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APPLICATION OF ARTIFICIAL NEURAL NETWORKS IN REGIONALISATION TECHNIQUE

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Abstract

The estimation of flood corresponding to a selected return period for a catchment area having no records of discharge continues to be one of the principal problems facing the engineering hydrologist. The traditional regional flood frequency method to this problem such as mean annual flood plus growth curve approach suffers from several disadvantages of which the development of reliable equations for predicting mean annual flood (MAF) is among the most important. As an alternative, the relationships between catchment characteristics and parameters of EVI (Extreme Value Type I) distribution in a site can be modelled by means of artificial neural networks (ANN). A series of trials using data from Java and Sumatra islands in Indonesia has demonstrated the viability of this approach. The results produced by the trained networks were found to be superior in terms of root mean square error (RMSE) to those provided by the traditional method.

Introduction

Hydrologists for design purposes often require estimates of flow quantiles of ungauged sites. In this situation, techniques are often applied by which flow information from nearby gauging station is generalized and transferred to the desired site. This process is known as regionalisation.

The transfer of information from gauged sites to the ungauged sites within a region is accomplished by using the relationship between the flow quantiles and catchment characteristics measured from a topographic map and rainfall statistics derived from the national raingauge network.

The accuracy and reliability of these regionalised curves depend mainly on the success of delineating the flood region. It should be understood that adding more catchments to a region implies that more knowledge about the flood characteristics is available, but the quality of the additional information may be poor if the added stations are hydrologically different from the other stations in the region.

The identification of homogeneous regions is traditionally done by the geographical approach, which depends heavily on geographical proximity in defining the sub-regions. However, within the last decade, attention has turned to the use of multivariate techniques, such as cluster analysis, to identify homogeneous sub-regions, and discriminant analysis, to allocate ungauged catchments to an appropriate region. The difficulties encountered in applying these multivariate techniques for hydrological

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regionalisation have been clearly summarized by Nathan and McMahon (1990). For example, any group of variables is capable of yielding clusters, and different structures are often produced by adopting different linkage algorithms and distance measures. Moreover, discriminant analysis always allocates an ungauged catchment to one of the sub-regions, irrespective of the degree of similarity. However, with these techniques, the sub-regions need not be geographically contiguous, which is also a characteristic of the recently introduced region-of-influence approach (Burn, 1990). This paper considers the application of a knowledge encapsulation technique, artificial neural networks (ANNs) as a basis for regionalisation of floods. The proposed approach is explained in more detail in the following section. The third section of the paper includes data analysis and the fourth section presents the results from an application to data from the two islands, Java & Sumatra. Some concluding remarks on the potential of the approach are included in the final section.

Artificial neural networks

Artificial neural networks (ANNs) originated largely in the field of pattern recognition, and are notable for their ability to "learn" the relationship between sets of inputs and outputs without a priori knowledge of the underlying physical processes that connects them. French *et al.* (1992) applied ANNs to forecast the rainfall over a grid of $25{\times}25$ points at time $t+1$ from that at time t, provides an illustration in which the network had 625 input and 625 output nodes. Sandwiched in between the layers of input and output nodes is a third layer of nodes which are connected to all those in the input and output layers. Associated with each connection is a weight that can either amplify or inhibit the signal being transmitted. The nodes then act as summation devices for the incoming (weighted) signals, which are transformed into an output signal using a threshold function that restricts its range to the interval zero-to-one. Standard algorithms, such as the back-propagation technique, are available for adjusting the weights such that outputs are reproduced from inputs with minimum error. The process of adjusting the weights is referred to as 'training the network', and the set of weights then encapsulates the desired input-output relationship. As argued by Hornik *et al.* (1989), the standard multilayer feed-forward network with one hidden layer may approximate any measurable function to any desired degree of accuracy, and errors only appear to arise if there are too few hidden nodes or the relationship being sought is insufficiently deterministic.

In applying ANNs to the ungauged catchment problem, the inputs can be selected catchment and rainfall characteristics, and the outputs can be either specified flood quantiles or the parameters of a standard frequency distribution that is capable of describing observed floods at the gauged sites within the region. The choice of a suitable standard frequency distribution is often controversial, but the General Extreme Value (GEV) distribution has obtained widespread acceptance. This distribution may be expressed as:

$$F(x) = exp\left[-\left(1-\frac{k(x-\beta)}{\alpha}\right)^{\frac{1}{k}}\right]; \qquad k\#0 \qquad (1)$$

$$F(x) = exp\left[-exp\left(-\frac{(x-\beta)}{\alpha}\right)\right]; \qquad k=0 \qquad (2)$$

Where F(x) = (1- 1/T) is the cumulative distribution function of the variable x, T is the return period and α, β, & k are the scale, location & shape parameters of that distribution respectively. Of the three parameters, k, which depends upon the sample skewness of the observed data, exhibits a high variability.

Greenwood *et al.* (1979) obtained the following expressions for scale and location parameters of EVI distribution using PWMs:

$$\alpha = \frac{2b_1 - b_0}{\ln 2} \tag{3}$$

$$\beta = b_0 - 0.5772a \tag{4}$$

Data collection

The present study was implemented by using the mean annual maximum flood data and catchment characteristics from two separate islands Java and Sumatra resulting in a total of 91 sites (48 in Java and 43 in Sumatra). The catchment characteristics included were: AREA (catchment area), MSL (main stream length), S1085 (main stream slope), SIMS (simple slope), AAR (average annual rainfall), APBAR (mean annual maximum catchment 1-day rainfall), FOREST (proportion of forest area to catchment area), PADDY (proportion of paddy area to catchment area), SWAMP (proportion of swamp area to catchment area), PLTN (proportion of plantation area to catchment area, LAKE (proportion of lake area to catchment area) and SHAPE (proportion of catchment area to main stream length). All these data were available from Flood Design Manual for Java and Sumatra (FDMJS, 1983).

The total 91 data were divided into a training set of 65 catchments and a cross validation set of 26 catchments The 26 catchments for the cross validation were chosen such that the values of their variables are within the range of variable values of the catchments in the training data set.

Results and disscussion from application of ann

The neural network software used in this study was the NeuroSolutions simulation environment developed by NeuroDimensions Inc of Florida. Of the many ANN architectures available within NeuroSolutions, the 'standard' multilayer perceptron (MLP), trained using the back-propagation algorithm, was chosen. The object-oriented architecture of the NeuroSolutions environment was found to provide a high degree of flexibility in setting-up, training and testing various ANN configurations. The inputs to the ANN were 11 catchment characteristics (AREA, MSL, S1085, SIMS, AAR, APBAR, FOREST, PADDY, SWAMP, PLTN & LAKE)and the outputs were the scale & location parameters of the EVI distribution fitted to the annual floods recorded at each gauged site.

Having adopted an MLP with 11 inputs and two outputs, a further series of trials was carried out in order to determine the best configuration of nodes in the hidden layer. The results showed that six nodes in the hidden layer gave the best generalization properties on the verification data set while maintaining a high degree of accuracy on the training data set. An ANN with this configuration was therefore trained several times for 20,000 repetitions on the input data, and the set of weights associated with the smallest residual error between ANN and the desired outputs was selected to produce estimates of the EVI parameters for the 26 catchments in the verification data set. These parameters could then be inserted into the equation for the EVI distribution to provide estimates of the flood magnitude x(T) corresponding to an arbitrary return period T. Equation (2) for k=0 may be rearranged in its inverse form to give:

$$x(T) = \beta - \alpha \ln\left[-\ln\left(1-\frac{1}{T}\right)\right] \tag{5}$$

The values of the parameters of the regional GEV distribution obtained from Equation (1) for the combined Java and Sumatra data set were: shape parameter, k_R=-0.0342; location parameter, b_R=0.8381; and scale parameter, a_R=0.2646. Applying the Z-test given by Hosking et al. (1985), using the null hypothesis H_0: k_R=0, the PWM growth curve could reasonably be assumed to come from an EVI distribution with location parameter, b_R=0.8422 and scale parameter, a_R= 0.2733. Therefore, the regional frequency curve (also called the growth curve) can be expressed as follows:

$$q_T = 0.8422 + 0.2733\left[-ln\left(-ln\left(1-\frac{1}{T}\right)\right)\right] \tag{6}$$

In order to compare the performance of this 'Regional ANN' for the EVI parameters with the regression mean annual flood plus growth curve approach, two flood magnitudes were selected: the mean annual flood, MAF, which for an EVI distribution has a return period of 2.33 years, and the 50-year flood. In the case of the former, the regression equation predicting MAF (m³/s) obtained using stepwise regression procedure which is a 4 variable equation given Equation (7) were employed.

$$MAF = 10^{-3.887} AREA^{0.78} AAR^{1.241} (1+PLTN)^{-1.769} (1+LAKE)^{-2.282} \tag{7}$$

A visual impression of the results may be obtained from Figure 1, which shows plots of the regression MAF from Equation (7) and the ANN MAF computed from the EVI parameters obtained by ANN model by applying Equation (5) against the 'observed' *MAF* from the Flood Design Manual for Java and Sumatra (FDMJS). The ANN results appear to be more closely grouped around the line of equal values than those from the regression model. In addition to the scatter plot, the RMSEs between computed and observed values were calculated. The values obtained were 275.1 m³/s for the regression model and 190.0 m³/s for the ANN, an improvement of 31% of the latter over the former. The above exercise was repeated for the 50-year flood. In the case of the ANN model, Equation (5) gives the required quantile directly. For the regression model, the *MAF* from Equation (7) must be multiplied by the appropriate growth factor, which is 1.91 from Equation (6) for the combined data set of Java and Sumatra. The 'observed' 50-year flood may be estimated from Equation (5), but using the α and β obtained from the recorded annual floods, *i.e.* the values on which the network was verified. The scatter plot shown in Figure 2 is similar to Figure 1 where ANN results also appear to be more closely grouped around the line of equal values than those from the regression model. The RMSEs were 669.2 m³/s for the regression model and 408.0 m³/s for the ANN model, a 39% improvement of the latter over the former.

