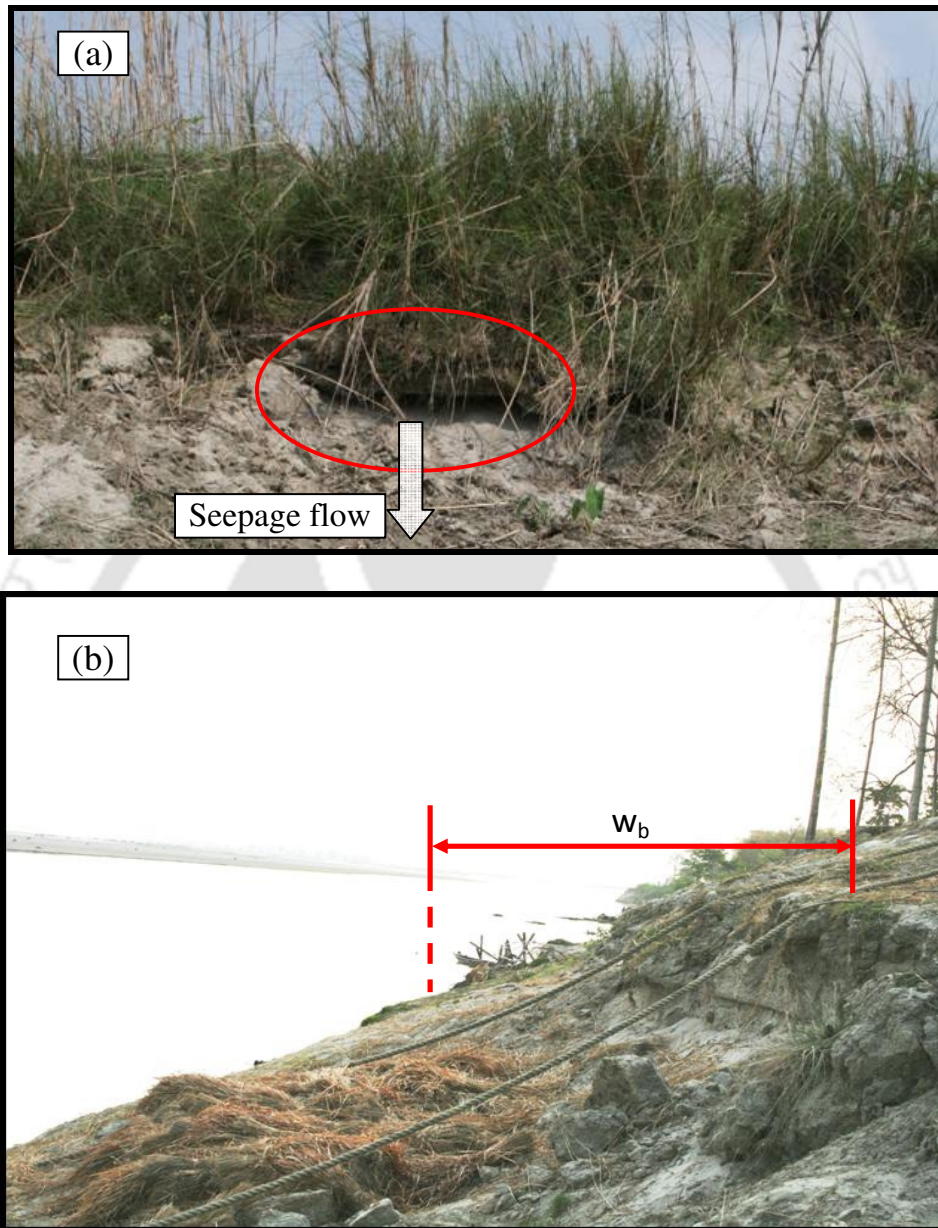


Fig. 6.11. Field photographs showing (a) seepage undercut at Sualkuchi ($26^{\circ}09'50''N$, $91^{\circ}34'26''E$) during February, 2010 and (b) seepage erosion at Jamuguri ($26^{\circ}50'08''N$, $93^{\circ}46'08''E$) during December, 2008.

6.5 Conclusions

The controlling factors for the seepage bank erosion have been described in the present chapter. The soils collected from the composite riverbank of Brahmaputra have been repacked into the lysimeter to represent the bank stratification. Total 67 lysimeter experiments were conducted to characterize the controlling parameters of seepage erosion.

Results indicate an inverse power relationship between the seepage gradient and time of bank collapse. Moreover, increase in the number of semipermeable layer significantly increases the time of collapse. Bank height has been found to be insignificant effect on the time of bank collapse. However, the functions developed from the experiments show critical seepage gradient for the stratified soils, lower of which there will be practically no seepage erosion.

Functions between the seepage gradient and the time of collapse as established from the lysimeter were used for modeling daily seepage erosion for flood waves of different return periods. The development of the mathematical model is a new concept presented in this chapter. It was found that as the number of semipermeable layer increases, total seepage erosion decreases significantly. Field seepage erosion measurements were also carried out at the end of the monsoon period. The field measurements remain within the ranges as predicted by the mathematical seepage model. The model prediction is acceptable as lot of uncertainty involved in the natural bank seepage erosion processes and bank stratification. As the total annual bank erosion of a river reach is the sum of seepage and fluvial erosion for the composite riverbank; integration of seepage erosion with fluvial erosion processes will be addressed in the following chapters.

CHAPTER 7

ANALYTICAL MODELING OF COMPOSITE BANK EROSION

7.1 Introduction

High rate of bank erosion in the alluvial rivers has been of great concern, causing serious problems to the habitats, river and environmental engineers and others through loss of fertile land, danger to floodplain structures, increased downstream sedimentation, loss of valuables and lives (Muramoto et al., 1992; Mosselman et al., 1995; and Tingsanchali and Chinnarasri, 1997). Being associated with so many controlling variables with uncertainty in the measurement, hydrodynamic and morphodynamic processes involved in the bank erosion are always difficult to be completely understood/modeled in a large river (Karmaker and Dutta, 2009).

Duan (2005) developed an analytical approach to calculate the rate of bank erosion for cohesive soil. He related the bank erosion with the frequency of flooding and thus treated the erosion phenomenon as probabilistic approach. Chen and Duan (2006) solved the two dimensional depth averaged flow equations for a sine generated channel with bank erosion processes. They considered the rate of bank erosion as a net result of near bank sediment deposition and transport.

In spite of recent advancements in the bank erosion modeling and monitoring, limited studies so far have been carried out for composite riverbank; especially for the alluvial riverbank which is mostly composed of sand, silt and clay in layers. There is no mathematical river model available till date for estimate the composite bank erosion rate. In this chapter an analytical river model is formulated and developed for the composite

river bank considering the entrainment and deposition of the sediment particle from the bank surface, basal erosion due to excess shear stress, cantilever mass failure and near bank net sediment transport rate and the seepage erosion that are commonly found in the composite river banks (Fox et. al., 2007). Another objective was to couple the analytical bank erosion and bed degradation model (Chen and Duan, 2006) with seepage erosion of the composite river bank.

7.2 Analytical Model Development

The basic processes involved in the modeling of the erosion in a composite river bank are described in Fig. 7.1. Due to the excess shear stress and seepage erosion near the toe of the bank, the silt/cohesive soil layers is eroded keeping the upper bank soil mass overhang. Then due to the cantilever failure mechanism, the overhang soil mass fails and a fraction of the collapsed mass deposits near the bank toe. It may be assumed that the finer fraction of the collapsed bank mass is transported instantaneously as wash load with the river flow. The excess shear stress and the transport capacity of the river flow takes out the deposited bank mass in due course. Then the process goes in a cyclic order.

The different components of the processes described above are summarized in the flow chart (Fig. 7.2). From the given stage and the initial river channel geometry, the velocity and the discharge are computed. Then the average fluvial erosion of the bank can be estimated. As the seepage erosion is a independent process and hence from the stage records, the seepage erosion can be estimated simultaneously. The summation of the seepage erosion and the fluvial erosion at the bank toe will indicate the overhang bank mass. The overhanging bank mass then checked for the cantilever stability and once the

bank mass fails, the fractional part deposits near the toe. In the next the stage, the bed degradation is estimated and the new channel geometry is updated.

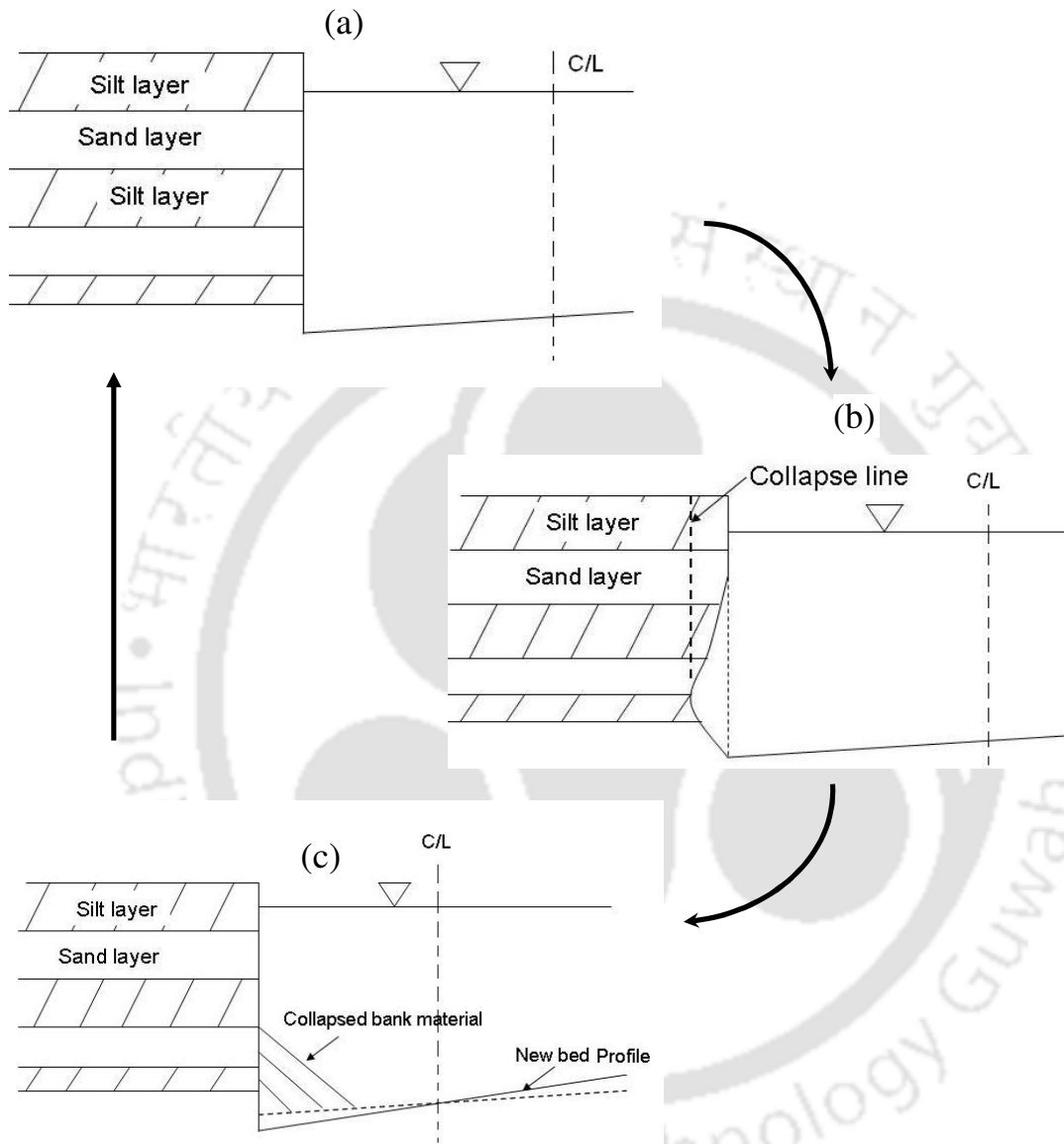


Fig. 7.1. Schematic diagram of various stages of the bank erosion in a composite river bank. (a) Initial bank profile (b) progression of basal erosion (c) failure of the cantilever mass and its deposition near river bed with angle of repose along with bed degradation.

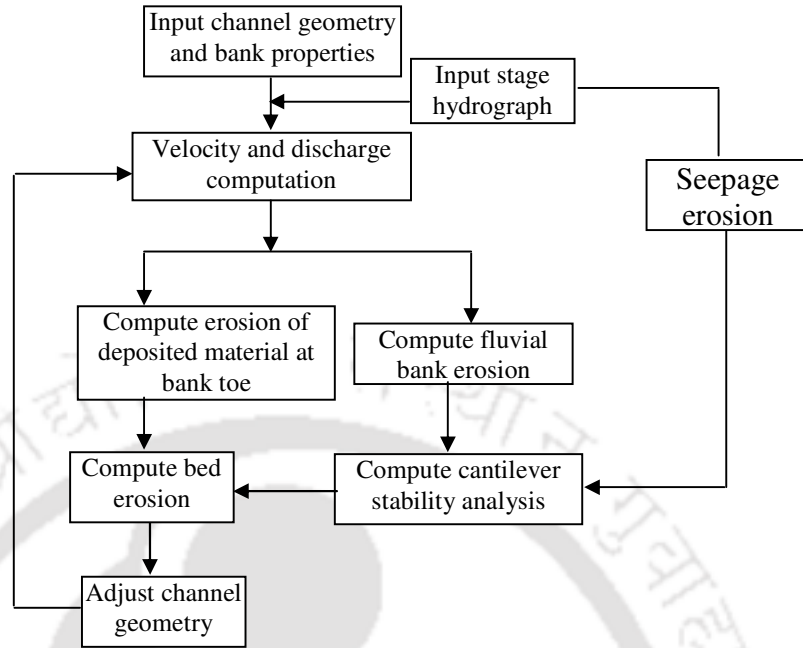


Fig. 7.2. Flow chart describing the hydrodynamic, morphological and bank erosion processes.