Concluding remarks

This pilot study of the use of an ANN as a means of representing the relationship between flow quantiles and catchment & rainfall characteristics has indicated its potential to improve on the performance of existing ungauged catchment flood estimation methodologies, such as multiple linear regression analysis (MLRA) and regional growth curve.

In effect, the catchments are defined by a set of features, the selection of which is not necessarily limited by the correlation between the variables, such as the relationship between MSL and AREA, often referred to as Hack's Law (Rigon et al., 1996) The essential purpose of the selected features is to encapsulate as completely as possible the full range of variation in the catchments making up the region. The number of features employed in this study has been limited by the extent of readily-available published information.

The number of stations should be increased because adding more stations to a region implies that more knowledge about the flood characteristics is available. There were a total of about 1000 stations identified (FDMJS, 1983) but in the present analysis only the reliable 92 stations were considered.

A more comprehensive study involving a wider selection of catchment characteristics and the possible use of Genetic Programming to identify the pertinent features may be the most productive direction for further work.

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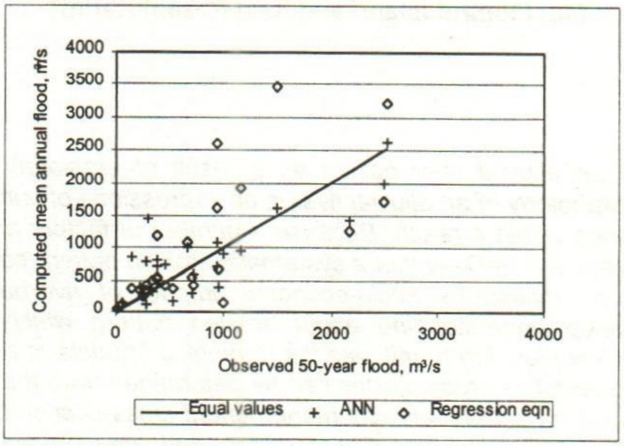

Figure 1 Plot of computed versus observed mean annual floods for the combined stations in Java and Sumatra.

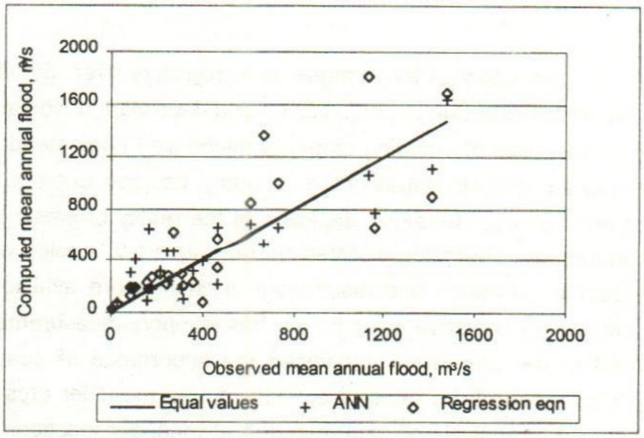

Figure 2 Plot of computed versus observed 50-year floods for the combined stations in Java and Sumatra

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RATES OF BANK EROSION OF THE ARIAL KHAN RIVER

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Abstract

Bank migration in an alluvial river occurs as a result of erosional and depositional processes. The morphology of an alluvial river is an expressions of fluctuating discharge and sediment balance within a reach. Because a number of factors are involved in the morphological process it is unlikely that a systematic erosion pattern could be found. But keeping in view the disastrous socio-economic impact of riverbank erosion it is imperative to develop understanding about erosion pattern which will be of very important use in policy planning to mitigate the damaging impacts of river bank erosion. In this regard representative erosion rates can be determined from the survey of certain dimensions. One method is to resurvey a monumented cross-section or series of points and the other method is to make use of landsat data and superimposed imageries. This paper reports the rates of bank erosion of the river Arial Khan based on the thirty two monumented cross-section survey data and maps and spot imageries of different years. An attempt is made here to relate the erosion rates with different discharge magnitudes.

Introduction

The river Arial Khan, a distributory of the Ganges, is a migratory river. Significant river channel pattern changes occur in this river due to bank erosion and deposition of sediment. The shift in the river courses often causes loss of standing crops, farmland and homestead land. In fact it is a problem that reaches to the delicate issues of the economy, ecology, demography, transportation and even of politics and culture. Therefore, depiction of the reality of erosion and accretion has much to do with management of erosion disaster. So far only a few studies which have reported the actual rate of channel migration and morphological change are available. Wolman (1959) reported rates of erosion in cohesive river banks based upon measurements collected from erosion pegs secured in the bank and recognized the importance of seasonality in rates of erosion. Hughes (1976) reported the rates of erosion around meander arcs in relation to peak discharges, based upon the analysis of data recorded at meander locations in the river Cound catchment, Shropshire. Hickin and Nanson (1984) studied the influence of bend curvature on bank erosion rates for rivers in Canada and found that erosion rate was a function of the radius of curvature to width ratio (r/w). As bank erosion occurs under a variety of discharges it is important to measure actual rates of erosion in relation to the magnitude and frequency of discharge events. Most studies of channel adjustment have stressed the importance of the annual flood, but Harvey(1975) remarked on the effectiveness of intermediate discharges. Wolman and Brush (1961) recognized the importance of the unstabilizing effects of fluctuating discharges

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experienced in nature.In this paper the bank erosion rates are analyzed within the framework similar to the work of Hickin and Nanson (1985) for meandering rivers. The bank erosion is considered to be the function of width and relative curvature assuming that Chezy co-efficient and overall bank resistance co-efficient do not vary along the river.An attempt is also made here to relate the erosion rates with different discharge magnitudes following the approach used by Hughes(1976). The sources of information used in the analysis are cross-sections, maps, landsat data and spot imageries and hydrological data. The monumented cross-section survey was infrequent and over the time period of 1968 to 1996 survey conducted in each cross-section ranges from 6 to 9 times. In case of a few cross-sections it was not possible to take measurement of the bank margins from a fixed point in space for the considered timespan due to loss of the monument/peg through encroachment of the river and also due to lack of information about the exact distance between the lost and newly founded monument. However, those cross-sections were still important to have idea about erosional changes over a shorter time period.From these survey data the rate of bank erosion integrated over the time period between surveys may be obtained although no information about the complex changes that took place between surveys was available and also these can seldom be related to the events which cumulatively might have produced them. However, whatever the problems may be such sources of information can be used because there is no alternative. In most cases it was seen that erosion trend can shift from one direction to another even in shorter time frame. In some cases gradual migration of river bank was observed indicating formation of meander loops. The shifting pattern of the river was also available from the superimposed images (maps, spot and landsat imageries). All the above mentioned data and information were analyzed and results are presented in tables and plotted in graphs to report rates of bank erosion, their frequency distribution and possible relationships with other parameters. It is possible that the results obtained from this study differ from the results of other research studies. This is because the purposes, methods and measures used in different studies are diverse and diffuse to a confusing extent. But all such variety of interest and approach demonstrate that there are compelling practical reasons for studying the rates of bank erosion.These reasons include, firstly, an awareness that human activity in the vicinity of river channels has unfortunately proceeded in ignorance of the shifting pattern that may be expected and that this ought to be corrected. Secondly, understanding of bank shifting rates can help substantially in river bank erosion disaster management.

Literature review

Bank migration through erosion-deposition processes in an alluvial river is a characteristic feature and one of the most conspicuous changes affecting fluvial landscapes.At a meander bend high velocity occurs in the outer bank causing recession of bank and also the spiral flow tends to deepen the outer bank. In a river rates of such bank erosion can be rather high but such rates apply to certain bends only ; others on the same river at the same time shift more slowly. Generally, the rate of bank migration is determined by the strength of the bank on one hand and the fluid forces on the other hand. Under natural conditions regular pattern of bank migration can not survive. This is due to the fact that apart from the effects of river flow fluctuations, river and valley floor sediments are rarely uniform and the lateral redistribution associated with bank cutting and point bar construction introduces size sorting. Continued migration with spatially variable boundary conditions must inevitably lead to distortion of the waveform with some bends, or parts of bends, eroding faster than others as the pattern as a whole becoming irregular. A deterministic analysis of meander development is extremely complicated because an irregular meander pattern is even less likely to be in a steady state. However, a statistical equilibrium can be envisaged in which the pattern retains its aggregate characteristics despite changes in detail. If some bends grow, but others decline or are eliminated, the scale and degree of meandering, and the overall level of irregularity, may remain more or less constant over the years. The river occupies different positions at different times and experiences different spatial sequences of disturbances about the average condition. The precise course of the channel depends on the detailed pattern of these disturbances, but the overall nature of the waveform need not alter. But such a statistical equilibrium is possible only if no change occurs in hydrologic regime of sediment load and the disturbance sequences are realizations of a single stationary stochastic process.If changes take place the problems in approaching to establish whether lateral movement of channels can be discerned from the various forms of evidence available are questions on their spatial and temporal distributions: questions about the controls on movement,including the effect of specific disturbances such as alteration can be analyzed based on the knowledge of the relationship between form and movement and on identification of stable forms. The problem in analyzing any aspect of river bank erosion is to find a method which adequately demonstrates the properties of the course and planform, especially where meanders are ill-defined or irregular. The aim here is to characterize the channel pattern as objectively as possible whilst also being able to identify and measure change and movement in a meaningful way (Islam, Kanungoe and Zaman,1999). In the recent past a number of important studies have been done by various researchers to develop insight into the bank erosion process as well as rates of bank erosion. Wolman (1959), Osman and Thorne (1988), Hickin and Nanson (1984), Klaassen and Masselink (1992) and Hughes(1976) played a notable role in this regard. Hickin and Nanson (1984) presented channel bend migration data for a range of meandering rivers in western Canada and assessment of the factors that control these rates.Channel migration rates transformed to a reference bend of curvature ($r/w = 2.5$) are shown to be a simple function of stream power, outer bank height and a co-efficient of resistance to lateral migration.An earlier study of the authors on the Beatton river showed that channel migration rates are strongly controlled by bend curvature.Klaassen and Masselink studied the bank erosion of the braided Jamuna River of Bangladesh and found that in most of the cases erosion rate along the curved channels is between 0 and 500 m/year. The bank erosion rates analyzed within the framework similar to the work of Hickin and Nanson revealed that low relative curvatures lead to relatively fast erosion rates and vice versa. Hughes(1976) investigated the rates of bank erosion around the meander arcs in relation to peak discharges, based upon the data recorded at meander locations in the river Cound catchment. From his study it was revealed that the pattern of period rates of erosion is similar for each of the arcs investigated. The values of mean loss per site for each arc were also similar. It is also revealed from his study that discharge magnitudes represents a threshold for major channel changes along the reach of the river. From the study a range of discharges representing the erosion threshold for the reach as a whole was recognized and three erosional classes based on erosion rates were indicated.The frequency distribution of the discharges representing the lower erosion threshold and the higher erosion threshold was determined.

Methodology

Data used

The rates of bank erosion of the river Arial Khan have been studied by use of cross-sectional data, survey maps, landsat and spot imageries and hydrological data. The monumented cross-section survey data were obtained from Morphology Division, BWDB, Dhaka and the hydrological data were obtained from Surface Water Hydrology, BWDB, Dhaka. Maps were collected from SOB
(Survey of Bangladesh) and landsat and spot images were collected from SPARSO.

Cross-sectional data

Thirty two cross-sections of the river Arial Khan have been studied. The locations of the studied cross-sections are shown in Fig. 1. The cross-sections cover the entire river stretch. The monumented cross-section survey was infrequent and over the time period of 1968 to 1996 survey conducted in each cross-section ranges from 6 (six) to 9 (nine) times. From these data it was possible to take measurements of bank margins from a fixed point (monument/peg) in space.The cross-section surveys had been conducted in winter and the bank margins in this study correspond to the low flow water levels.An examination of the bank migration data obtained from the survey of monumented cross-sections reveals that there is no systematic pattern of bank shifting.In many occasions the shifts are rather dramatic than gradual and also a bank which shifted in one direction during a particular time period was seen to shift in opposite direction in the next time period. The cross-section change that took place at AKU# 7 between 1970 and 1996 appears in Fig.2.It is also evident that other than the variations in the amount of discharge many other factors are involved in the process as well.The role of some local factors are of significance in this regard.

Maps and satellite images

Maps of the year 1941,1944 and 1961 and landsat and spot images of 1973,1976, 1980, 1985,1988,1992,1995 and 1998 were used for the study. From these maps and spot images 10 (ten) bends were selected for measurements of width, radius of bend curvature and the rate of bank erosion. The maps and images were prepared at scale of 1:50,000 and this scale had been identified as suitable for analysis.Channel displacement was measured by superimposing maps and images of different years and matched by using common water bodies located as near as possible to the plane of the river flood plane and more or less fixed in space and time. Fig.3 shows typical meander development and cut-offs between 1944 and 1998. It can be seen from the superimposed maps and images that marked changes in the channel pattern have been taken place over the timespan of 54 years.Channel width for each bend was taken from maps and images as the mean of several bank crest to crest measurements recorded near points of inflection and on other adjacent straight reaches at a spacing of approximately one channel width along the channel center line.Since bends are not exactly circular arcs, difficulties arise in defining and measuring radius of curvature. In this regard the procedure described by Hickin and Nanson (1983) was followed. The authors defined the radius of curvature in the form $r_m = (r' + r'')/2$ for their meander bend migration rate evaluation.In this method, the radius of curvature for the bend is the mean radius of two circles; one passing through X_2Y_2,X_3Y_3 and X_4Y_4 and the other passes through X_1Y_1, X_3Y_3 and X_5Y_5. Here X_3Y_3 represents the point of maximum curvature. The digitizing interval Δx for the five sets of coordinates is equal to the mean channel width measured as mentioned above. Channel migration rate (m/year) for a bend was measured as the maximum outer-bank displacement normal to the former channel.

Hydrological data

There are several stage monitoring station along the river Arial Khan.But discharge is monitored only at one station located near the off-take of the Arial Khan (upper). Therefore no discharge data are available for the river that takes off downstream of the river Padma (Fig.1) and also for the combined flow.However, the available discharge data can be used for the Arial Khan(upper). From the daily discharges monitored at the station situated at Chowdhury Char during 1965 to 1993 it is possible to calculate annual maximum discharges, annual average discharges and annual minimum discharges. It was seen from the discharge data that annual maximum discharge ranges from 1130 cumec to 5810 cumec, annual average discharge ranges from 350 cumec to 1390 cumec and annual minimum discharge ranges from 0.83 cumec to 176 cumec. The ratio between maximum to minimum discharge for each year ranges from 14.9 to 2156.6.In the present study an investigation is made to relate the rates of erosion with peak or maximum discharges although erosion occurs under a variety of discharges.The annual peak discharges for the period 1965 to 1993 have been shown in Figure 4.

Erosion rates

From the examination of the superimposed maps and images and monumented cross-sectional suvey data 3 (three) bends and 5 (five) monument sites were identified to show the extent of erosion that had occured at each site over a timespan of 25 years (Table 1).

Table 1 : Erosion at different sites over a timespan of 25 years

Location	Time period	Erosion (m)
Bend E*	1973 to 1998	1415
Bend F	1973 to 1998	1060
Bend G	1973 to 1998	1180
AKU # 7	1971 to 1996	836
AK # 7	1971 to 1996	692
AK # 11	1971 to 1996	499
AK # 18	1971 to 1996	759
AK # 19	1971 to 1996	717

* Selected bends were identified in this way for analysis

The above table reveals that the maximum erosion for the period of 25 years is 1415 m. The mean erosion per site is 894.8 m. The yearly rates for the observed maximum erosion and mean erosion are 56.6m and 35.8 m respectively. An analysis of all erosion data shows that the bank erosion rate along the bend in most of the cases is between 0 and 133 m/year with larger values upto 178 m/year under exceptional conditions. The arithmetic mean of all erosion rates observed along the bends is 50.9 m/year with standard deviation 37.5. The co-efficient of variation is 73.6 which indicates that there is large variation in erosion rates.The amounts of erosion that were calculated between consecutive surveys at different monumented cross-sections for the Arial Khan (upper) reveal that amounts of erosion are greatly influenced by the magnitude of the peak discharges.Although it is not possible from the recorded data to estimate the % of total erosion contributed by the peak discharges it can be fairly assumed that the greatest amounts of erosion were caused by these discharges. The following table (Table 2) shows the amounts of erosion in relation to peak discharges.

Table 2 : Amounts of erosion in relation to peak discharges

Location	Time period	Peak discharge (m^3/s)	Amounts of erosion (m)
AKU # 2	1977 to 1978	2350	52
	1978 to 1979	1790	25
	1979 to 1980	2260	10
	1992 to 1994	3900	157
AKU # 3	1971 to 1973	3970	238
	1992 to 1994	3900	76
AKU # 5	1992 to 1994	3900	213
AKU # 7	1971 to 1973	3970	105
	1977 to 1978	2350	6
AKU # 11	1969 to 1970	1910	7
	1970 to 1973	3970	77
	1978 to 1979	2310	10
	1979 to 1980	2260	15
	1980 to 1981	3040	97

The information presented in the above table shows the effects of the peak discharges on amounts of erosion. In order to depict the influence of peak discharge magnitudes on amounts of erosion the erosional changes over a shorter time period had been chosen. It can be seen from the table that each location suffers greater amounts of erosion in response to peak discharges of higher magnitude than to that of lesser magnitude.From the recorded discharge data it can be seen that the highest peak discharge occured in the year 1991 (5810 cumec) and also two high peak discharges having magnitude of 4890 cumec and 4640 cumec occured in the year 1988 and 19990 respectively. Due to lack of survey data it is not possible to show the influence of those discharges on amounts of bank erosion during that period.However, erosion data obtained from the infrequent surveys still can be used to show that the discharge magnitudes have the major influence on bank erosion. Attention is drawn in this regard to the following table (Table 3).But, before that it is necessary to have a look over Fig.4.It can be seen from the Fig. 4 that over the period 1969 to 1978 no annual peak dischage exceeded the magnitude of 4000 cumec and only three discharges were greater than 2500 cumec. On the other hand during the period from 1978 to 1992 three annual peak discharges exceeded the magnitude of 4000 cumec and eight discharges crossed the discharge of 2500 cumec.

Table 3 : Total amounts of erosion at the same site in two different time period

Location	Total amounts of erosion (m)	
	1969 to 1978	1978 to 1994
AKU # 5	50	408
AKU # 6	No erosion	924
AKU # 7	219	689

The information obtained from the Fig. 4 and Table 3 confirms the contention that the discharge magnitudes have much to do with the amounts of erosion.However, in a single period the amounts of bank retreat are different at different locations.It implies that many other factors are involved in the erosion process together with the discharge.