7.2.1 Hydrodynamic Model

The governing hydrodynamic equations used in this study are Navier-Stokes equations for depth averaged steady-flow condition in two-dimensions as follows (Chen and Duan, 2006):

$$\frac{\partial(hu)}{\partial s} + \frac{\partial}{\partial n}[(1+nC)hv] = 0 \quad (7.1)$$

$$\frac{1}{1+nC}u \frac{\partial u}{\partial s} + v \frac{\partial u}{\partial n} + \frac{C}{1+nC}uv = \frac{-1}{1+nC}g \frac{\partial \xi}{\partial s} - C_f u \frac{\sqrt{(u^2+v^2)}}{h} - \frac{1}{h} \left[\frac{\partial}{\partial n} \left(uh \int_{\eta}^{\xi} T v_s dz \right) + \frac{2C}{1+nC} uh \int_{\eta}^{\xi} T v_s dz \right] \quad (7.2)$$

$$\frac{1}{1+nC}u \frac{\partial v}{\partial s} + v \frac{\partial v}{\partial n} - \frac{C}{1+nC}u^2 = -g \frac{\partial \xi}{\partial s} - \frac{\tau_n}{\rho h} \quad (7.3)$$

where, u and v are the depth integrated velocity components in the longitudinal and transverse direction, ξ and η are the water surface and bed elevation from datum respectively, h the flow depth, C the curvature of the channel centerline and z axis is considered positive upward (Fig. 7.3). The channel centerline can be estimated as follows:

$$C(s) = \frac{d\theta}{ds} = \frac{1}{r} \quad (7.4)$$

where, θ is the deflection angle between the downstream and stream-wise direction and r the local radius of curvature.

The two integral terms in Eq. (7.2) are the dispersion terms generated due to the redistribution of the momentum along the stream-wise direction for the secondary flow in a river bend. T is the dimensionless shape function denoting the vertical fluctuation of longitudinal velocity and v_s the secondary flow velocity in the transverse direction.

The individual river bend in a large braided river can be simplified as a sine-generated curve similar to the meandering channels (Langbein and Leopold, 1966). The periodic shape of the loop can be defined by the channel centerline angle (θ) with respect to down-valley direction and wavelength of the loop. The sine generated curve can be expressed as follows (Chen and Duan, 2006):

$$\theta = \theta_0 \cos(ks) \quad (7.5)$$

where, $k = 2\pi/\lambda$, λ is the wavelength of the braided loop, θ_0 the value of θ at the deflection point of the main channel. By assuming the ratios of the flow depth to the radius of channel curvature are small and neglecting the second order terms in Eqs. (7.1)-

(7.3), the first order analytical solution of the streamwise velocity, considering the effect of dispersion terms in a sine generated curved channel can be given as:

$$u = U + UNbk\theta_0[-\alpha \cos(ks) + \beta \sin(ks)] \quad (7.6)$$

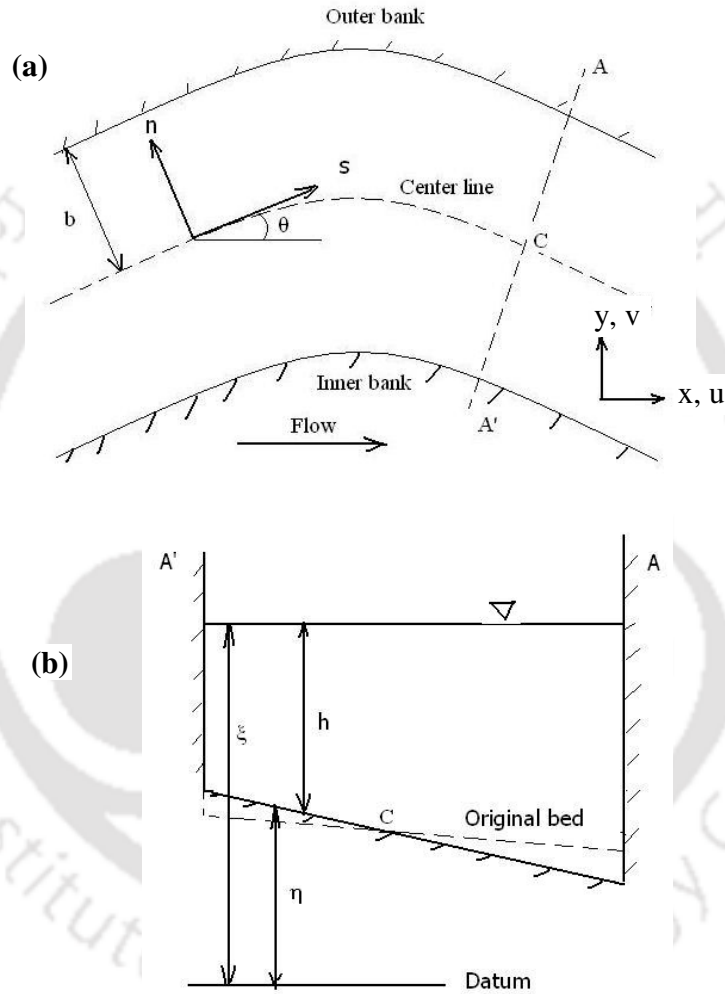


Fig. 7.3. Definition sketch and the coordinate system: (a) Plan view; (b) Channel cross-section at A-A'

where, U is the reach-average velocity, b the half width of the channel. The value of N ranges from positive unity at the outer bank to the negative unity at the inner bank. The parameters α and β can be given as follows:

$$\alpha = \frac{kHC_f (A + A_s + F^2 + 1)}{H^2 k^2 + 4C_f^2} \quad (7.7)$$

$$\beta = \frac{2C_f^2 (A + A_s + F^2 - 1) - H^2 k^2}{H^2 k^2 + 4C_f^2} \quad (7.8)$$

where, C_f is the Chezy's friction constant; F the Froude number and equals to $U / \sqrt{gH}$, H the reach average flow depth, A is the scour factor, and A_s the momentum redistribution factor due to secondary current. However, the parameter A_s can be neglected for a mildly sinuous channel bends ($\delta < 1.2$), when the secondary flow is weak (Chen and Duan, 2006). The analysis of the satellite imagery shows that the sinuosity of the braided loops ranged between 1 and 1.2 for most of the cases and hence the approximation for the mildly sinuous channel can be considered. The analysis of the channel velocity also indicates the relatively insignificant strength of the secondary current for the river bend (refer to Chapter 3). Johannesson and parker (1989) suggested that the parameter A can be neglected for the flat channel bed in transverse direction. However, in a bend, the natural channel bed is never flat and the transverse bed slope can be computed from the following equation (Johannesson and parker, 1989):

$$\frac{\partial \eta}{\partial n} = -AHC(s) \quad (7.9)$$

From the parametric study of the laboratory flume, Chen and Duan (2006) suggested that the parameter A follows a parabolic distribution as follows:

$$A = \sqrt{\frac{t}{30}} \quad (7.10)$$

where, t is the time of model run and ranges from 0 to 120 min. However, for natural river condition, with the existing channel bend the parameter A can be considered with a

maximum value computed from the following equation (Kikkawa et al., 1976; Chen and Duan, 2006):

$$A_{\max} = \left(\frac{3}{4} \frac{\mu C_D}{1 + \mu \frac{C_L}{C_D}} \right)^{1/2} \frac{7.0}{\sqrt{\left[\left(\frac{\rho_s}{\rho} - 1 \right) g D \right]}} (1+m) \left(\frac{D}{2H} \right)^m U \quad (7.11)$$

where, μ is the kinetic friction coefficient between sediment particle and channel bed, C_D and C_L are the drag and lift coefficients, respectively.

7.2.2 Fluvial Erosion Model

The fluvial erosion rate can be quantified using the excess shear stress theory proposed by Partheiades (1965) and Arulanandan et al. (1980):

$$\varepsilon = k_d (\tau_b - \tau_c)^a \quad (7.12)$$

where ε is the fluvial erosion rate per unit time, k_d the erodibility coefficient of the bank soil, τ_b the boundary shear stress for the cohesive soil, τ_c the critical shear stress and a the empirical exponent generally considered as 1.0. Total fluvial erosion (ω') for time Δt can be computed as:

$$\omega' = \varepsilon \times \Delta t \quad (7.13)$$

Erodibility parameters are highly variable and difficult to estimate (Rinaldi et al., 2008). *In situ* submerged jet testing devices was used to estimate the erodibility parameters for the cohesive soils (Hanson and Simon, 2001). The bed shear stress can be estimated by using, $\tau = \gamma R S$, where, γ the unit weight of water, R the hydraulic radius, S the energy slope. Bank shear stress was estimated by using, $\tau_b = 0.76 \gamma R S$, as suggested by Leutheusser (1963) [For details refer to section 5.3.4].

7.2.3 Entrainment of the Bank Material Deposited at Toe

A fraction of the collapsed bank mass will deposit at the bank toe. The remaining fraction will be removed instantaneously with the flowing water as a wash load. Depending on the bank soil characteristics, the deposited fraction can be up to 80% of the total bank mass. In this present study, as the majority of the bank material is either sand or cohesive silt layers, the fractional part deposited near the toe is considered as 80%. The net average rate of bank erosion due to entrainment and deposition of the sediment particle can be given through the following analytical relation (Duan, 2005):

$$\bar{\xi} = E_1 \left(1 - \frac{\tau_{bc}}{\tau_{b0}} \right)^{3/2} \sqrt{\tau_{b0}} \quad (7.14)$$

where $\bar{\xi}$ is the depth averaged erosion rate of the deposited material at toe, τ_{bc} the critical shear stress of the deposited material, τ_{b0} the shear stress at the toe of the bank slope, E_1 coefficient can be defined as:

$$E_1 = \sin \bar{\beta} \sqrt{\frac{C'_L}{3\rho_s}} \left(1 - \frac{C}{C_*} \cos \bar{\beta} \right) \quad (7.15)$$

where $\bar{\beta}$ is the average bank slope, C'_L the coefficient of lift force using friction velocity, C the depth averaged suspended sediment concentration, C_* the equilibrium suspended sediment concentration.

7.2.4 Stability and Cantilever Failure

Cantilever stability of the bank can be assessed by the combined effect of self weight, shear strength, tensile strength and the compressive strength of the bank materials. Three modes of failures can occur: (1) shear failure, (2) beam failure and (3)

tensile failure. The factors of safety against these failures can be computed from the equations by Thorne and Tovey (1981):

$$F_{ss} = A \times \frac{(\beta - \chi)}{\beta} \times \frac{\beta}{2r} \quad (7.16)$$

$$F_{sb} = A \times \frac{\beta^2}{(1+r)B'} \quad (7.17)$$

$$F_{st} = A \times \frac{B}{(1-\beta)} \quad (7.18)$$

where F_{ss} the factor of safety against shear failure, F_{sb} the factor of safety against beam failure, F_{st} the factor of safety against tensile failure, r the ratio of the tensile strength to the compressive strength of soil. Other parameters can be given as follows:

$$A = \frac{\sigma_t}{\gamma b_1} \quad (7.19)$$

$$B = \frac{b_1}{H'} \quad (7.20)$$

$$B' = B \left(\frac{\beta}{\beta - \chi} \right)^2 \quad (7.21)$$

$$\beta = \left(\frac{H' - l}{H'} \right) \quad (7.22)$$

$$\chi = \left(\frac{m}{H'} \right) \quad (7.23)$$

where H' is the overhang height of the bank, m the length of the upper crack, l the length of the lower desiccation crack, b_1 the width of the overhang bank.

7.2.5 Bed degradation model

Bank advancement is caused by the deposition of the sediment at the bank. The bank retreat as it is eroded and transported away by the flow. Predicting bank advance or retreat is based on the mass conservation of the sediment in a control volume near the bank (Fig. 7.4), including sediment from bank erosion and bank failure, sediment stored near the bed and sediment flux transported in and out from the control volume (Chen and Duan, 2006).

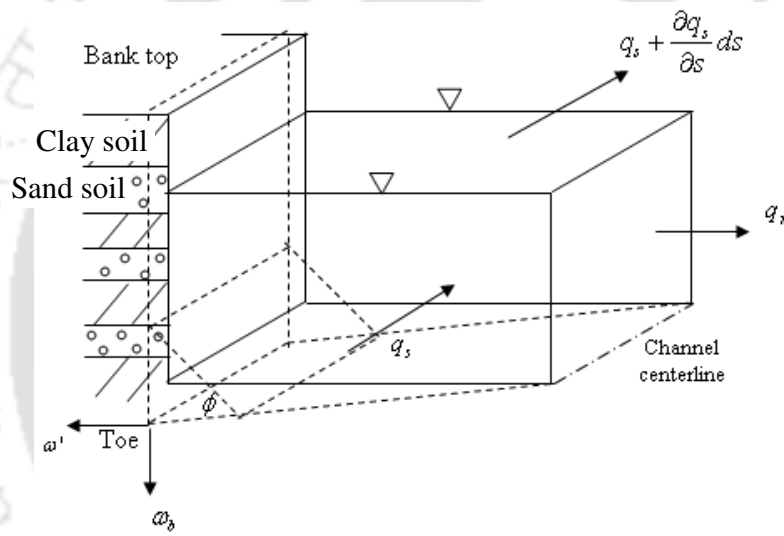


Fig. 7.4. Sediment mass balance in a river bend

Within a control volume near the bank, b is the width and ds is the stream-wise length of the control volume; q_s is the longitudinal sediment flux entering into the control volume, $q_s + \frac{\partial q_s}{\partial s} ds$ is the longitudinal sediment flux and q_n is the net transverse sediment leaving the control volume. The bed degradation and near bank morphological changes is based on the mass balances of the sediment within the control volume. Thus the sediment continuity equation can be written as follows:

$$\left(q_s + \frac{\partial q_s}{\partial s} ds \right) b - q_s b + q_n ds = -\omega' h ds (1-P) \rho_s - \omega_b \frac{b ds}{2} (1-P) \rho_s - \bar{\xi} \frac{dh}{\cos \theta} ds (1-P') \rho_{sand} \quad (7.24)$$

where ω' is the lateral bank erosion rate at the toe per unit time in transverse direction, ω_b the bed degradation rate at the bank toe, ρ_{sand} the density of the sand from river bank, q_s and q_n are the sediment transport rate component in longitudinal and transverse directions, P the porosity of cohesive soil, P' the porosity of the sandy deposits from the bank, θ the angle of repose of deposited soil mass at the bank toe, b the half width of the channel. The term $\omega' \frac{h ds}{2} (1-P) \rho_s$ represents the sediment contribution from the bank within the control volume, $\omega_b \frac{b ds}{2} (1-P) \rho_s$ represents the sediment contribution from the river bed erosion, and $\bar{\xi} \frac{dh}{\cos \theta} ds (1-P') \rho_{sand}$ represents the contribution from the deposited soil mass at the toe of the bank. Rearranging the above equation, we can write:

$$\omega_b = -\bar{\xi} \frac{2}{b} \frac{dh}{\cos \theta} - \omega' \frac{2h}{b} \frac{(1-P)}{(1-P')} \frac{\rho_s}{\rho_{sand}} - \frac{2}{b(1-P')} \frac{\rho_{sand}}{\rho_{sand}} \left(\frac{\partial q_s}{\partial s} b + q_n \right) \quad (7.25)$$

It may be noted here, Chen and Duan (2006) considered that due to excess shear stress the bed and the bank toe erodes first. Once the eroded depth of bed near the toe exceeds a limit, then due to slope failure mechanism, subsequently river bank fails. Hence, in that model, the bed degradation was the cause for bank erosion in a non-cohesive river bank. But in the present model, the composite bank erosion occurs first, and subsequently bed degradation occurs.

7.2.6 Empirical Sediment Transport

In equation (7.24), the stream wise sediment transport rate can be computed using Leiwei bed-load equation (Chien et al., 1989). The equation is valid for a wide range of sediment particle grain size ($D_{50} = 0.25\text{-}23\text{mm}$) and the flow conditions ($H/D = 5\text{-}500$). After simplifying the final form of the equation can be given (Chen and Duan, 2006):

$$q_s = k_q U^4 \quad (7.26)$$

where U is the reach average velocity and k_q can be given by:

$$k_q = \frac{2D_{50}^{1.25}}{g^{1.5}H^{1.75}} \quad (7.27)$$

where g is the gravitational acceleration and H the reach average flow depth.

The first order derivative of q_s in the longitudinal direction can be expressed as (Chen and Duan, 2006):

$$\frac{\partial q_s}{\partial s} = k_q 4U^3 \frac{\partial U}{\partial s} \quad (7.28)$$

The conceptual diagram for the mass balance in the control volume is depicted in Fig. 7.4. The transverse bed load transport rate can be given as the ratio as derived by Ikeda (1989) by analyzing the force balance of the moving spherical particle on a plane inclined to both the longitudinal and transverse direction:

$$C_s = \frac{q_n}{q_s} = \tan \varphi + \frac{1 + \frac{C_L}{C_D} \mu}{\lambda \mu} \sqrt{\frac{\tau_c}{\tau}} \tan \psi \quad (7.29)$$

where φ is the angle where the bed shear stress deviates from the longitudinal direction, $\tan \psi$ the transverse bed slope, μ the coefficient of the kinetic friction between the particle and can be considered as 0.43 (Kikkawa et al., 1976; Ikeda, 1989), λ the ratio of the kinetic friction coefficient to the static friction coefficient equals to 0.59 (Ikeda, 1989), τ_c and τ are the critical and actual bed shear stresses respectively, C_D and C_L the drag and lift coefficients, can be considered as C_L/C_D the 0.85 for spherical particle (Kikkawa et al., 1976). Engelund (1974) expressed the direction of bed shear stress with the deviation angle can be given as:

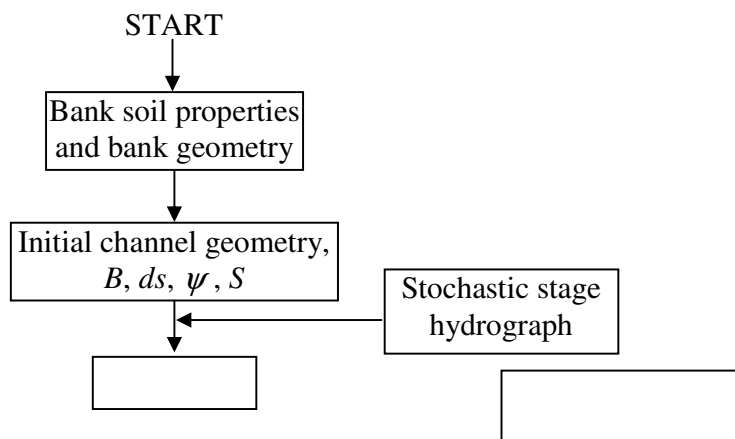
$$\tan \varphi = \frac{\tau_n}{\tau_s} = 7.0 \frac{h}{r_1} \quad (7.30)$$

where τ_n and τ_s are the bed shear stress in the transverse and longitudinal direction, h the flow depth, r_1 the local radius of curvature. After substituting the final form of the equation (7.29) can be rewritten as:

$$q_n = \left(7.0 \frac{h}{r_1} + 5.382 \sqrt{\frac{\tau_c}{\tau}} \tan \psi \right) q_s \quad (7.31)$$

7.2.7 Seepage Erosion Model

The detail of the mathematical modeling is already described in chapter 5. As the seepage erosion is independent of the other processes of bank erosion, the total erosion will be the algebraic sum of the fluvial erosion and the seepage erosion. The detail steps involved in the model is described in a flow chart (Fig. 7.5).



Variable	Computational procedure	Value used	Remarks
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Figure 7.5. Process-based flow chart for bank erosion.

Erodibility coefficient, (k_d)	Through field experiments, e.g., submerged jet test	Site specific
Critical shear stress (τ_c)	Through field experiments	Site specific
Boundary shear stress (τ_b)	Computed from average energy slope and depth of flow	variable
Coefficient of lift force (C_L')	Reported value	0.178 for uniform sediment
Depth averaged actual suspended sediment concentration (C)	Measured <i>in situ</i>	450 mg/lit
Equilibrium suspended sediment concentration (C_*)	Estimated from empirical equation, e.g., Yang's approach	600 mg/lit
Ratio of the tensile strength to the compressive strength of soil (r)	Computed through laboratory experiments	Reported value 0.10
Tensile strength (σ_t)	Determined through the laboratory experiments of undisturbed samples	5000 Pa
Sediment transport rate component in longitudinal direction (q_s)	Determined from empirical equation, which depends on reach average velocity (U)	--
Sediment transport rate component in transverse direction (q_n)	Fractional part of q_s estimated through empirical equation	--
Erosion rate of deposited sediment near toe, ($\bar{\xi}$)	Computed based on near bank sediment transport rate, critical and developed shear stress angle of repose and coefficient of lift	--
Coefficient of the kinetic friction between the particles (μ)	Based on particle shape	0.43
Ratio of the kinetic friction coefficient to the static friction coefficient (λ)	Reported value	0.59

Table 7.1. Details of critical parameters used in the mathematical model with brief description.

The functional relations in Eq. (6.10) can be determined from the laboratory experiments using lysimeter (Fox et al., 2007). These experiments are carried out with the soils from the composite river bank with similar stratification as found *in situ*. Various combinations of bank stratifications with cohesive and cohesionless soils were

investigated for different seepage gradient and the relationship between time and seepage gradient was determined for different bank stratifications. A brief descriptions of the critical parameters used in this model are presented in Table 7.1.

7.3 Data Collection

Detailed hydrographic and river bank survey was conducted during the moderate flow condition and during high flood condition (water level nearly 1m higher than the bank full discharge) for the years 2007 and 2008. The survey includes the data collection like bathymetry using GPS aided Echo-sounder, velocity profile through ADCP, river bed and bank soil samples, depth averaged water samples for suspended sediment concentration. The measuring devices were mounted on an engine propelled vessel. The accuracy of the GPS instrument was ± 1.5 meters. The details of the eroding bank has already described in Chapter 4.

7.4 Results and Discussions

The river bankline migration of the braided loop at Jamuguri site over the period from 2005 to 2008 was simulated for the verification of model prediction of river bank erosion. The satellite imagery supplemented with DGPS survey data was used to plot the annual bank erosion during the study period. Annual changes in river channel planform as observed during bankline survey are plotted in Fig. 7.6a. When compared with the observed banklines, simulated banklines (Fig. 7.6b) show a very good agreement for each year. The predicted magnitudes and shape of annual bank erosions agree closely with the observed one.

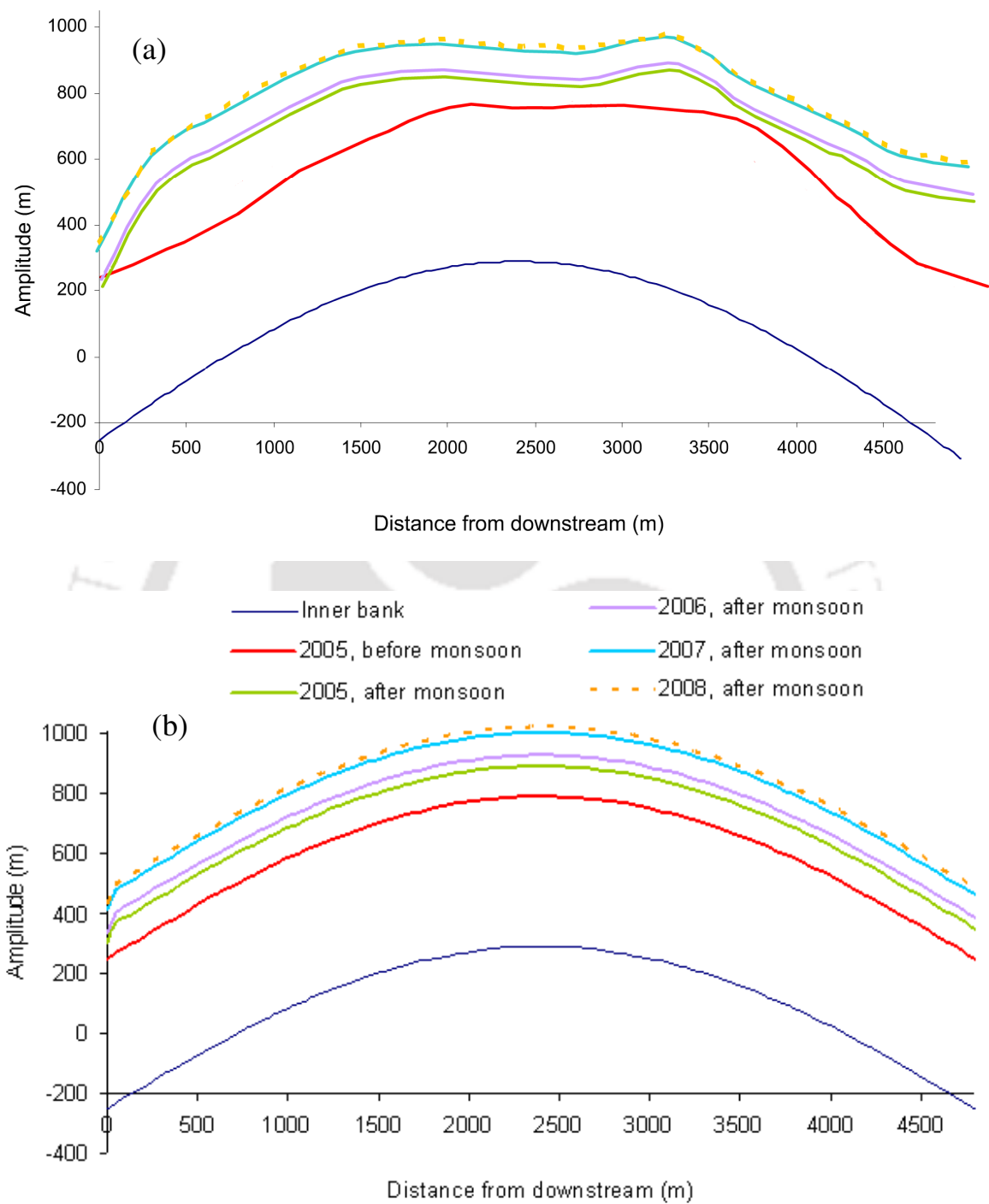


Fig. 7.6. Bankline migration of the channel from 2005 to 2008: (a) Observed and (b) Simulated

However, the local irregularities in the bankline can not be modeled with the simplified model geometry used in this study. Moreover, the bank erosion at the apex of the outer bend has been shown in Fig. 7.10 and discussed its characteristics in details in Section 7.5. Table 7.2 summarizes the magnitude of the observed and simulated maximum bank erosion.

Table. 7.2. Observed and computed maximum annual erosion at Jamuguri

Year	Observed erosion (m)	Computed erosion (m)
2005	216	185
2006	32	31
2007	110	87
2008	<20	19.7

The longitudinal gradients of the sediment transport rate for two different flow conditions are plotted in Fig. 7.7. Initially, during the low flow condition, the longitudinal sediment transport gradient was almost zero; indicating no stream-wise sediment transport. However, during the high discharge condition, the longitudinal sediment transport rate increases and the highest gradient of sediment transport rate was found near the inlet and outlet part of the channel. Near the apex of the channel, the gradient of sediment transport rate is almost zero; which indicates the uniform rate of sediment transport near apex.

The transverse bed slope variations along the stream-wise direction for different flow condition in the year 2008 are plotted in Fig. 7.8 using Eq. (7.9). It can be noticed that the initial transverse bed slope was almost flat before simulation start (average water depth = 3.0 m). However, the slope increased to its maximum during the peak flow, while

reach average water depth was about 8.0 m. At the end of the monsoonal period, the transverse bed slope again decreased to a value, slightly higher than the initial condition. This indicates the slope adjustment with the higher discharge condition. Further, it can be noticed from Fig. 7.8 that during low flow condition, the both, channel curvature and transverse bed slope are in same phase; however, during high flow condition, they are out of phase and the maximum transverse slope exists slightly downstream of the apex.

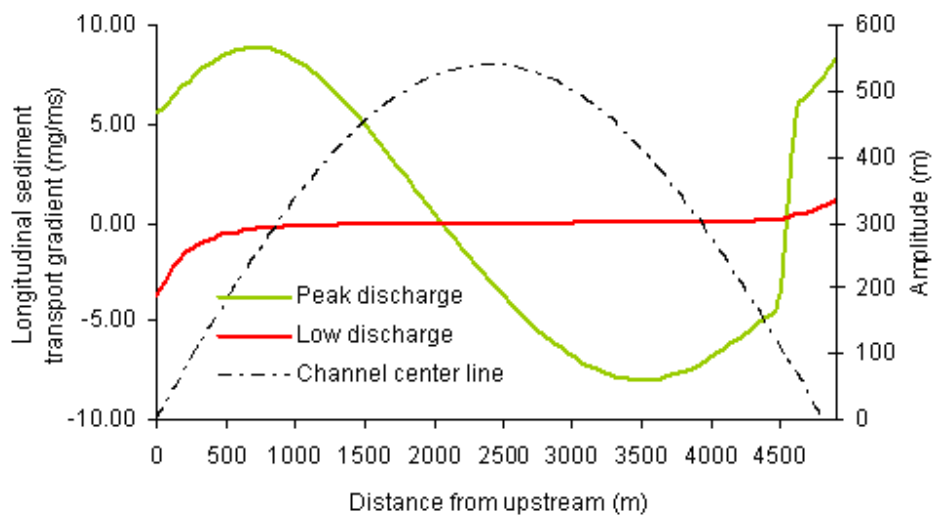


Fig. 7.7. Variation of longitudinal gradient of sediment transport during the low and high discharge period along the channel bend

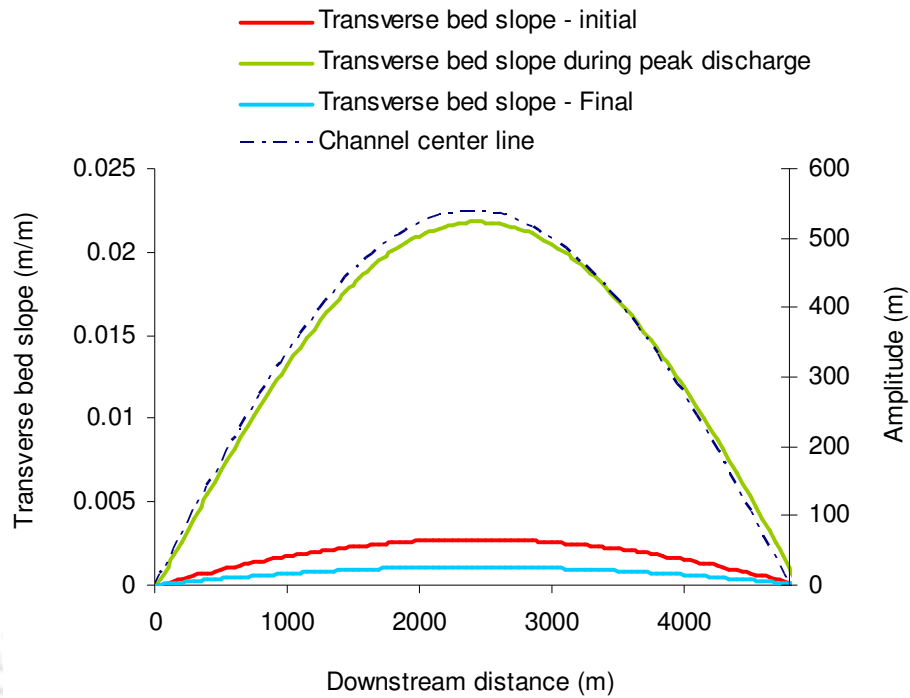


Fig. 7.8. Variation of transverse bed slope during the monsoon period along the channel bend

The simulated and observed channel transverse bed profiles for the two years at two transects (2006 and 2007) have been plotted in Fig. 7.9a-d. The figure indicates good agreements between the observed and the simulated one. It can be noted that, the present model considers the linear bed profile and hence the local morphological variations as observed *in situ* can not be modeled.