Data analysis

The rate of bank erosion in meandering rivers depends upon bank resistance, flow characteristics and sediment transport. In order for a channel to migrate, erosion on one bank (generally the outer bank in a bend) must be balanced with the deposition on the opposite bank. If point bar deposition does not keep up with opposite site bank erosion, the gradual widening of the channel will decrease shear stresses until migration ceases. The erosion rate of a meandering river depends functionally on the degree of flow asymmetry across the cross-section and in particular on the contrast in shear stresses between opposite banks. In the case where the rate of sediment deposition is the limiting factor, cross-channel bed slope and secondary circulation set up by the flow asymmetry are also important. Although in natural channels the flow patterns and corresponding stresses and bed topography are spatially and temporally complex, the regularity of meander patterns and cross-sectional asymmetry in meandering channels suggests that migration rates averaged over a period of years can be related to simple descriptions of river flow properties. In this study the rate law governing bank erosion is based upon the measurements of Hickin and Nanson and theoretical approaches of them who related bank erosion rates to local channel curvature. Flow asymmetry should increase with channel curvature for given channel dimensions and discharge leading to greater bank erosion and point bar deposition. Nanson and Hickin (1983) summarized detailed measurements of channel migration rates in the Beatton River, Canada as a fuction of bend radius (r) normalized by the channel width (w). They found that migration increases to a maximum as r/w decreases to a value near 3 but drops rapidly for smaller values. The observed bank erosion rates from 10 (ten) selected bends had been analyzed within a frame-work similar to work by Nanson and Hickin (1984). According to Nanson and Hickin the yearly bank erosion rate M (m) can be expressed as:

$$M = f(C, Y_b, w, r/w)$$

where, M = yearly erosion rate (m), C = Chezy co-efficient ($m^{0.5}/s$), w = channel width (m) , Y_b = the opposing force per unit boundary area resisting migration and r = bend radius (m). If it is assumed that the parameters C and Y_b do not vary along the river the equation reduces to:

$$M = f(w, r/w)$$

The measurements of channel width, radius of bend curvature and the rate of bank erosion were used to test the above relationship for the river Arial Khan. The measured parameters were divided in classes and number of observations in each class were determined. Figure 5 provides information of low flow widths versus the number of observations. The observed erosion rates versus the number of observations appears in Figure 6. In Figure 7 observed relative radius of curvature versus the number of observations has been shown. A plot of erosion rate (M) versus relative bend curvature appears in Figure 8. The plot clearly shows that the bank erosion rates are strongly controlled by bend curvature. A 2 (two) degree polynomial trendline well represents the relationship between erosion rate and relative bend curvature. The trendline indicates that maximum erosion rates occur at $3< r/w < 4$. It is similar to the findings of Hickin and Nanson who found it at $2 < r/w < 3$ for the rivers in Western Canada. But unlike their finding that in the domain $r/w < 2$ the erosion rate declines very rapidly to zero at $r/w \cong 1$ here it can be seen that in the domain $r/w < 3$ the decline is rather gradual than rapid. In the domain $r/w > 4$ the erosion rates tend to decline gradually from its maximum value. The equation of the two degree polynomial trendline is given below:

$$M = -9.3043 (r/w)^2 + 70.825 (r/w) - 41.57$$

The relationship between dimensionless relative migration rate and bend curvature has been plotted in Figure 9. Here also the relationship is shown by a 2 (two) degree polynomial trendline. This trendline confirms that the maximum erosion rates occur at $3 < r/w < 4$ and gradually declines from the maximum value in both the domain $r/w < 3$ and $r/w > 4$. The equation of this trendline is given by:

$$M/w = - 0.027 (r/w)^2 + 0.2012 (r/w) - 0.0942$$

The figure shows some very low values of M/w which indicate that a number of bends migrated a little or did not migrate at all. It might have happened due to the fact that erosion rates were obtained from maps and images which were taken after a long interval and consequently yielded very low values of yearly erosion rates.

Discussions and conclusions

The study of bank erosion rates of the river Arial Khan based on maps, satellite images, cross-sections and hydrological data provides some important information to develop understanding about erosion pattern, rate law governing bank erosion and erosion rates in relation to peak discharges. However, due to lack of adequate information and more frequently recorded data progress could not be made upto expectation to this end. As for instance, there is only one station for recording discharges and the recorded discharges can be applied only for the Arial Khan (upper). The monumented cross-section survey had been conducted infrequently due to which enough data regarding amounts of erosion within a short time period were not available. Availability of such data could help in identifying different erosion thresholds and corresponding discharges and also classes of erosional response if exists could have been identified. Still, the study appears very fruitful in terms of assessing erosion rates and developing rate law governing bank erosion.In this regard it should be noted here that the relationship between bank erosion rate and river size is difficult to isolate because of the confounding effect of intermittent channel migration and the complex relationship between migration and curvature. Migration rate based on simple mean of the raw data is biased by the proportion of bends in particular curvature classes. An examination of the estimated curvatures for the present study shows that such problem does not arise in this study.The following conclusions can be drawn from the study:

(1) The river is very much dynamic in nature and experiences very wide range of fluctuation in discharges. The maximum to minimum discharge ratio can be as high as 2156.6.

(2) The maximum extent of bank erosion over a time period of 25 years is 1415 m and yearly rate for this erosion to take place is 56.6 m.

(3) In most cases the yearly erosion rate varies between 0 and 133 m and under exceptional conditions it can be as high as 178 m. The average erosion rate is 50.9 m with a standard deviation of 37.5.There is large variation in yearly erosion rates.

(4) Generally greater amounts of erosion occur in response to annual peak discharges of higher magnitudes and amounts of erosion depend on discharge magnitudes as well as other factors.

(5) The bank migration rates are strongly controlled by the bend curvature and an analysis within a frame-work similar to Nanson and Hickin reveals that maximum erosion rates occur at $3 < r/w < 4$. In the domain $r/w < 3$ and $r/w > 4$ the erosion rates decline from its maximum value in a gradual manner.

(6) The findings of this study can be fairly used for prediction of erosional changes of the river Arial Khan.

(7) A further study including detailed information and continuous series of data would help develop a clear understanding about the erosion process and therefore need for such a study is emphasized.

Acknowledgement

This study has been performed within the framework of the research project "Strength Characteristics of Soil and Bank Shifting Characteristics of the Arial Khan River". The research works of this project is now in progress. The data used in this study were collected from different organisations in connection with this project. The necessary fund for the project was granted by the Ministry of Science and Technology (MoST),GOB. The authors are, therefore, very much grateful to the MoST for providing financial assistance without which this study was not possible.

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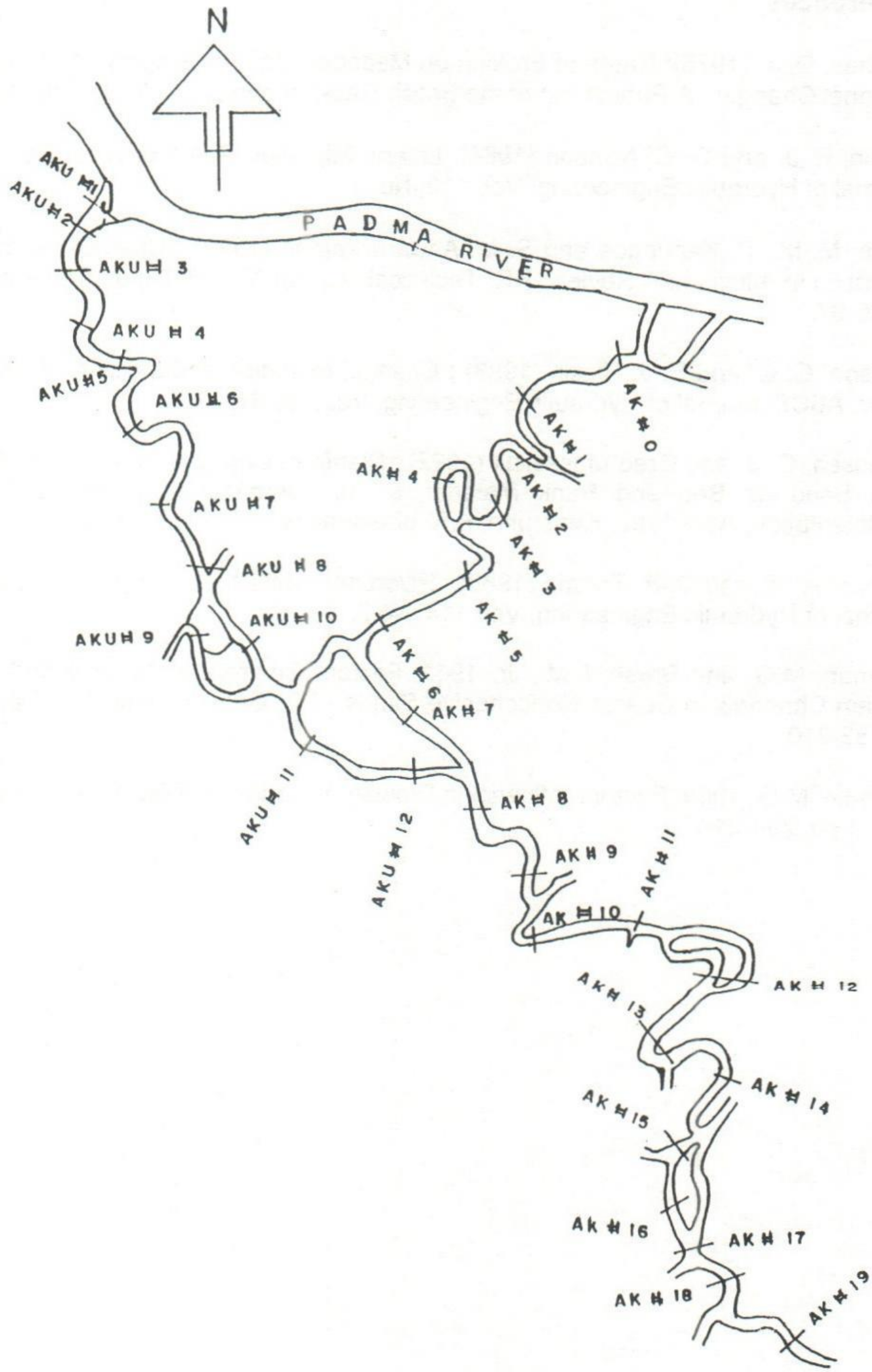