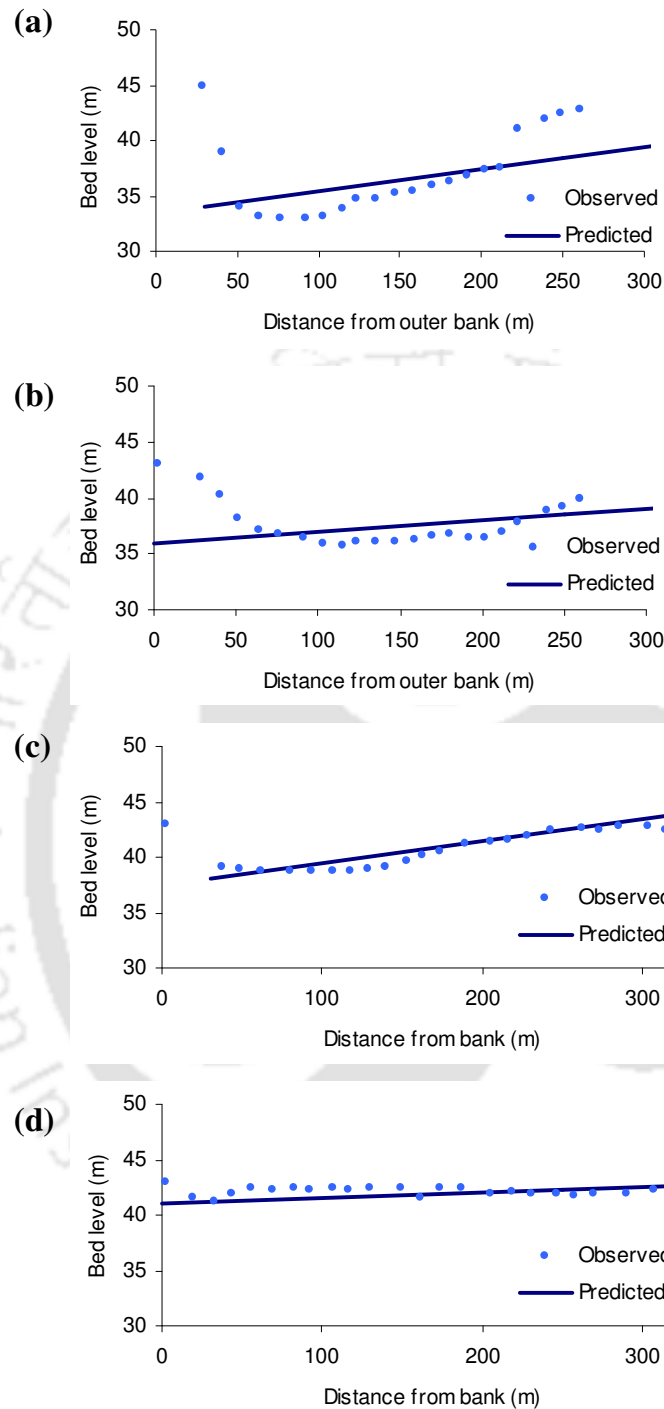


Fig. 7.9. Simulated and observed bed level during the peak discharge at the study reach at (a) bend apex in 2006, (b) upstream in 2006, (c) bend apex in 2007 and (d) upstream in 2007.

7.5 Characteristics of Bank erosion

7.5.1 Temporal Bank Erosion

The cumulative seasonal bank erosion at Jamuguri is shown in Fig. 7.10. As expected, the figure indicates that the major bank erosion occurs during the receding phase of the stage hydrograph. However, upto the 130 simulation days, the seepage erosion was negligible. After that period, there was two major recession in the flood wave and thus the significant seepage erosion was observed. Moreover, the figure indicates that the major erosion is due to the poor critical shear stress and hence the fluvial erosion is dominant.

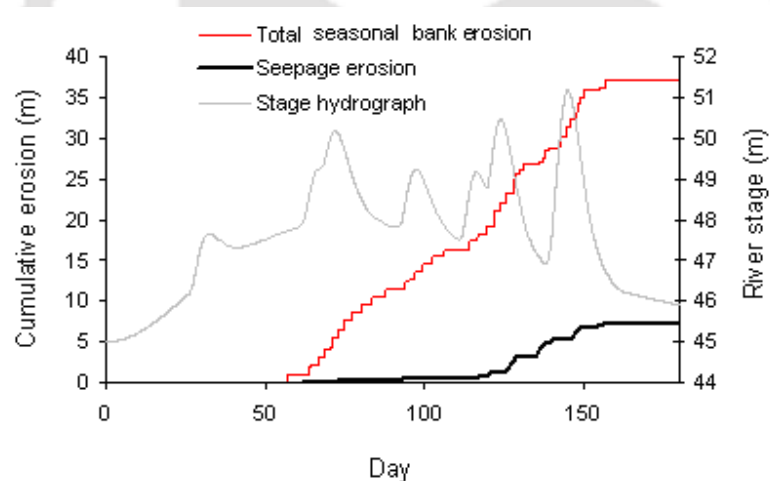


Fig. 7.10. Simulated cumulative bank erosion during a monsoon season

7.5.2 Spatial Variation of Bank Erosion

For the point specific bank erosion estimation, four monitoring points have been selected (Pt-1, Pt-2, Pt-3 and Pt-4) at Jamuguri (Fig. 7.11). The hydrographic data of year 2008 is used for this computation. The results are discussed in the following sections.

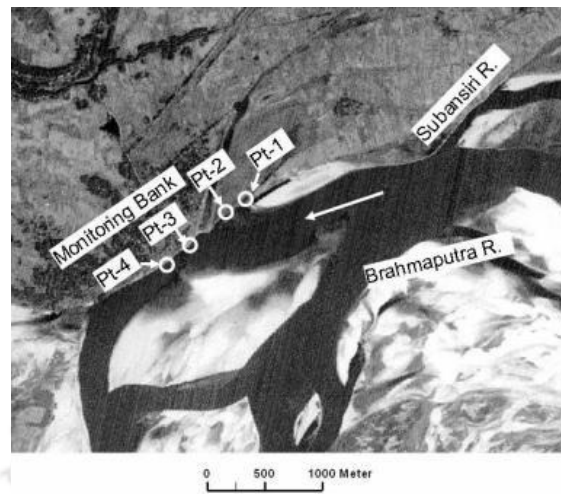


Fig. 7.11. Satellite imagery of Jamuguri site in the year 2008 (Pt-1, Pt-2, Pt-3 and Pt-4 are the monitoring points for measuring point specific bank erosion rates)

7.5.3 *Cantilever Stability*

The analysis of the cantilever failure for the monitoring point-4 is discussed here. The estimated factors of safety for three types of cantilever failure, as given in Fig. 7.12, indicate that among the four cases of mass failure, beam type of failure occurred three times and one case is for shear failure. Although it was reported by Thorney and Tovey (1981), that the shear failure is the most common type of cantilever bank failure. No case of tensile bank failure was found from this simulation. Moreover, the simulated factors of safety fall within the comparable limit for beam and shear type of failure cases.

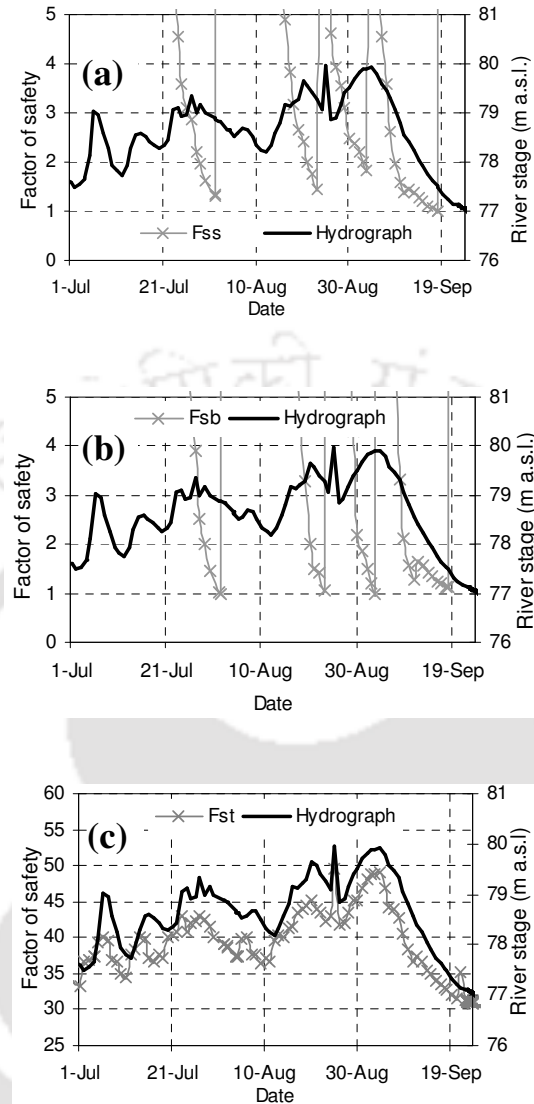


Fig. 7.12. Temporal variation of factors of safety in cantilever bank failure: (a) for shear failure, (b) for beam failure and (c) tensile failure.

The cumulative bank erosions at the four different monitoring points are compared with the observed bank erosion rate. Fig. 7.13 shows the simulated seasonal cumulative bank erosion. The sudden increase in the erosion indicates the cantilever mass failure. Results indicate the variability of the time of failure of the bank in spite of the monitoring points not so far apart. Fluvial erosion is the key

factor prior to the cantilever failure in this model. So the higher critical shear stress, lower the fluvial erosion and longer cantilever stability. Among the four monitoring points, the critical shear stress at Point-2 is quite high and about 10 N/m^2 . During the entire period no seasonal bank erosion was found at that point. It was also observed that the cantilever failure lagged well behind the flood peaks. Similar nature of bank failure was confirmed from the local people during the field survey. However, the lag time could not be compared due to the absence of the temporal *in-situ* bank erosion data (refer to Section 3.3.5).

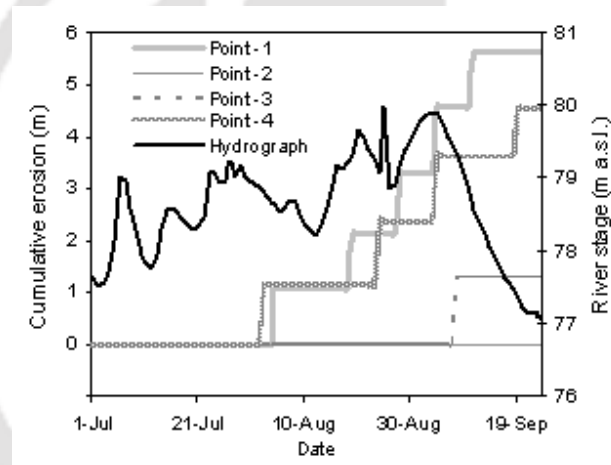


Fig. 7.13. Predicted cumulative bank erosion for four monitoring points

Total observed and computed seasonal bank erosion at the four monitoring points are shown in Table 7.3. The model predicts little higher for the observation points 1 and 4, but slightly under predicts for point 3. Apart from the various types of erosion considered in the present model, there may be seepage erosion, which is quite common for composite bank (Wilson et al., 2007; Fox et al., 2007). The under predicted result may be justified as the presence of seepage erosion, which was observed during the field survey.

There was no erosion at the monitoring Point-2. Interestingly, similar result is predicted by the model.

Table 7.3. Comparison of the observed and computed bank erosion at four monitoring points.

Location	Observed erosion (m)	Computed erosion (m)
Pt-1	4.8	5.63
Pt-2	0	0
Pt-3	2.85	1.28
Pt-4	3.72	4.54

7.6 Conclusions

The present analytical bank erosion model has been developed considering the near bank hydraulic processes and the seepage erosion commonly found in case of a composite river bank. The inclusion of the developed seepage erosion model in Chapter 6 in the total bank erosion model for composite river bank is a new concept. Thus, the fluvial erosion in a control volume has been modified from the model developed by Chen and Duan (2006). The developed model has been verified at a selected channel bend. The predicted annual bank erosion over 4 monsoon periods has been compared with the observed bank erosion. Moreover, two predicted transverse bed slope one at the apex and the other at the upstream have been compared with the observed one for the two years. The predictions show a good agreement with the observed one. However, the model needs general validation for more channel bends. The main objective of this study is to predict the annual bank erosion with minimum river survey data, general planform from the satellite imagery and the soil characteristics and bank information. The next chapter describes the

satellite imagery analysis of the major river channel bends of the Brahmaputra river and the validation of the stochastic prediction of the model for annual maximum bank erosion rate.



8.1 Introduction

The analytical model developed in Chapter 7 was validated in detail for 4 monsoon seasons at a channel bend of the Brahmaputra river. In reality, each and every parameter in the analytical model discussed in Chapter 7 contains various degree of uncertainty. It may be due to *in situ* measurements, prediction of the annual rainfall, extreme storm event, soil erodibility or the bed material size. Therefore it is more realistic to estimate the probabilistic bank erosion rather than the particular magnitude of bank erosion at any site. In this chapter, the stochastic bank erosion is predicted considering the uncertainties in the flood wave return periods, monsoonal conditions, bed material size, bank soil erodibility and channel bend configuration. The probabilistic characteristics of the annual bank erosion rate observed at the middle reach of the Brahmaputra river is estimated from the multi-date satellite imagery. Finally, the predicted probabilistic bank erosions are validated with the observed bank erosion. This chapter also briefly discusses about the procedure of satellite imagery analysis.

8.2 Satellite Imagery Analysis

Multi-date satellite imagery analysis was carried out to assess the annual bankline migration rate and the wavelength of the river bends in the middle reach of the Brahmaputra river. Yearly satellite imagery of 5 years from 2004 post monsoon to 2008 post monsoon was collected from an open source of NASA website (<http://glovis.usgs.gov/>). These satellite imagery were obtained from LANDSAT 7 satellite with ETM+ sensors. The raw imagery comprises of 8 bands: blue band wavelength of 0.45-0.52 μm (B1), green band wavelength of 0.52-0.60 μm (B2), red band having wavelength of 0.63-0.69 μm (B3), near infra red

band having wavelength of $0.76\text{--}0.90\ \mu\text{m}$ (B4), middle infra red band having wavelength of $1.55\text{--}1.75\ \mu\text{m}$ (B5), thermal band with wavelength of $10.40\text{--}12.50\ \mu\text{m}$ (B6), far infra red with wavelength of $2.09\text{--}2.35\ \mu\text{m}$ (B7) and panchromatic band having wavelength of $0.52\text{--}0.90\ \mu\text{m}$ (B8). The spatial resolution of the spectral bands (B1-B6) are 30 m, while thermal band is 60 m and that of panchromatic band is 15 m. Considering the higher order annual bank erosion occurred in the Brahmaputra river, the spatial resolution of imagery is enough to assess the annual bank erosion rate and the wavelength of the bends. The datum used for these imagery is WGS 84. The satellite imagery was analysed manually. For visual analysis, standard false colour composition (FCC) was prepared using the three bands B2, B3 and B4. The bands were assigned with blue, green and red colour, respectively. These entire data stack were reprojected into UTM-46N projection system using a satellite image processing software (Geomatica[®] 10.1). Geo-referencing is not required in this case since these available data are of geo-referenced form.

The yearly imagery were stacked in chronological order and the major channel bends were identified visually. Figs. 8.1 and 8.2 show the two channel bend formations and their annual variation. Due to mechanical problems in the sensors, black stripes were observed in the imagery. But, for estimating the bank line migration rate, the imagery can be used. The river bank lines were digitized using the image processing software. Fig. 8.1e shows the bank line migration of the channel bend shown in Fig. 8.1a-d. From the study area, 45 major channel bends have been selected and the annual average bank erosion rates were measured from the digitized banklines. The details of these bends with locations, annual bank erosion rate from 2005 to 2008, and the wavelength is given in Table 8.1a and b.

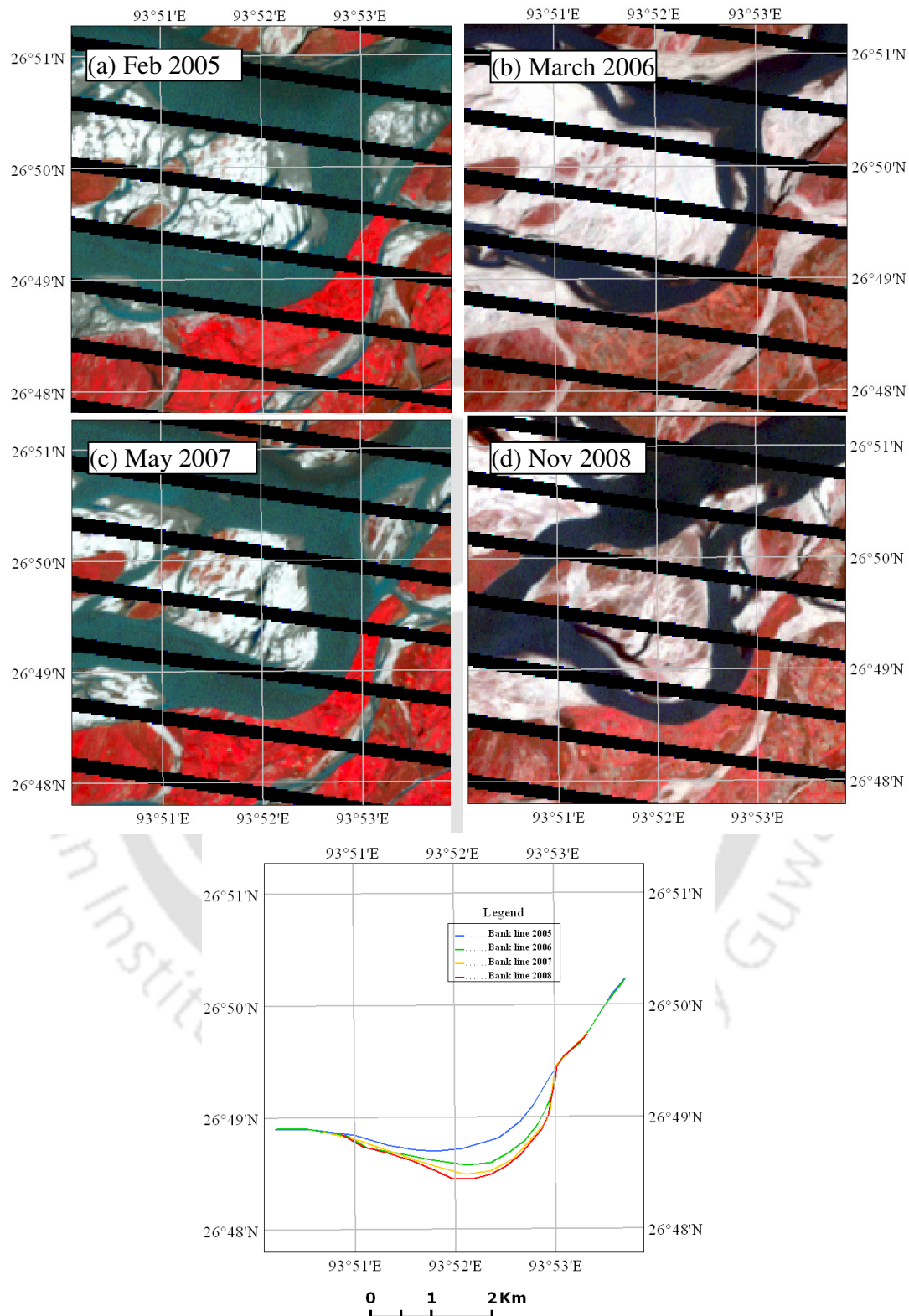


Fig. 8.1. River channel planform obtained from satellite imagery at a major bend: (a) 2005, (b) 2006, (c) 2007, (d) 2008, and (e) the bank line migration

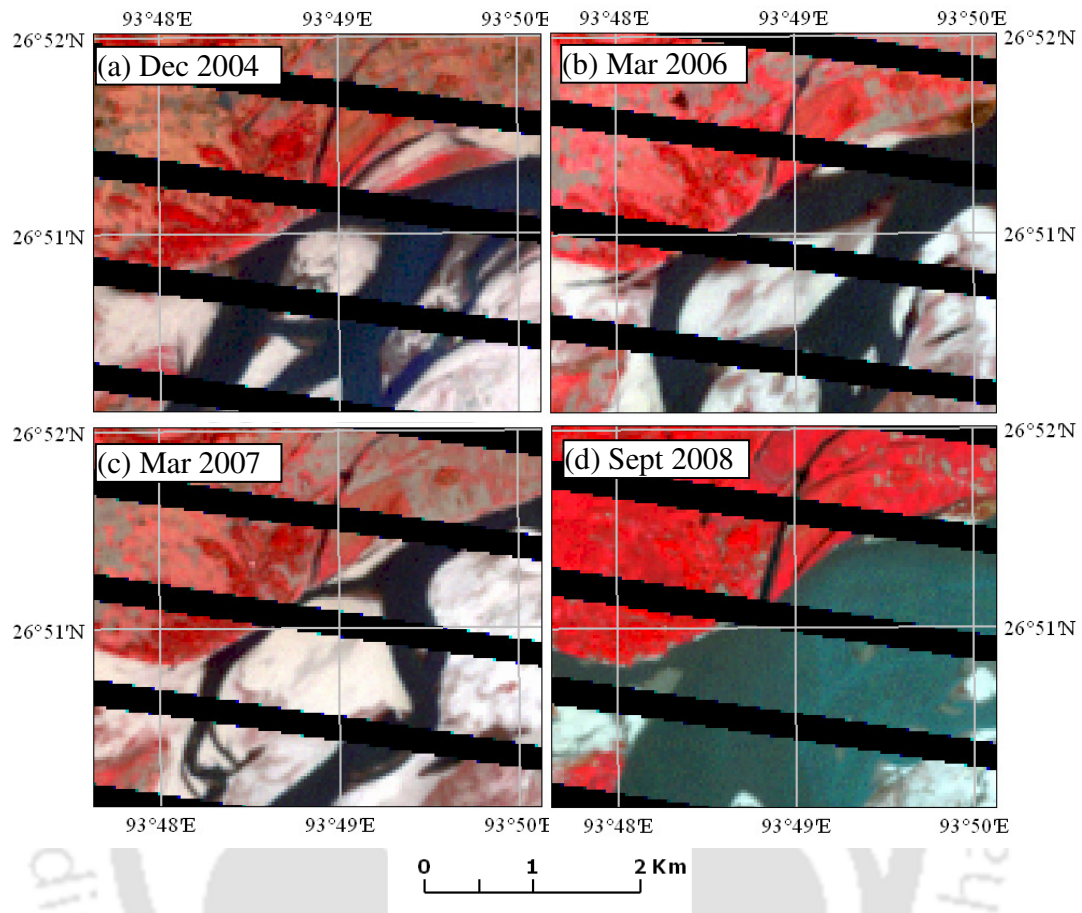


Fig. 8.2. River channel planform obtained from satellite imagery at Jamuguri: (a) Dec 2004, (b) Mar 2006, (c) Mar 2007 and (d) Sept 2008.

The annual bank erosion rate has been divided into two regions. The first one is the region upstream of Guwahati (Table 8.1a); whereas the second one is the region downstream of Guwahati (Table 8.1b). The average annual bank erosion is about 87 m at the upstream of Guwahati, whereas, the average annual bank erosion is found to be about 68 m at its downstream. The analysis also shows that the most of the wavelengths of the braided loop lie in between 9000 and 12000 m with an average of $\sim 10,000$ m.

Table 8.1a. Observed annual bank erosion at Brahmaputra river, India, at upstream of Guwahati.

Sl. no.	Longitude	Latitude	Annual average erosion (m)	Wave length of the bend (m)
1	94 ⁰ 12'18" E	26 ⁰ 55'47" N	276	11000
2	94 ⁰ 03'33" E	26 ⁰ 49'02" N	27	7750
3	94 ⁰ 48'38" E	27 ⁰ 31'26" N	81	9670
4	93 ⁰ 52'36" E	26 ⁰ 48'55" N	116	8950
5	93 ⁰ 54'39" E	26 ⁰ 47'51" N	64	8000
6	93 ⁰ 55'07" E	26 ⁰ 46'26" N	303	8000
7	93 ⁰ 48'34" E	26 ⁰ 43'48" N	0	14000
8	93 ⁰ 46'21" E	26 ⁰ 50'52" N	0	10900
9	93 ⁰ 53'55" E	26 ⁰ 52'51" N	0	8100
10	93 ⁰ 44'21" E	26 ⁰ 42'04" N	7	8900
11	93 ⁰ 20'24" E	26 ⁰ 41'43" N	167	6500
12	93 ⁰ 13'29" E	26 ⁰ 43'12" N	26	9290
13	93 ⁰ 00'23" E	26 ⁰ 35'01" N	0	9800
14	93 ⁰ 01'37" E	26 ⁰ 38'25" N	56	11200
15	92 ⁰ 54'03" E	92 ⁰ 54'03" N	30	9670
16	93 ⁰ 00'05" E	26 ⁰ 35'13" N	75	9320
17	92 ⁰ 53'49" E	26 ⁰ 35'00" N	85	7260
18	92 ⁰ 47'23" E	26 ⁰ 33'15" N	16	8588
19	92 ⁰ 34'37" E	26 ⁰ 37'48" N	104	9030
20	92 ⁰ 41'52" E	26 ⁰ 32'21" N	42	7910
21	92 ⁰ 30'28" E	26 ⁰ 34'18" N	85	5100
22	92 ⁰ 26'18" E	26 ⁰ 35'01" N	43	9350
23	92 ⁰ 19'47" E	26 ⁰ 26'34" N	150	10000
24	92 ⁰ 13'08" E	26 ⁰ 28'06" N	110	5624
25	92 ⁰ 16'04" E	26 ⁰ 26'01" N	200	8980
26	92 ⁰ 07'48" E	26 ⁰ 18'30" N	220	9800
27	91 ⁰ 54'44" E	26 ⁰ 14'18" N	30	14340
28	91 ⁰ 54'18" E	26 ⁰ 16'18" N	130	10900
29	91 ⁰ 46'53" E	26 ⁰ 12'05" N	35	14850
30	91 ⁰ 44'42" E	26 ⁰ 13'15" N	20	6650
31	92 ⁰ 07'46" E	26 ⁰ 18'40" N	200	9900

Table 8.1b. Observed annual bank erosion at Brahmaputra river, India, at downstream of Guwahati.

Sl. no.	Longitude	Latitude	Annual average erosion (m)	Wave length of the bend (m)
1	91 ⁰ 43'25" E	26 ⁰ 11'33" N	10	6350
2	91 ⁰ 36'19" E	26 ⁰ 11'50" N	70	10500
3	91 ⁰ 29'39" E	26 ⁰ 06'58" N	0	26400
4	91 ⁰ 05'54" E	26 ⁰ 15'29" N	220	10824
5	90 ⁰ 52'33" E	26 ⁰ 13'10" N	70	12370
6	90 ⁰ 43'26" E	26 ⁰ 08'55" N	32	9000
7	91 ⁰ 19'30" E	26 ⁰ 10'22" N	60	10314
8	90 ⁰ 52'25" E	26 ⁰ 13'26" N	45	10100
9	90 ⁰ 32'31" E	26 ⁰ 13'52" N	15	11400
10	90 ⁰ 22'17" E	26 ⁰ 06'58" N	200	12060
11	90 ⁰ 16'49" E	26 ⁰ 03'51" N	55	10200
12	90 ⁰ 10'43" E	26 ⁰ 02'27" N	22	10600
13	90 ⁰ 30'46" E	26 ⁰ 14'32" N	65	14200
14	91 ⁰ 57'07" E	26 ⁰ 17'35" N	95	8310

The annual bank erosion rate from these 45 river bend data was analyzed for the cumulative distribution. The annual average bank erosion data are divided into a number of classes from zero to 150 m with 10 m increment. In the parametric study various cases are considered and the cumulative distribution of the predicted annual bank erosion rate compared with the observed distribution of the annual bank erosion. This comparison can indicate the ability of the analytical bank erosion model to predict the stochastic annual bank erosion with minimum *in situ* data in the alluvial river bends.