Figure 1 : Location of BWDB cross-sections

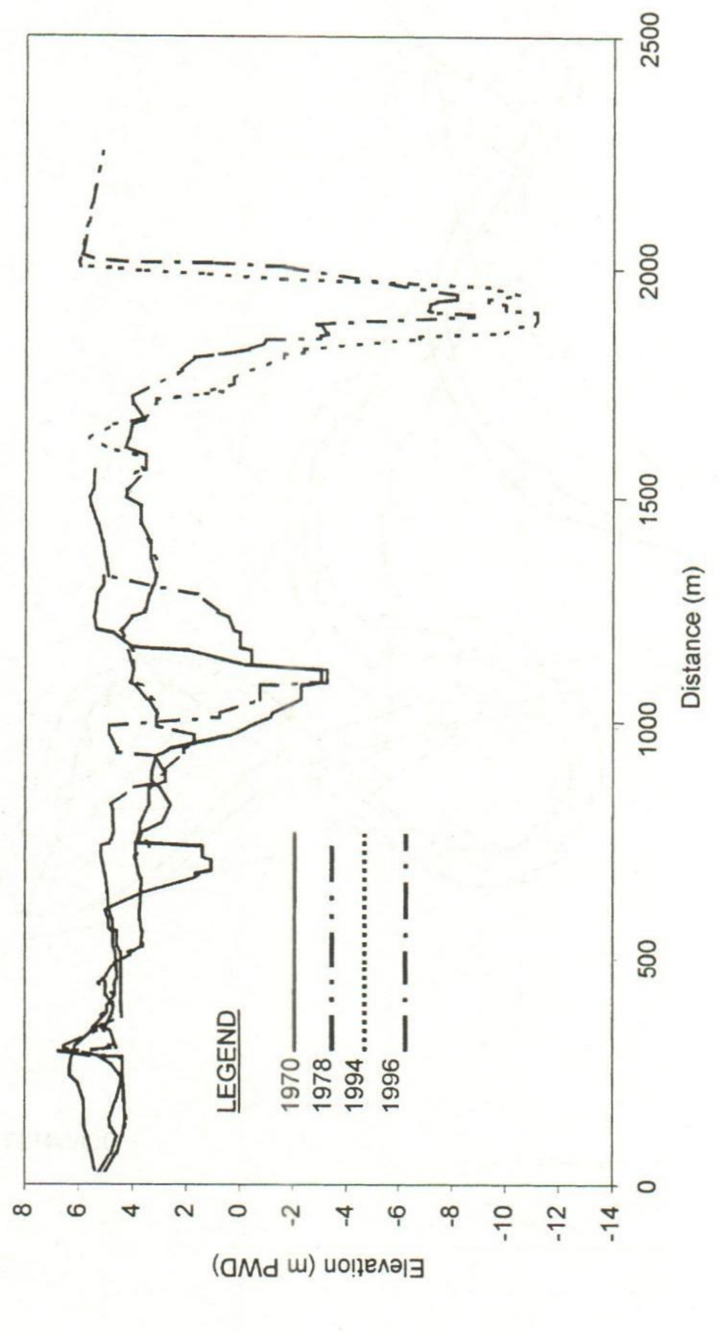

Figure 2 : Cross section Change 1970-1996 at AKU # 7

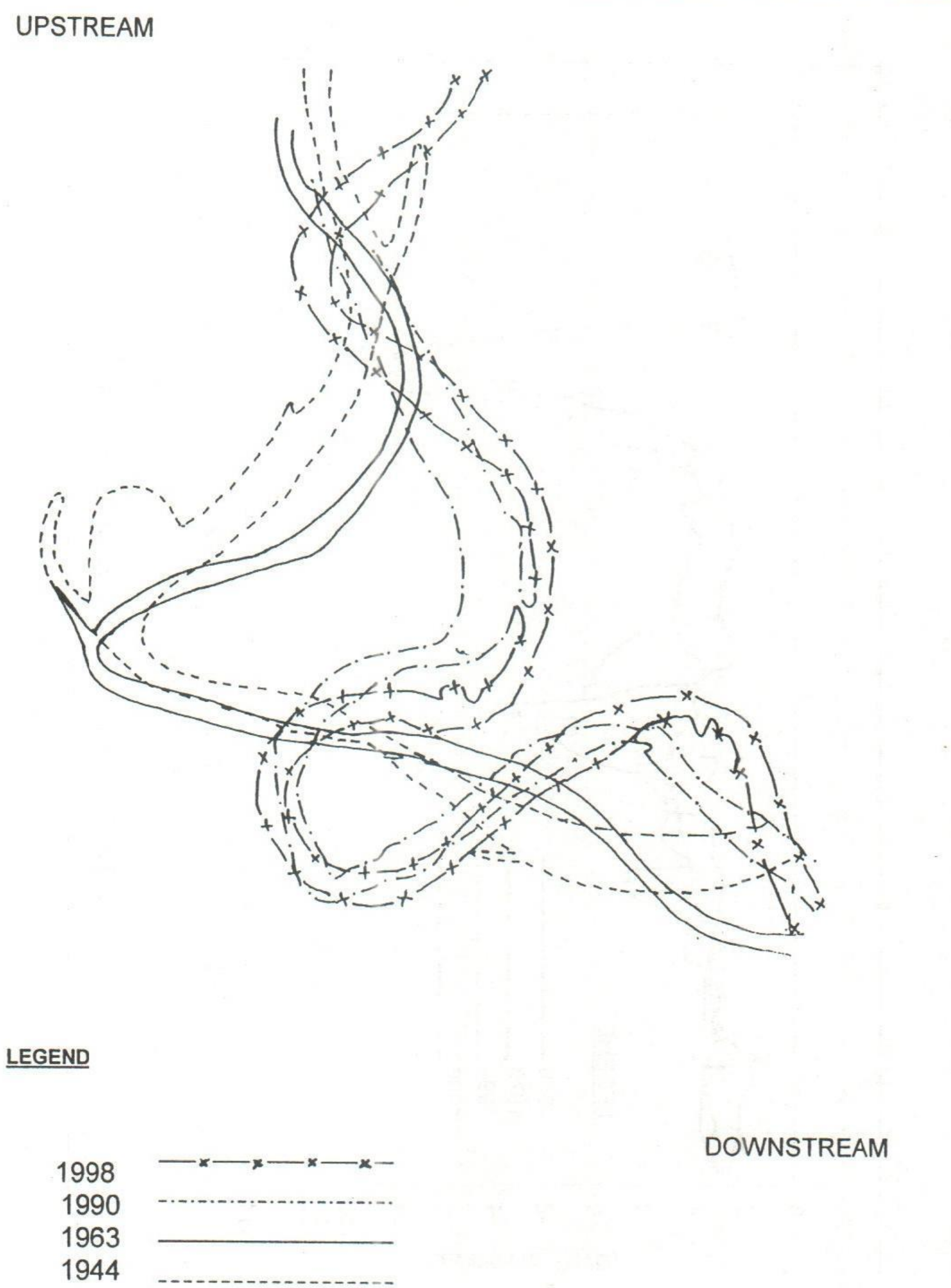

Figure 3 : Meander development and cut-offs between 1944 and 1998

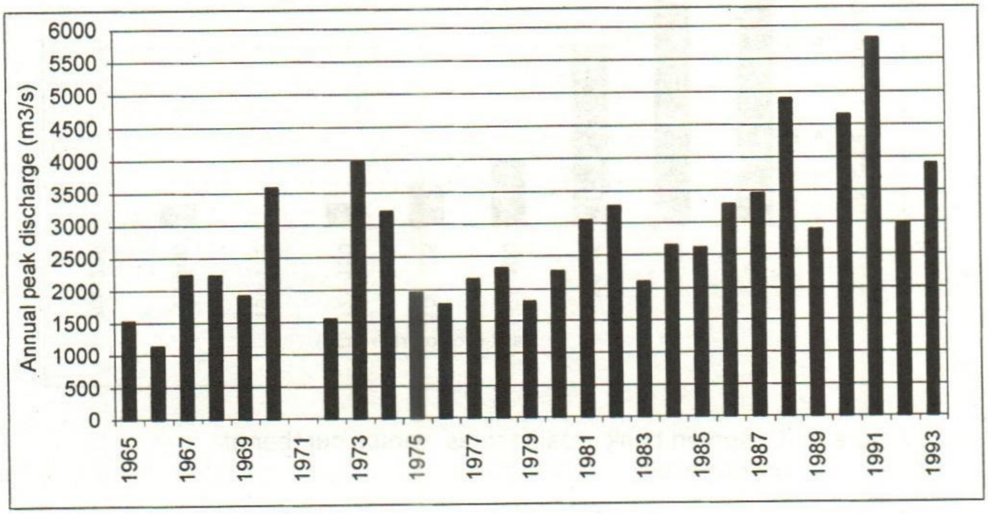

Figure 4 : Annual peak discharges for the period 1965 to 1993

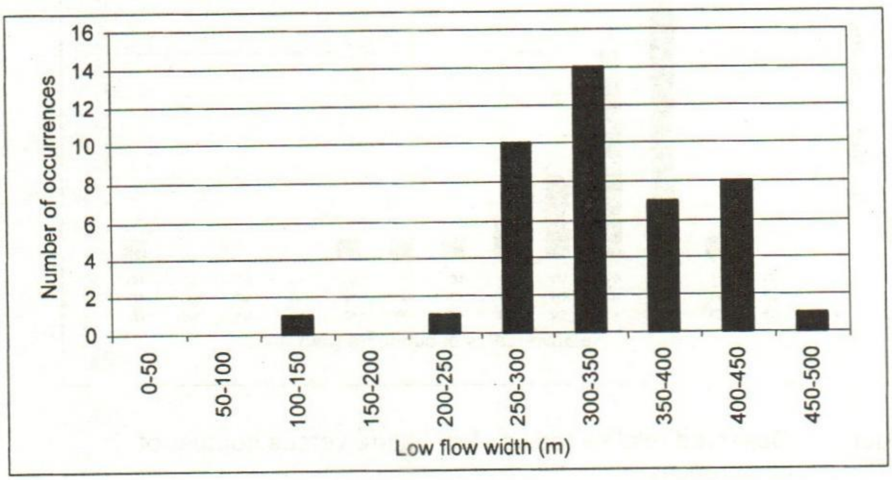

Figure 5 : Observed low flow widths versus number of occurrences

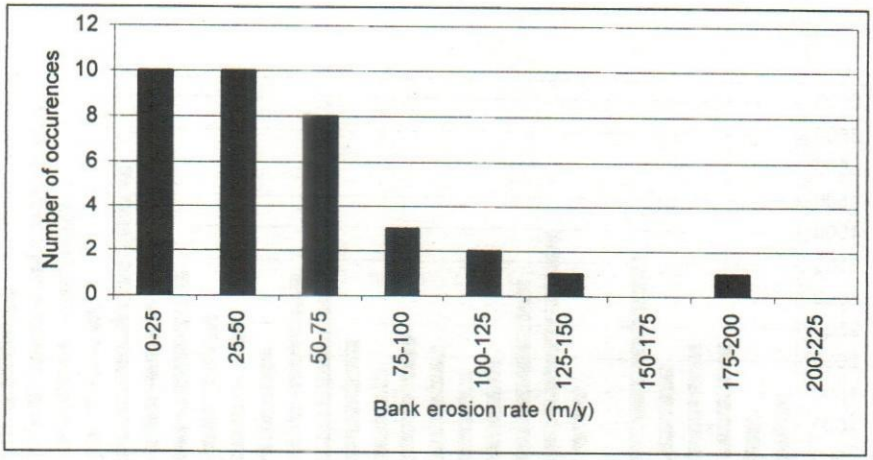

Figure 6 : Observed bank erosion rates versus number of occurrences

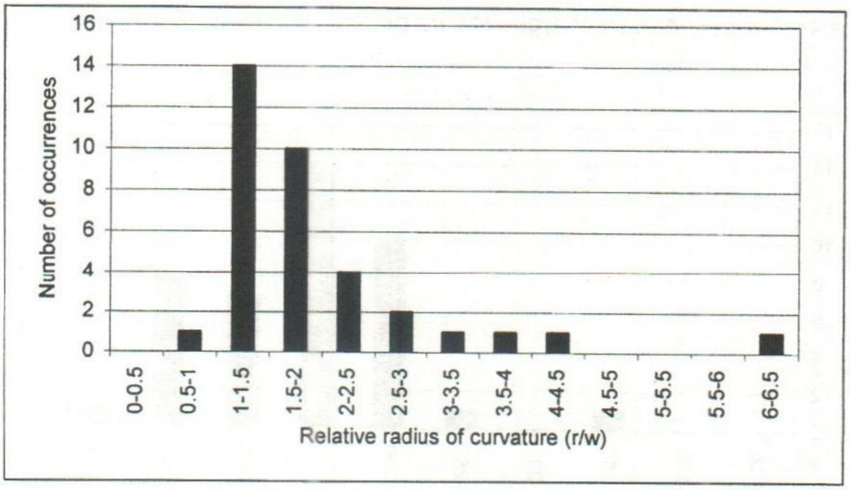

Figure 7 : Observed relative radius of curvature versus number of occurrences

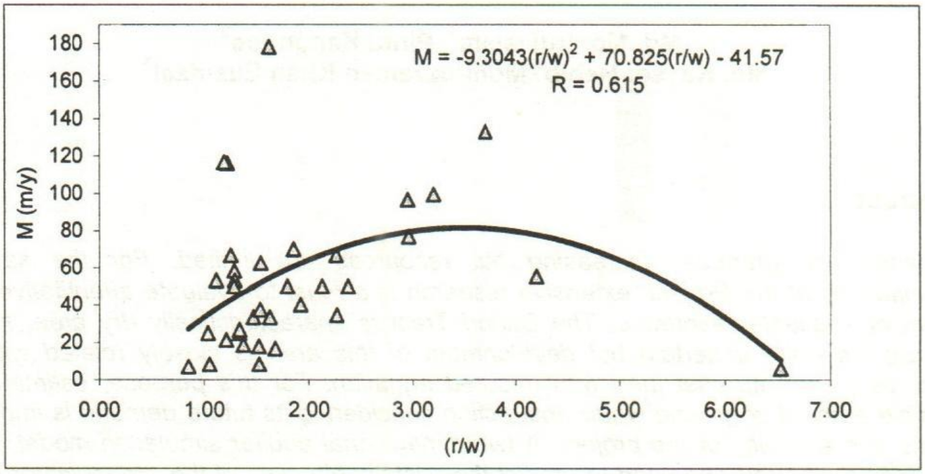

Figure 8 : The relation between erosion rate (M) and relative bend curvature (r/w)

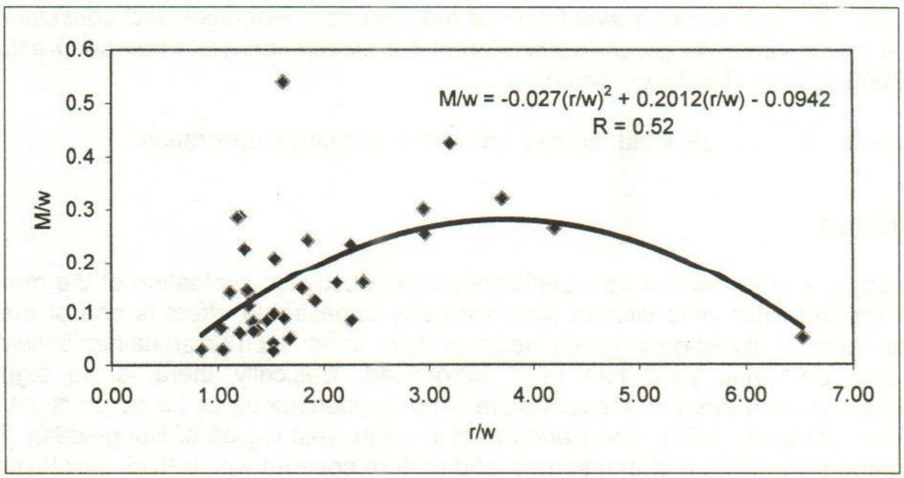

Figure 9: The relation between relative migration rate and bend curvature ratio

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SIMULATION OF GROUND WATER OF THE BARIND AQUIFER AND ESTIMATION OF THE SUSTAINABLE YIELD

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Abstract

Demands are gradually increasing but resources are limited. For the sake of sustainability of the Project, extensive research is a must to evaluate quantitatively the extent of available resources. The Barind Tract is characteristically dry area, natural recharge is quite uncertain but development of this area is closely related with the intensive use of its vast land with ensured irrigation. For this purpose, estimation of possible amount of ground water abstraction considering its future demand is important for the sustainability of the project. A two dimensional aquifer simulation model (ASM) has been calibrated and used to predict the sustainable yield of the groundwater basin. In calibrating the model, groundwater recharge is used as the variable and all other aquifer properties are used as obtained from the field. Sustainable yield has been estimated considering constraints of economic lifting head and limiting draw down. From aquifer simulation model the average equivalent annual recharge is found from calibration test and through simulation of the historical response and considering the dynamic behavior of the groundwater system the sustainable yield has been estimated up to 2000 A.D. for the Barind basin.

Key words: Sustainable yield, aquifer simulation, recharge, calibration.

Introduction

In managing a groundwater basin efficiently and effectively, evaluation of the maximum annual groundwater yield without producing any undesirable effect is one of the most important aim. In investigating such yields, certain terms, such as sustainable yield, safe yield, and perennial yield has been introduced. Basically there is no significant difference between them, these terms are almost synonymous as far as practical use is concerned (Kashef, 1987). The Barind tract in north-west region of Bangladesh (Fig. 1) is characteristically dry and of elevated land feature covered with a thick clay layer up to a depth of 30m in some places and underlying a sand layer which serve as the aquifer. Natural recharge in this region is quite uncertain because of the thick clay layer and the aquifer may be treated as the confined one (Ahmed and Burgess, 1993). Development of this area is closely related with the intensive use of vast land, which is now under utilization. Utilization of this land is possible only under ensured irrigation. The Govt. has a plan to develop the irrigation facilities by optimum utilization of available ground and surface water (BWDB, 1989). The available surface water for irrigation purpose is very low in comparison with the demand. In this context under groundwater utilization programme, installation of deep tube wells have been proposed (2000 in 1st phase and