8.3 Parametric Analysis

The influence of the various parameters on the result of the model has been investigated in this section. The parameters considered include: the monsoonal condition (dry, normal or

wet), return period of flood waves, deflection angle (θ_0), longitudinal slope of river channel (S), bed material size (D_{50}), and the random probabilistic field of critical shear stress.

The basin average normal monsoonal rainfall (2068 mm) was estimated from the 30 years rainfall collected from open source of NASA (see Chapter 3 for detail). The range of the monsoonal average rainfall (1298-2960 mm) was equally divided into three sub-ranges (1298-1812 mm; 1812-2324 mm; and 2324-2960 mm). The lowest one denotes the range of 'Dry' monsoonal season, the middle one 'Normal' monsoonal season, while the rest denotes the 'Wet' monsoon seasons. For the estimation of the annual maximum bank erosion, the maximum rainfall magnitudes in each sub-range were considered. Thus, in the present analysis, the dry monsoonal rainfall is considered as 1812 mm, the normal monsoonal rainfall is considered as 2324 mm, while, the wet monsoonal rainfall is considered as 2960 mm.

8.3.1 *Generating Random Probabilistic Field for Critical Shear Stress*

The critical shear stress (τ_c) determined through the submerged jet test apparatus along the river Brahmaputra has the range 0.1-15 Pa (refer to chapter 5). In order to find out the appropriate random field, *in-situ* critical shear stress data were fitted with various probability distribution functions. The distribution of the critical shear stress was found to follow the Gaussian probability function (Fig. 8.3). The 58 sample points were grouped from 0-15 Pa with increment of 1 Pa. The co-relation coefficient is found to be 0.99 for this distribution. The relationship between the critical shear stress and the erodibility coefficient developed in Chapter 5 is used here for estimation of k_d .

The stochastic bank erosion estimation has been carried out considering the various probabilities of exceedence of critical shear stress and return period of the flood waves. The details of the parametric study carried out are listed in Table 8.2.

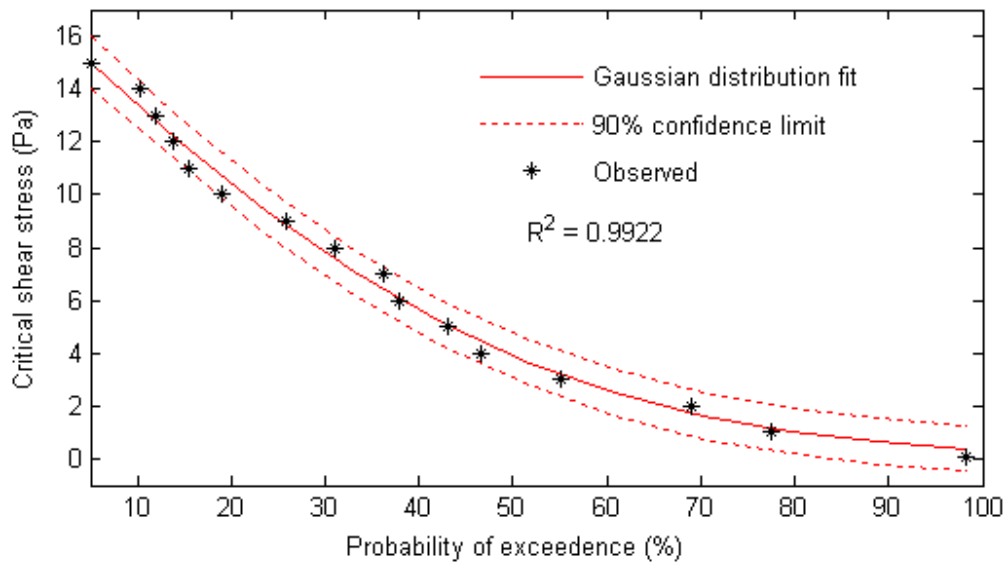


Fig. 8.3. Distribution of the critical shear stress of the bank fine soil

Table 8.2. Lists of cases considered for the stochastic bank erosion estimation

Case	Longitudinal bed slope	Deflection angle	Monsoon condition	Median sediment size (mm)
1	1 in 7500	35 ⁰	Dry	0.2
2	1 in 7500	35 ⁰	Normal	0.2
3	1 in 7500	35 ⁰	Wet	0.2
4	1 in 7500	55 ⁰	Normal	0.2
5	1 in 10000	35 ⁰	Wet	0.2
6	1 in 12000	35 ⁰	Wet	0.2
7	1 in 7500	35 ⁰	Normal	1.0

8.3.2 Effect of basin-average monsoon rainfall

Fig. 8.4 demonstrates the predicted bank erosion during the dry monsoon season, the seasonal bank erosion is highly influenced by the various return periods of flood waves. However, for the higher critical shear stress (probability of critical shear stress lower than 40%), the maximum annual bank erosion is dominated by seepage erosion. The seepage erosion rate found from the model simulations ranges between 1 and 6 m depending on the return period of the flood waves. However, the annual erosion rate in the composite banks with fine soils of low critical shear stress and high erodibility coefficient are significantly affected by the return period of flood waves. The higher flood wave return period, more bank erosion rate.

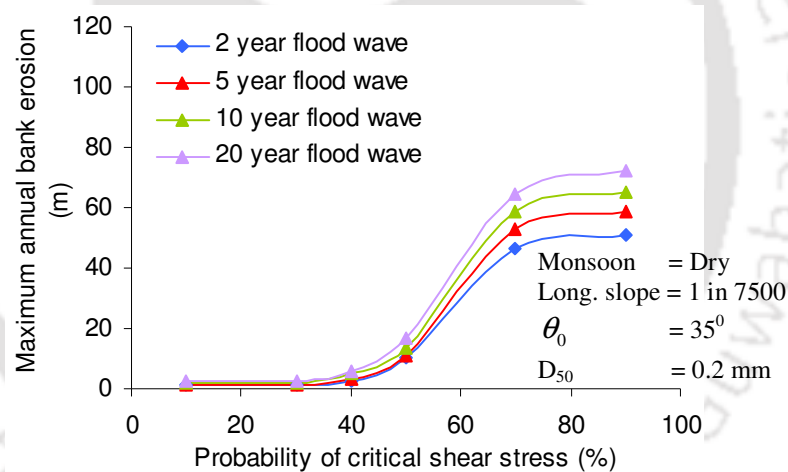


Fig 8.4. Variation in maximum annual bank erosion rate during 'Dry' monsoonal season.

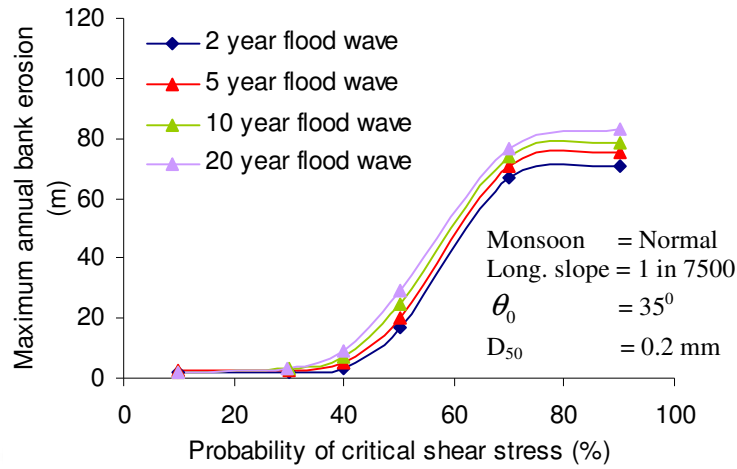


Fig 8.5. Predicted maximum annual bank erosion during ‘Normal’ monsoonal season.

The influence of normal monsoonal season with various return periods of flood waves is shown in Fig. 8.5. Interestingly, the basic nature of the maximum annual erosion rate is similar to the dry monsoon year. However, the predicted bank erosion rate is higher than that of dry monsoon condition. Although, the maximum annual bank erosion rate does not vary significantly with different return period of flood waves. Moreover, Figs. 8.4 and 8.5 show that the maximum annual bank erosion reaches to its maximum limit. This indicates that even with very low critical shear stress in the bank fine soils, the annual maximum bank erosion will not increase beyond this limit.

In case of wet monsoon year, there is less influence of the different return period of flood waves (Fig. 8.6). The limit of maximum annual bank erosion is higher than that of normal and dry condition. The predicted maximum annual erosion rate is about 120 m with a 20-year return period of flood wave. Moreover, the annual erosion has not reached to its maximum limit. This indicates with lower critical shear stress of the bank fine soils higher

bank erosion rate is expected for wet monsoon years. The predicted maximum annual bank erosion increases upto 80% from dry to wet monsoon season.

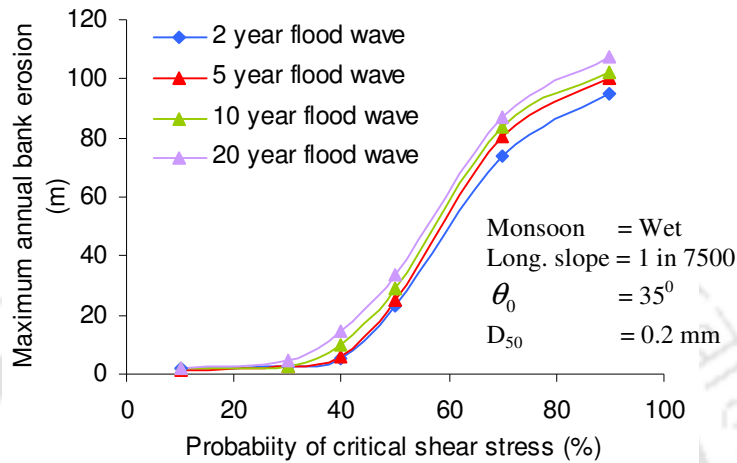


Fig 8.6. Predicted maximum annual bank erosion during ‘Wet’ monsoonal season.

8.3.3 Effect of Deflection angle

The influence of the deflection angle on the maximum annual bank erosion is shown in Fig. 8.7. The trend is similar to that shown in Fig 8.5. The only difference is that, earlier one was the simulated results with the deflection angle 35° , while in the present case, the deflection angle is considered as 55° . The results indicate the deflection angle has less influence on maximum bank erosion in between 35° and 55° . However, marginally higher bank erosion is found by increasing the deflection angle from 35° to 55° .

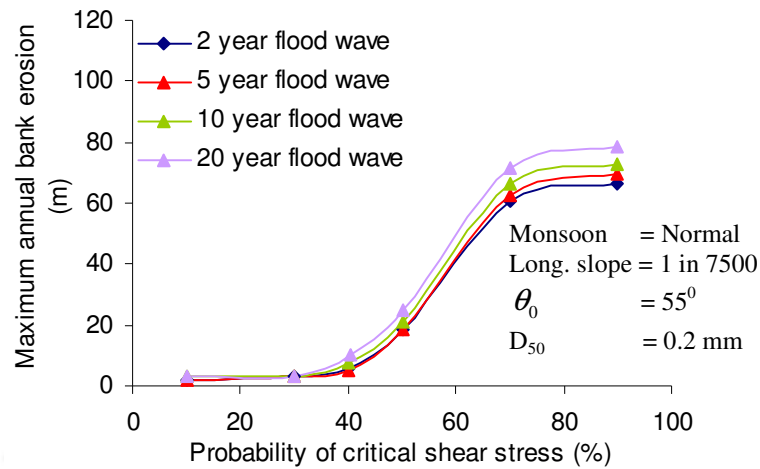


Fig 8.7. Predicted maximum annual bank erosion for 55° deflection angle of the river bend.

8.3.4 Effect of Longitudinal Slope

The influence of the longitudinal bed slope on annual bank erosion rate is shown in Figs. 8.6, 8.8 and 8.9. For the longitudinal slope of 1 in 10000 (Fig. 8.8), the annual bank erosion was found to be lesser than that with 1 in 7500 slope (Fig. 8.6). Although the maximum annual erosion is nearly same for the both slopes, the bank erosion is found to be steadily decreased to the minimum value. However, for the longitudinal slope of 1 in 12000, it shows further decrease in maximum bank erosion rate (Fig. 8.9). In the both cases, the influence of different return of flood wave was found to be less. The similar finding on the longitudinal slope effect on bank erosion was also reported by Hanson and Simon (2001).

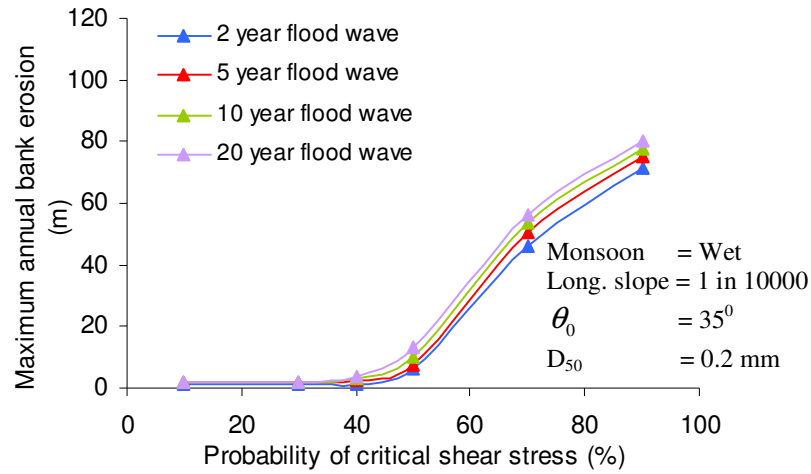


Fig 8.8. Predicted maximum annual bank erosion for longitudinal slope of 1 in 10000.