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3000 in 2nd phase). The sustainability of this large-scale abstraction programme has been questioned by environmental scientists (Ahmed et al., 1995). The present rate of extraction for irrigation is $1.2*10^9$ m^3/year which may lead to exhaustion of the aquifer in nearly 53 years time (Khan & Sattar, 1995). The rivers around Barind tract have an effluent character, which gain water from the aquifers almost throughout the year.

The groundwater table is lowering rapidly and the whole region is in the acute state of deforestation. Indiscriminate groundwater development may activate deforestation trend (Khan & Sattar, 1995). In this context estimation of sustainable yield of groundwater in Barind tract will unquestionably assist in proper management and planning of environmentally viable abstraction schemes.

Literature review

The term "sustainable" indicates something that go continuously or can bear without breaking or falling (Fowler&Fowler, 1964; Hornby, 1994). It is known that continuous damage of life support systems for any development work is not possible without paying a price for that.

Sustainable development as defined by the World Commission in Environment and Development (1987): " development that meets the need of the present without compromising the ability of the future generations to meet their own demands."

An Canadian Water Resources Association (1993) defined sustainability as "wise management of water resources achieved by a genuine commitment to social equity for present and future generations."

ASCE (1972) introduced the term 'perennial yield' which is a similar term to sustainable yield. They defined perennial yield as the variable annual yield that can be continuously withdrawn under specified operating conditions without introducing undesirable results.

Shahin (1991) defined safe yield in investigating the groundwater resources at different places of Egypt as the maximum amount of withdrawal of water that will not lead to any further intrusion of the sea water into the aquifer, the draw down does not fall below the assumed limits and the Nubian sand stone reservoir will not exercise any considerable mining.

Methodology

A simple mathematical model (implicit finite difference scheme) is to be used to simulate the groundwater flow and to predict the sustainable yield of the proposed basin considering the total demand.

Numerical model for groundwater flow simulation

Groundwater movement in an aquifer can be described by the Partial Differential Equation (PDE) derived from continuity and Darcy's equations as given below:

$$\frac{\partial}{\partial x}(T_x \frac{\partial h}{\partial x}) + \frac{\partial}{\partial y}(T_y \frac{\partial h}{\partial y}) = S\frac{\partial h}{\partial f} + Q$$

where,

T_x and T_y = coefficient of transmissivity along x and y directions, respectively (m^2/s),
h = groundwater level (m)
S = storage coefficient (dimensionless)
t = time (s), and
Q = withdrawal and recharge ($m^3/s/m^2$).

The model area can be discretized into an asymmetric network of polygons.