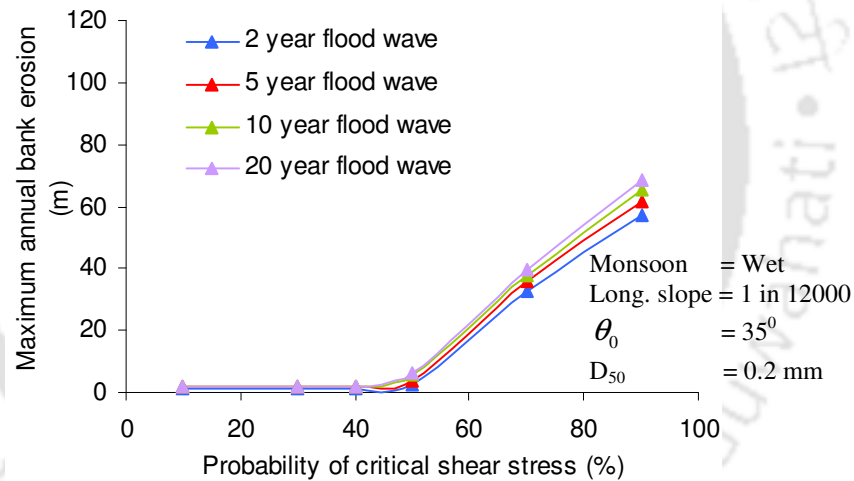


Fig 8.9. Predicted maximum annual bank erosion for longitudinal slope of 1 in 12000.

8.3.5 Effect of Bed Material Size (D_{50})

Fig. 8.10 demonstrates the influence of the bed material size (d_{50}) on the maximum annual bank erosion. The sediment particle size considered for the present case is 1 mm, which is the higher limit of the sediment size found in the Brahmaputra river (Ojha et al.,

2004). It can be noticed that the influence of particle size is insignificant on maximum bank erosion for these ranges of particle sizes (0.1-1 mm). The sediment transport capacity during the monsoon season is very high. For this reason, the bank erosion does not depend on the bed material size but on the sediment transport capacity of the river. However, during the peak discharge condition, the longitudinal sediment transport gradient is higher for larger particle size (Fig. 8.11). In Fig. 8.11, due to boundary effect, there is a sudden change in the sediment transport gradient (from -5.0 mg/ms to +5.0 mg/ms) near the downstream boundary.

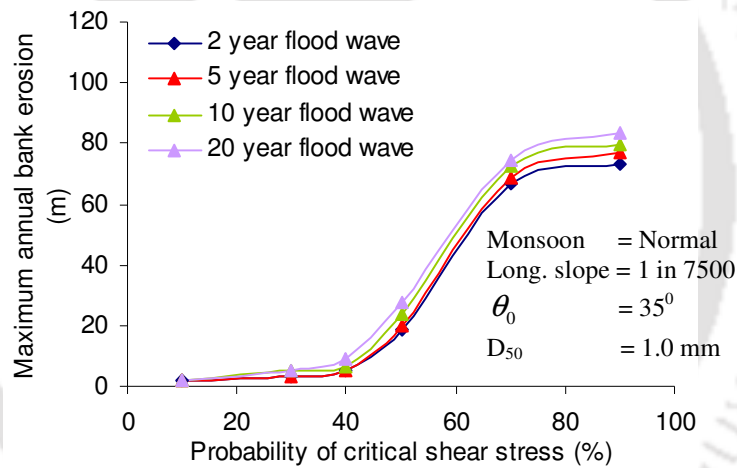


Fig 8.10. Predicted maximum annual bank erosion for median bed material size of 1.0 mm

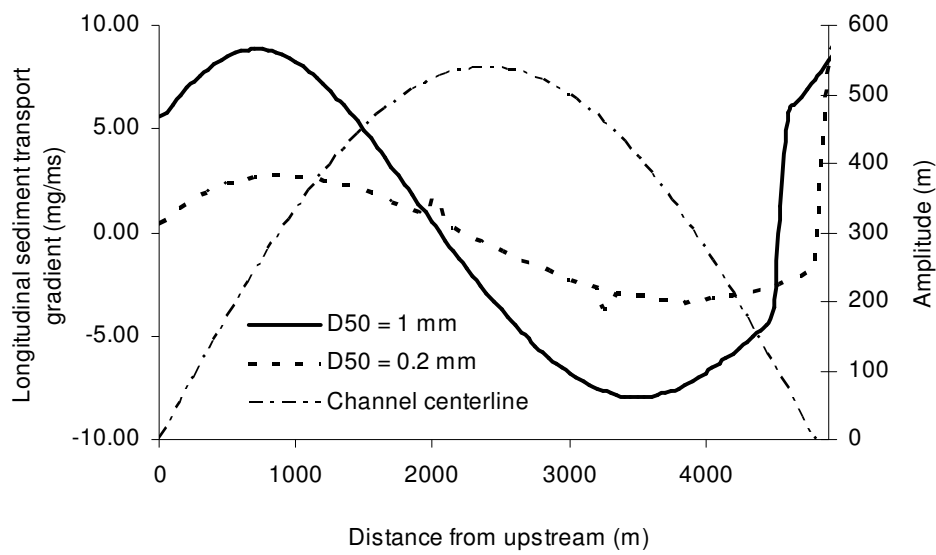


Fig 8.11. Simulated longitudinal gradient of the sediment transport rate during peak discharge for two different bed material size.

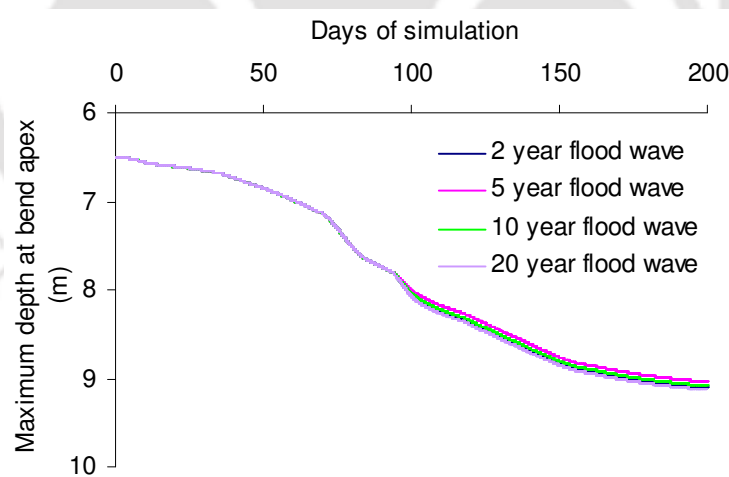


Fig 8.12. Simulated maximum flow depth at the bend apex for different flood wave return periods.

Moreover, the simulated results for different flood wave return periods indicate that near bank bed level is almost same for different return period of flood waves (Fig. 8.12). This depends on the bank soil and bed material characteristics. During the field survey, it has been found that the maximum depth of the river near bank is almost same and the average near bank depth is about 9.0 m.

8.4 Evaluation of Stochastic Bank Erosion Prediction

The cumulative distribution of the bank erosion simulated by the model is used to evaluate the model performance on stochastic prediction. Therefore, all the simulation results mentioned in the pervious sections are considered as the sample dataset. The cumulative distribution is shown in Fig 8.13 as the box plots with its average value. For its evaluation, observed annual bank erosion rate obtained from satellite imagery analysis (Section 8.2) is considered. The envelop of the predicted bank erosion is shown in the figure with dash lines. The cumulative probability distribution of the observed bank erosion was computed and plotted in the same figure as a line. Interestingly, the stochastic erosion predict follows fairly the observed distribution, close to the upper bound of the envelop.

However, the satellite imagery analysis indicates the annual bank erosion, which also includes the erosion during the non-monsoon periods. During the non monsoon periods, the bank erosion is mainly due to sub aerial erosion and bank collapse due to poor cohesion. Moreover, the average bank height was considered for the prediction of the stochastic bank erosion. Interestingly, the orders of the observed and predicted bank erosion match fairly well. Thus, the stochastic bank erosion prediction from the present model can be used to identify the vulnerable river reaches under the threatening of the bank erosion.

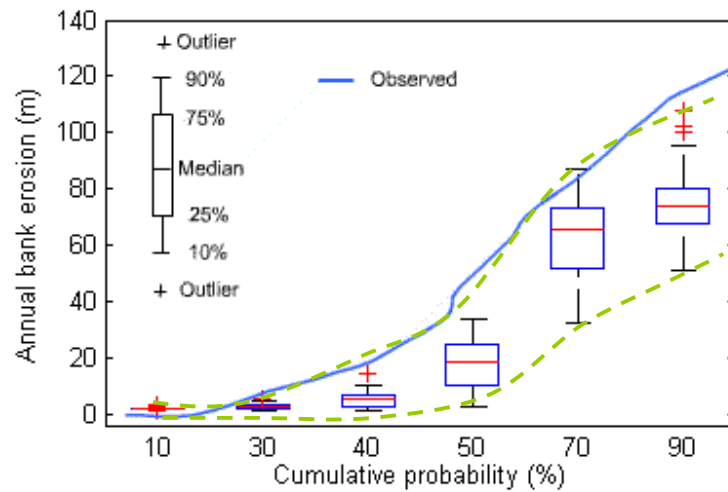


Fig 8.13. Simulated and observed cumulative probability of annual bank erosion in channel bends (Dash lines indicate envelop of the predicted bank erosion).

8.5 Conclusions

The major findings from the stochastic nature of the bank erosion in the middle reach of the Brahmaputra river are given below:

- The magnitude of the bank erosion increases nearly 80% from dry to wet monsoon years.
- For lower probability of the critical shear stress (i.e., high critical shear stress) of the bank material, seepage erosion dominates over the fluvial erosion and its mass failure.
- The magnitude of the bank erosion increases with increasing flood wave return period and longitudinal bed slope.
- For the different deflection angles of the river bends found in the middle reaches of the Brahmaputra river, there is no significant effect of them on bank erosion rate.

- Particle size of the bed material does not have significant affect on the magnitude of the annual bank erosion; although during high floods, the longitudinal sediment transport gradient varies with bed material size (d_{50}).
- The predicted bed level at the bend apex indicates that the maximum bank height is dependent only on the bank soil erodibility and the bank stratifications. The observed bank heights at different reaches support the model findings.

Comparison between the observed and predicted probabilistic bank erosion indicates that the developed analytical bank erosion model is able to predict the range of the annual bank erosion in the river bends. The observed bank erosion fairly matches with the upper bound of the predicted bank erosion envelop. The measured annual bank erosion covers the erosion during the lean periods due to sub-aerial processes, soil desiccations, and slope instability and other local erosion processes. Moreover, the average height of the banks at the selected river bends is considered. Considering all these factors, the developed bank erosion model is useful in prediction of the bank erosion ranges and vulnerability mapping in composite river banks.

9.1 A Brief Review of the Work Done

This thesis has described an attempt to estimate stochastic erosion in composite river banks of an alluvial river. The conclusion drawn from the research is summarized in the following sections.

9.1.1 Review of Literatures

As discussed in Chapter 2, previous researchers have contributed in developing the mathematical model for the cohesive or non-cohesive river banks incorporating different bank erosion processes. The models were well validated with the laboratory flume or natural river data. However, the reviews of their works show some drawbacks for the mathematical modeling for composite river bank erosion and can be highlighted in the following paragraphs.

Seasonal hydrograph characteristics for a large river basin are required to be analysed from the historical stage records and the new method is required to be developed for stochastic hydrograph in this river basin for different return periods of flood waves.

The erodibility of the fine soils is the key component for the fluvial erosion of the composite river banks. Therefore the soil erodibility and their spatial variations and its distribution functions are required to be determined *in situ*.

Seepage erosion from the composite river bank and its subsequent collapse is required to be characterized. A mathematical formulation is required to be developed to quantify the bank seepage erosion for different flood magnitudes.

Finally, a mathematical hydrodynamic and morphological model describing the major bank erosion processes is required to be developed for estimating the composite river bank

erosion. This model should have capability to predict the composite bank erosion considering various uncertainties.

9.1.2 Study Area and Data Collection

The details of the study area along the Brahmaputra river considered in the present research work have been discussed in Chapter 3. The chapter has briefly described about the basic bank soil, hydraulic and hydrodynamic data collection and analysis of these data. The analysis of the hydrographic data indicates the presence of the multi-thread flow in the river channel of the Brahmaputra. However, the hydrographic data analysis also indicates that the secondary current strength is negligible for the mildly sinuous river bends. Thus, for simplification, the secondary current component can be ignored for mathematical modeling of these river bends. The transverse bed slope at a river bend can be approximately estimated by using Odgaard's approach.

9.1.3 Generation of Stochastic Hydrograph

The historical stage hydrograph analysis of the Brahmaputra river has been carried out for two gauging stations at the middle reach. The characteristics of the flood waves such as flood lift, time of occurrence, response time period, number of flood waves and monsoonal lift have been analyzed; as these are the key components for the bank erosion. The seasonal hydrograph is decomposed into two components: flood waves and the monsoonal response. Maxwell distribution functions were used for approximating the flood waves and monsoonal responses. The monsoonal lifts were found to have close relationship with basin average monsoonal rainfall.

The proposed method of generation of the synthetic hydrograph is able to capture the both characteristics: maximum stage level and the rate of change of river stage. The generated

hydrographs was evaluated for two gauging stations for three different flood wave return periods.

A new equation is developed for the generation of synthetic stage hydrographs with the designed return period of the flood waves and the basin average monsoonal rainfall:

$$h(t) = h_b + f_1(R_s) \left[\frac{t}{k_s} \exp \left(1 - \frac{t}{k_s} \right) \right]^{r_s} + f_2(T) \left[\frac{t}{k_f} \exp \left(1 - \frac{t}{k_f} \right) \right]^{r_f} + \sum_{i=1}^{n-1} h_f^i \left[\frac{t}{k_f} \exp \left(1 - \frac{t}{k_f} \right) \right]^{r_f} \quad (4.6)$$

The proposed method of generation of synthetic hydrograph can capture both maximum stage and the rate of change of river stage during the monsoon season.

9.1.4 Erodibility of the Fine Bank Soils

The soil erodibility parameter of fine soils from the composite banks of the Brahmaputra river were measured *in situ* using submerged jet test apparatus. The analysis of the results indicates the banks of the Brahmaputra mostly fall within erodible to very erodible classes. The ANNOVA analysis of the critical shear stress and the erodibility coefficients indicate that these parameters vary significantly from one site to another, and thus for erosion estimation *in-situ* erodibility parameters need to be measured locally. Layer-wise analysis of the erodibility parameters in a particular location indicate that the variation of the erodibility parameters is not significant, and thus average erodibility parameters of the soils from various layers can be considered for estimating erosion in a bank site.

From the *in situ* experiments of the submerged jet test, a new relationship for the bank fine soils of Brahmaputra river is derived:

$$k_d = 3.16 \tau_c^{-0.185} \quad (5.10)$$

9.1.5. Bank Seepage Erosion: Experiments and Modeling

The seepage erosion is a bank erosion process often found in the composite river banks. The controlling factors for seepage bank erosion have been discussed in details in the Chapter 6. The lysimeter experiments were carried out for characterizing the controlling parameters of seepage erosion. The analysis of the results shows the inverse power relationships between the seepage gradient and time of bank collapse. Moreover, the increase in number of semi-permeable layers significantly increases the time of bank collapse, thus reducing the bank erosion. Bank height was found to be insignificant for seepage erosion.

The developed functional relations from the lysimeter were used for modeling daily seepage bank erosion for flood waves of different return periods. The development of the mathematical seepage erosion model is a new concept presented in this chapter. Model prediction shows that with the increase in the number of semi-permeable layers, total seepage erosion decreases significantly. The range of model predictions for seepage bank erosion agrees with the one found from the field measurements.

Through 67 lysimeter experiments of the bank soils, the relationship between the time of bank collapse with the seepage gradient is proposed:

$$t_b = 81.87 \times i^{-1.426} \quad \text{for case-1} \quad (6.12)$$

$$t_b = 150.57 \times i^{-1.356} \quad \text{for case-2} \quad (6.13)$$

$$t_b = 249.28 \times i^{-1.416} \quad \text{for case-3} \quad (6.14)$$

The coefficients in the above equations change with the bank stratifications and thus the bank stratification controls its collapse time due to seepage erosion.

9.1.6. Analytical Modeling of Composite Bank Erosion

An analytical bank erosion model has been developed in Chapter 7, considering the near bank hydraulic, morphological and bank erosion processes, commonly found in a composite river bank. The developed model was validated in detail for a selected river bend over 4 monsoon season. The predicted results show a good agreement with the observed one, when detailed morphological data are available for initial model setup.

By equating the near bank control volume of an eroded composite river bank, the following equation is developed.

$$\omega_b = -\bar{\xi} \frac{2}{b} \frac{dh}{\cos \theta} - \omega' \frac{2h}{b} \frac{(1-P)}{(1-P')} \frac{\rho_s}{\rho_{sand}} - \frac{2}{b(1-P')} \frac{\rho_s}{\rho_{sand}} \left(\frac{\partial q_s}{\partial s} b + q_n \right) \quad (7.25)$$

9.1.7 Stochastic Bank Erosion

Considering the probabilistic distribution of bank soil erodibility and flood waves of return periods, monsoonal season, longitudinal bed slope, the validated model has been simulated to estimate stochastic annual bank erosion rate. In these simulation studies, variations in bed material size, geometry of the bend and monsoon condition were considered, based on the variations observed in the middle reach of the Brahmaputra river. The annual bank erosions of the major river bends were measured from the satellite imagery from 2005-2008. The comparison between the observed and the predicted bank erosion indicates that the developed model is able to predict the range of annual bank erosion in the river bends within the study reach.

Moreover, the present research indicates the following findings:

- i) The instability in the braided river system is due to high sediment flux from the bank erosion, which in turn controls the morphology of the braided river.
- ii) Bank protection design works is required to understand the detail bank erosion process prior to its design. The present findings indicates that the seepage protection and bank toe protection is essential for the bank protection works in a composite river bank.

9.2 Recommendations for Further Research

The present research work can further be extended in following ways:

- In the present model, a site average lumped critical shear stress of the bank soil is considered. The distribution of the critical shear stress can be considered for future study.
- Even in the composite river bank, partly cohesionless banks are observed in some sites. The present model can be further developed to include the non-cohesive bank erosion processes, which in turn controls the hydrodynamic and morphological characteristics in a river bend.
- The bank erosion processes in a single channel bend can be further extended for modeling the morphology of the braided river loops by incorporating the stream power concepts.
- Further validation of the model can be carried out for other rivers with composite banks.

- Moreover, the present analytical model can be replaced with a 2-dimensional hydrodynamic and morphological model addressing the bank erosion mechanism considered in the present study.



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Appendix - A

A.1 Submerged jet test apparatus



Fig. A.1. Submerged jet test apparatus: (a) Transparent top plate, (b) Jet tube with point gauge and deflector plate, (c) Top view of submergence tank (d) Side view of submergence tank

(a)



(b)



Fig. A.2. (a) Submerged jet apparatus with all the assembly, and (b) adjustable water head tank

Microsoft Excel - Jet_jamuguri13

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Arial

	A	B	C	D	E	F	G	H	I	J	K
1	Entered?			STREAM BED JET DATA							
2	Checked?										
3								Date	14/03/09		
4											
5	Jet Test Location	Upper layer of test 12						Operators	TAPAS		
6											
7	Zero Point Gage Reading (cm)	11.500						Test #	13		
8											
9	Preliminary Head Setting (in)	39.25				PG Reading @ Nozzle (cm)	14.7				
10											
11	NOZZLE DIAMETER (IN)	0.236				Nozzle Height (cm)	7.900				
12											
13											
14	SCOUR DEPTH READINGS				HEAD SETTING						
15	TIME	DIFF	PT GAGE	MAXIMUM	TIME	HI	LOW	HEAD			
16	(MIN)	TIME	READING	DEPTH OF	(MIN)	PRESS	PRESS	(IN)			
17		(MIN)	(CM)	SCOUR (CM)		(PSI)	(PSI)				
18	0	-	6.80	0.000	0	1.5	0.08	39.34			
19	0.5	0.5	6.70	0.100	0.5	1.5	0.08	39.34			
20	1	0.5	6.60	0.200	1	1.5	0.08	39.34			
21	1.5	0.5	6.55	0.250	1.5	1.5	0.08	39.34			
22	2	0.5	6.50	0.300	2	1.5	0.08	39.34			
23	2.5	0.5	6.40	0.400	2.5	1.5	0.08	39.34			
24	3	0.5	6.30	0.500	3	1.5	0.08	39.34			
25	3.5	0.5	6.30	0.500	3.5	1.5	0.08	39.34			
26	4	0.5	6.20	0.600	4	1.5	0.08	39.34			
27	4.5	0.5	6.15	0.650	4.5	1.5	0.08	39.34			
28	5	0.5	6.100	0.700	5	1.5	0.08	39.34			
29	5.5	0.5	6.100	0.700	5.5	1.5	0.08	39.34			
30	6	0.5	6.050	0.750	6	1.5	0.08	39.34			
31											
32	Temperature (°C)				Bankfull Width (m)						

Data Entry / Data / Asymp Solver / Asymp Data / Asymp Chart / Blasdel Solver / Blasdel / Blasdel Chart Data / Con

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Fig. A.3. Spread sheet program front page for calculating the soil erodibility

Appendix-B

B.1 Lysimeter Apparatus

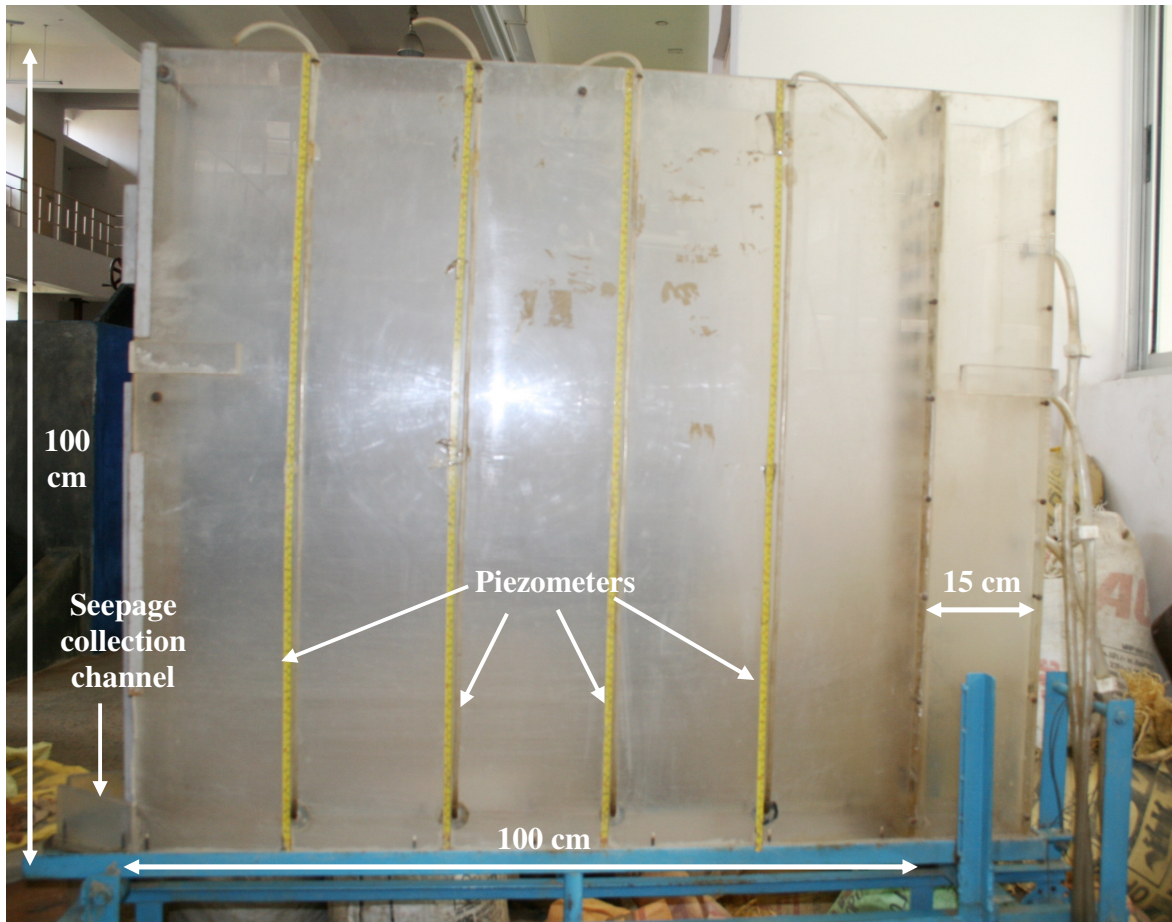


Fig. B. 1. Side view of lysimeter

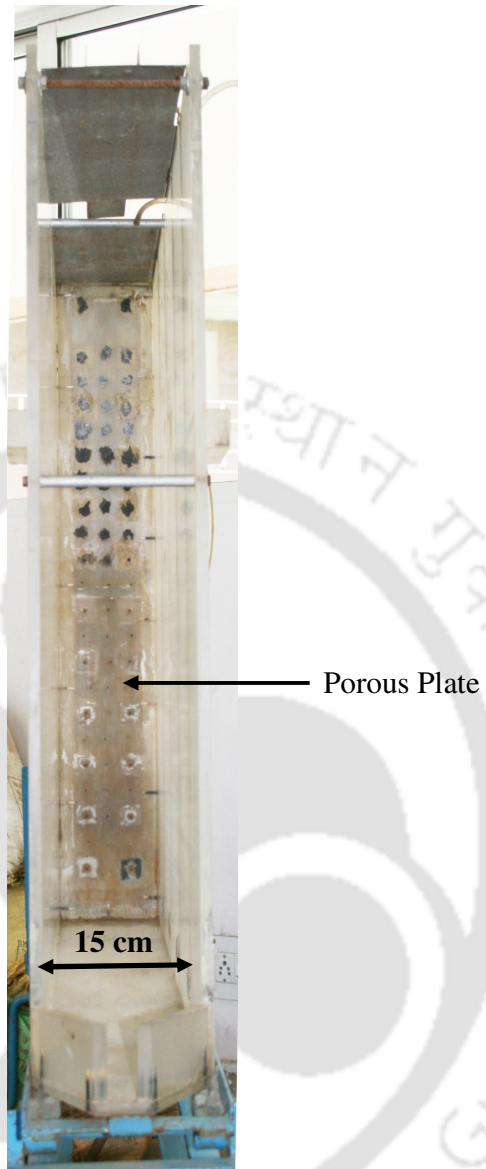


Fig. B.2. Front view of lysimeter

Appendix-C

C.1 Soil Moisture Meter

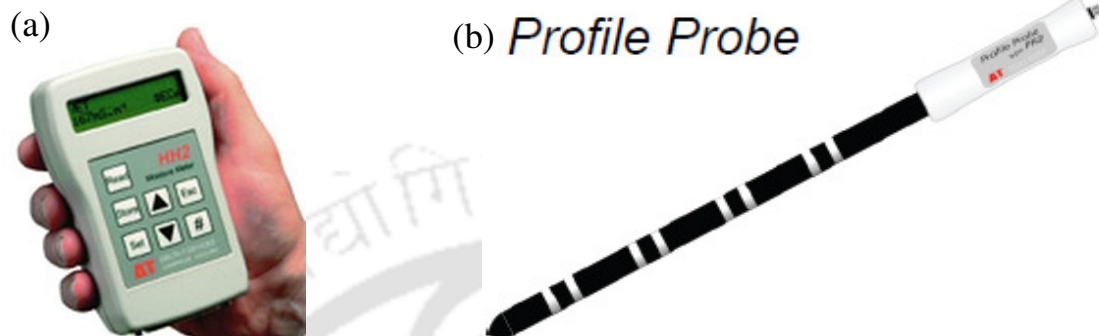


Fig. C.1. (a) Moisture data logger (HH2) and (b) Profile probe (PR2) (Source: delta-T)



Fig. C.2. Sets of augers (Source: delta-T)

Author's Publications

Research Journal Publications

1. **Tapas Karmaker** and Subashisa Dutta (2010). "Generation of synthetic seasonal hydrographs for a large river basin". *J. Hydrology*. **381**(3-4), 287-296.
2. **Tapas Karmaker** and Subashisa Dutta (2010). "Erodibility of fine soils from the composite bank of the Bharmaputra river in India". *Hydrological Processes*. **25**, 104-111. DOI: 10.1002/hyp.7826.
3. **Tapas Karmaker**, Y. Ramaprasad and Subashisa Dutta (2010). "Sediment transport in an active erodible bend of Brahmaputra river". *Shadhana*, Academy Proceedings. **35**(6), 693-706.
4. **Tapas Karmaker** and Subashisa Dutta (2010). "Modeling seepage erosion and bank retreat in the composite riverbank". *J. Hydrology*. (*Under review*)
5. **Tapas Karmaker** and Subashisa Dutta (2010). "Modeling bank erosion for the composite riverbank in a channel bend". (*In preparation*)

International Conferences/Workshops

1. **Tapas Karmaker** and Subashisa Dutta (2010). "Composite bank erosion modeling in an alluvial river bend". *Proc. of River Flow 2010, IAHR International Conference*, Germany. pp 1315-1322.
2. **Tapas Karmaker** and Subashisa Dutta (2009). "Predicting vulnerable bank erosion zones in a large river meander". *Proc. of Water, Environment, Energy and Society (WEES)-2009*, New Delhi. pp 1670-1676.