An implicit finite difference equation from equ^n at a typical node can be written as (Thomas, 1973):

$$\sum_i Y_{i,s} (h_i^{j+1} - h_s^{j+1}) + A_s (R - E) / \Delta t = (A_s S_s / \Delta t)(h_s^{j+1} - h_s^j)$$

where

$Y_{i,s}$ = ($w_{i,s}$ * $T_{i,s}$)/$L_{i,s}$= conductance of path between nodes i and s (m^2/s)
$w_{i,s}$ = width of the side between nodes i and s(m),
$T_{i,s}$ = co-efficient of transmissivity of the path between nodes i and s(m^2/s),
$L_{i,s}$ = distance between nodes i and s(m),
h_i = groundwater levels at node i(m),
h_s = groundwater level at node s (m^2),
A_s = area of node s (m^2),
R = recharge during Δt (m)
E = withdrawal during Δt (m),
S_s = storage coefficient (dimensionless) at node s,
j = previous time level, and
j+1 = forward time level.

$$W_s^{j+1} = - h_s^{j+1} (\sum_i Y_{i,s} + A_s S_s / \Delta t) \Delta t + \Delta t \sum_i h_i^{j+1} Y_{i,s} + A_s R^{j+1} + A_s S_s h_s^j$$

Rearranging equation (4.19) we get,

where,

$$W_s^{j+1} = A_s E^{j+1}$$

here W_s^{j+1} is the objective function and h_i and h_s are considered as the decision variables.

In the present study a two dimensional aquifer simulation model (ASM) which is based on a solution of governing partial differential by finite difference method, version 5.0 developed by Kinzelbach and Rausch (1989) is used. The model can be used with multiple options (steady state/ time varying flow conditions, homogeneous/ inhomogeneous aquifer, confined/ phreatic/ leaky aquifer, isotropic/ anisotropic aquifer, injecting/ pumping well, constant boundary fluxes, temporally/ spatially variable groundwater recharge from precipitation etc.). In flow computation IADI (Iterative Alternating Direction Implicit) method or the PCG (Preconditioning Conjugate Gradient) method 1 and 2 can be chosen. In this study PCG method - 2 is used which seems faster and gives quick convergence in the iteration process.

Simulation of barind aquifer

Discretization of model area

The project area is divided into rectangular grids with the nodal spacing of Δx and Δy in x and y directions. Total extension of the project area in x directions and y direction are approximately 50 km and 90 km respectively. The value of Δx and Δy are selected as 1000m and 1500m respectively and the number of finite elements considered in x and y directions are 50 and 60 respectively. Equal nodal spacing is considered althrough because wells are distributed almost uniformly with the little exceptions at Mohadevpur and Dhamairhat. The total abstraction rate at Mohadevpur and Dhamairhat is relatively higher than the other places. But this small variation can be neglected. As the total number of wells within the project area in 1995 is much higher as shown in Table-1.

Boundary conditions

Well defined hydrogeological formations are considered for boundaries of the system. The area enclosed by the rivers Mohananda in the west, Ganges in the south and Atrai in the east are considered for flow simulation, as this area is well defined as well as pertinent data also available. The shape of the project area is irregular in north- western side but in other sides it is almost regular as shown in Fig. 3. Outside the project area almost similar types of hydro-geological conditions are prevailing. Hence a permeable boundary in all sides is assumed for analysis.All nodes outside the project area is taken as permeable but assumed having relatively less storage coefficient. As no pumping is considered in the model study for this region. Aquifer condition of the region is semi confined and in a few places unconfined (as storage coefficients between 0.05 to 0.13) but in the model study it is considered confined for simplification for the entire area. Three rivers are flowing within and aside the project area. Their effect to the groundwater storage is considered. In monsoon, river water level becomes higher than the nearby ground water level and it remains in the same condition for three to four months.The river Ganges is flowing along the southern boundary, the Punarbhaba-Mohananda and the Atrai are flowing through the western and eastern side of the project area. The geological conditions of the area is poorly to moderately favorable to infiltration.

Hydrogeological characteristics of the aquifer

Transmissivity(T) and storage coefficient(S) are the main hydrogeologic characteristics to the model study. The transmissivity values and storage co-efficient are known for wells in 15 different areas(Thanas) within the project area from the field tests (BWDB, 1989) as shown in Table -2. Average values of transmissivities for respective thanas are used in the model as local transmissivity. Transmissivity values used in the model varies from .0046 to .0156m^2/s as per field data for different places. Average values of the storage co-efficients for respective thanas are used as model input. Its values used in the model ranges from 0.05 to 0.13.The recharge is provided in the model as the main calibrating parameter. Average recharge rate obtained from the water balance study are used as the initial value. Variable recharge is considered in the model as found for different years for sustainable yield estimation.

Groundwater pumpage

Large number of deep and shallow deep tube wells are existing within the project area and it is gradually increasing from 1981 as shown in Figs. B-2 and B-3. Before 1989 ground water abstraction was not a problem at all in Barind. But as the demands gradually increases large number of deep and shallow tube wells are installed. Total number of existing wells and its arial distribution within the Barind from 1981 to 1995 is compiled and shown in Table -1. The total number of wells and pumpage within a thana (local administrative region) is summed up and represented by two to three large wells during model calibration for simplification. As the total number of wells is so large that it cannot be presented in the model.Only contribution of deep tube wells and shallow tube wells are considered in the model study. In this way total 29 numbers of representative wells are selected during model calibration. The locations of representative wells are selected within the respective thana boundary and considering the uniformity of distribution as far as possible. This pattern as well as potential of the aquifer is used for the over all calibration.

Model calibration

Calibration of mathematical model to reproduce the field observed head is very important. Hydrogeological parameters i.e., transmissivity (T) and storage coefficient(S) are given in the model as obtained from field data for respective places assuming it to be representative. Groundwater recharge is adjusted to calibrate the model with little adjustment in river water levels. It is assumed that initial condition of the water levels was in equilibrium state. This level occurs nearly at the same time(September) almost in every years. The year 1981 is considered for initial calibration as at that time there were nominal pumping. Several test runs are required with certain adjustments in each run to get the model results fairly agreeable with the prescribed initial levels. Model is calibrated for wet season(September) as well as dry season(March).

For wet season

Test run 1, model is run with default recharge value as 4.48E-09$m^3/s/m^2$ and very high storage coefficient for the rivers Ganges, Mohananda and Atrai as IE +10. Initial default head is considered as 23.50 m (little a bit higher than the average head).Surface water level and bottom level of these three rivers for wet season (September) are defined. Heads in 18 observation wells are used to compare the calibration condition.No leakage but small boundary fluxes is considered on test. Model is run for steady state and heads are compared at the end of run. It is observed that in most of the places calculated heads found lower than the observed heads as shown in Table -1 and Fig. 2. Maximum and minimum heads found in the models as 25.1 and 18m.

- Local scour which is caused by the local flow field around the piers and abutments.

Two types of scour according to the condition of sediment transport in the approach flow are generally observed and these are as follows;

- Clear-water scour, where material is removed from the scour hole, but not replenished by the approach flow.

- Live-bed scour, where the scour hole is continually supplied with sediment by the approach flow.

Local and localized scour can occur in one of two ways, such as clear-water scour and live-bed scour. Again local scour can be superimposed on both a general and a localized scour.

Involvement of a number of factors influencing the depth of scour makes the problem complex for estimation of scour depth analytically as well as experimentally. The most important factors which influence scour depth around pier and groyne are: shape and size of pier and groyne, angle of attack, flow depth and velocity, size and gradation of bed materials, bed slope, and fluid properties. In this sense, scouring is a complex phenomena. It varies in different rivers due to variable parameters like sinuosity and braiding intensity, bed slope, bank and bed materials and discharge. Influence of these factors on scour dimension have been studied by many investigators and are available in various scientific literatures. Description of scour process and along with results of investigation were given by Thomson and Davoren (1985), Holmes (1974), Mosley (1982), Inglis (1949), Laursen (1958) and others. Ahmed (1962) conducted some experiments on models of several bridge sites and proposed empirical equation for the estimation of total scour depth. Similar experiments were conducted at various parts of the world, e.g., Inglis (1949), Blench (1962), Simons et. al (1970), Shen et. al (1969), Lacey (1930), Bata (Acres, 1970), Tarapore (Acres, 1970), Larras (Acres, 1970), Bhuiyan (1991), Jain (1981), Ahmed (1995), Melville (1983), Rajaratnam and Nwachukwu (1983), Raudkivi and Ettema (1983) etc. Evaluation of various scour formulae have been undertaken by Jain and Modi (1986).

A detailed knowledge of the scour pattern of different types of piers, caissons, abutment, groyne etc. are of great value when dealing with the design of bridges, barrages, training works, flood control works, channel improvement and so on. The safe and economical design of bridge pier, caisson, barrage, dam, retaining wall, flood wall etc. requires accurate prediction of the maximum scour depth of the stream bed around them. An under-estimation of the scour will result in failure, while its over- estimation will increase the cost of the structures. Hence the knowledge of predicting scour depth under various conditions is essential. With this context the present study has been planned with the following objectives:

- To estimate the maximum scour depth using selected scour formulae against the available data from piers and groynes in Bangladesh

- To compare the estimated scour depth with the observed values and determine the predictive performance of the selected formulae.

Methodology

Scour and related data for the caissons of East-West Interconnector for the period of 1986 to 1997 have been collected from Bangladesh Power Development Board (BPDB). Necessary additional data on scour around piers of Hardinge bridge and groyne head were collected from M. Sc. Thesis (Kabir, 1984) and model studies conducted at RRI respectively. These were then be processed and compiled to usable form. Later the data was utilized to estimate the scour depths using some of the well known scour formulae. The presently selected formulae are those of Inglis (1949), Ahmed (1962), Blench (1962), and Shen et al. (1969) for piers and Ahmed (1953), Liu et al (1961) and Bruser (1991) for groynes. The basis of selection of these formulae are due to their regional validity. Few formulae were selected on the basis of recommendation from some previous evaluations (Kabir,1984). The predicted scour depths were then be compared with the observed scour value interms of discrepancy ratios. Statistical analysis have been conducted to finalize the order of suitability of these equations against the observed data.

Data collection

Data sheet of the year of 1986 to 1997 have been collected for the caissons of East-West Interconnector. In the data sheet scouring has been measured from the top of the caissons. Also the water level inside and outside of the caissons have been measured from the top of the caissons. The diameter of the caissons are of 35 feet. Average of the water level inside and outside of the caissons has been taken. The scour depth from water level have been found by deducting average water level from the scouring level. Discharge data have been collected from BWDB for the discharge station at Bahadurabad. The discharge data were than similitude for Aricha –Nagarbari station.

Jamuna/Brahmaputra River originates in Tibet on the north slope of the Himalayas and drains an area of about 550,000 km^2, extending over China, Bhutan, India, and Bangladesh. Its traverse path is about 2,740 km before meeting with the Ganges River at Aricha. The mean annual discharge is 20,000 m^3/s, the maximum discharge recorded till now is 100,000 m^3/s (in 1988), and the bankfull discharge is about is 48,000 m^3/s. The slope of the river within Bangladesh decreases in the downstream direction and is $8.5*10^{-5}$ at the upstream end, and $6.5*10^{-5}$ near the confluence with the Ganges. The bed material sizes also decreases from the upstream towards downstream part and ranges from 0.22 mm to 0.16 mm.

The Chezy roughness values of the river Jamuna at Sirajgonj vary between 40 $m^{1/2}/s$ for low flow and 100 $m^{1/2}/s$ for flood condition (Klaassen et al, 1988). In the Jamuna Bridge Project study, it was found that the Chezy number also varies between 40 $m^{1/2}/s$ for low flow and 100 $m^{1/2}/s$ for flood conditions based on BWDB discharge measurements (RPT/NEDECO/BCL, August 1989). In that study it was seen that the Chezy's roughness value varies with water depth. On the basis of these studies, the Chezy roughness value of 70 to 74 $m^{1/2}/s$ was used for discharge ranging between 65,000 to 75,000 m^3/s for main channel and 64 $m^{1/2}/s$ was used for minor channels. In the present study the same value is used for calculating velocity and other necessary required parameters.

The Hardinge Bridge crosses the river Ganges approximately in between Paksey and Bheramara. It consists of sixteen composite piers. Scour and related data of this bridge were collected from M. Sc. Thesis (Kabir, 1984).

Some data for scour around groyne head were collected from various physical model study reports conducted at River Research Institute.

Comparison of scour data

Caisson of the east-west interconnector

The scour data for East-West Interconnector were analysed on the basis of the equations derived by Inglis (1949), Ahmed (1962), Blench (1962) and Shen et. al (1969). An attempt was made to compare the field data with it's respective similar equations. **Figs. 1 to 3** were plotted for this purpose and the statistical equations obtained are given below in tabular form:

Name of the Investigator	**Equations (FPS system)**	**Relations obtained on the basis of field data**
Inglis (1949)	$D_t/b = 1.7\ (q^{2/3}/b)^{0.78}$	$D_t/b = 0.8807\ (q^{2/3}/b)^{0.2818}$ -- 1 $R^2 = 0.2873$
Blench (1962)	$D_t/y = 1.8\ (b/y)^{0.25}$	$D_t/y = 1.4596\ (b/y)^{0.7168}$ ------ 2 $R^2 = 0.7219$
Shen et. al (1969)	$d_s/y = 2\ [F^2(b/y)^3]^{0.215}$	$d_s/y = 3.8754\ [F^2(b/y)^3]^{0.3387}$--3 $R^2 = 0.7168$

Equations 1 to 3 are for East-West Interconnector and shown similarity to the respective equations proposed by various investigators. Only differences are in constants and in the value of the index power. Comparing equation 3 to Shen et. al formula, it is found that the variations are 93% and 57% in the values of the coefficient and index power respectively.

The degree of compliance of the scour formulae representing the interrelationship beteween the observed and predicted scour depths was determined by calculating a term known as discrepancy ratio. The discrepancy ratio (D.R.) is difined as the ratio of scour depth calculated by using any of the selected equation to the true or measured scour depth. This ratio for each individual data using all the selected formulae were computed. These discrepancy ratio were then analysed to evaluate the degree of performance of various equations in predicting the scour depth. This is actually done by determining the percentage of data coverage between a selected band of discrepancy ratio. A discrepancy ratio of unity means a perfect agreement of the predicted value with the measured value. The selected band of discrepancy ratio is one-half to two. The data coverage between the selected bands of unadjusted values of discrepancy ratio for the available data of all equations are shown below in tabular form.

Name of Equations	**% of data coverage for a range of discrepancy ratio**			
	0.5-2.0	0.25-4.0	0.125-8.0	rest
Blench equation	81	99	100	-
Shen et al equation	77	98	100	-
Inglis equation	40	89	99	1
Ahmed equation	32	56	73	27

From the criteria of data coverage between the discrepancy ratio of one-half to two as shown in the above table, Blench formula is found to the top of the ranking. This is followed by Shen et al equation, Inglis equation and Ahmed equation.

Piers of hardinge bridge

The scour data for Hardinge Bridge were analysed on the basis of the equations derived by Inglis (1949), Ahmed (1962), Blench (1962) and Shen et. al (1969). The analysis were done with field data and compared with the predicted data by the aforesaid equations. The scattered diagram were plotted with observed data against predicted data to observe the deviation from the line of perfect agreement (fig. 4). It can be observed that Ahmed's formula highly over predicted and some cases under predicted. So, it can be concluded that Ahmed formula is not suitable for the prediction of scour for Hardinge Bridge. It can be observed that Inglis, Blench and Shen et. al formula were over predicted and under predicted and is very close to the line of perfect agreement compared to Ahmed formula. It is also observe that some values lies on the line of perfect agreement for these equations.

An alternative attempt was made to compare the field data with it's respective similar equations. Figs. 5 to 7 were plotted for this purpose and the statistical equations obtained are given below in tabular form:

Name of the Investigator	**Equations (FPS system)**	**Relations obtained on the basis of field data**
Inglis (1949)	$D_t/b = 1.7\ (q^{2/3}/b)^{0.78}$	$D_t/b = 1.9216\ (q^{2/3}/b)^{0.1612}$ ---------- 4 $R^2 = 0.2533$
Blench (1962)	$D_t/y = 1.8\ (b/y)^{0.25}$	$D_t/y = 1.8288\ (b/y)^{0.7576}$ ------------- 5 $R^2 = 0.8144$
Shen et. al (1969)	$d_s/y = 2\ [F^2(b/y)^3]^{0.215}$	$d_s/y = 8.7846\ [F^2(b/y)^3]^{0.6422}$ ------- 6 $R^2 = 0.4546$

Equations 4 to 6 are for Hardinge Bridge and shown similarity to the respective equations proposed by various investigators. Only differences are in constants and in the value of the index power. Comparing equation 5 to Blench formula, it is found that the variations are 2% and 200% in the values of the coefficient and index power respectively.

Similar analyses were done for Hardinge Bridge as in the case of East-West Interconnector. In this case, the selected band of discrepancy ratio is 0.8-1.25. The data coverage between the selected bands of unadjusted values of discrepancy ratio for the available data of all equations are shown below in tabular form. The equations selected were arranged according to their performance based on data coverage.

Name of	**% of data coverage for a range of discrepancy ratio**				
Equations	0.8-1.25	0.5-2.0	0.25-4.0	0.125-8.0	rest
Blench equation	90	100	-	-	-
Shen et al equation	74	100	-	-	-
Inglis equation	63	100	-	-	-
Ahmed equation	11	26	74	100	-

rom the criteria of data coverage between the discrepancy ratio of 0.8-1.25 as shown in the above table, Blench formula is found to the top of the ranking. This is followed by Shen et al equation, Inglis equation and Ahmed equation.

Scour around groynes

The scour data of groyne were analysed on the basis of the equations derived by Breuser (1991), Liu et. al (1961) and Ahmed (1953). The analysis were done with laboratory test data and compared with the predicted data by the aforesaid equations. An attempt was made to compare the laboratory test data for groyne with it's respective similar equations and for this purpose Figs. 8 to 10 were plotted and the statistical equations obtained are given below in tabular form:

Name of the Investigator	Equations (SI unit)	Relations obtained on the basis of Lab. Data
Breuser (1991)	$h_0+h_{sm}= 2.2[Q/(B-b)]^{2/3}$	$h_0+h_{sm} = 10.273\ [\{Q/(B-b)\}^{2/3}]^{0.673}$ ----- 7 $R^2 = 0.5042$
Liu et al (1961)	$h_s/h_0= 2.15\ (b/h_0)^{0.4}F_r^{0.33}$	$h_s/h_0 = 2.0556\ [(b/h_0)^{0.4}F_r^{0.33}]^{0.5659}$ ----- 8 $R2 = 0.6583$
Ahmed (1953)	$H_s = 1.34\ (q^2/f)^{1/3}$	$H_s = 9.6746\ [q^2/f)^{1/3}]^{0.6691}$ --------------- 9 $R^2 = 0.5084$

Equations 7 to 9 are for groynes and shown similarity to the respective equations proposed by various investigators. Only differences are in constants and in the value of the index power. Comparing equation 8 to Liu et. al formula, it is found that the variations are 4.7% and 43% in the values of the coefficient and index power respectively. This means that Liu et. al formula is suitable for this case.

Similar analysis were also been done for laboratory test data of groyne (RRI) as in the case of East-West Interconnector and Hardinge Bridge. In this case, the selected band of discrepancy ratio is 0.8 - 1.25. The data coverage between the selected bands of unadjusted values of discrepancy ratio for the available data of all equations are shown below in tabular form.

Name of Equations	% of data coverage for a range of discrepancy ratio				
	0.8-1.25	0.5-2.0	0.25-4.0	0.125-8.0	Rest
Breuser equation	38	100	-	-	-
Ahmed equation	13	75	100	-	-
Liu et al equation	-	100	-	-	-

From the criteria of data coverage between the discrepancy ratio of 0.8-1.25 as shown in the above table, Breuser formula is found to the top of the ranking. This is followed by Ahmed and Liu et al formula.

Conclusions

The deviation of scour depth obtained from the empirical equations and that of the field data may be due to the following reasons:

(a) Development of the empirical formulae from the small scale laboratory investigation where many parameters were kept constant, while the conditions differ in the actual field.
(b) Conditions and nature of bed material may vary considerably at different points and depths, data for which were not available.
(c) The scour also depends on the skewness of the flow and on the state i.e. weather the flood is rising or falling. Precise observations regarding direction of current and state of flood were not available.

After analyzing the data on the basis of scattered diagram, statistical analysis and discrepancy ratio, it is concluded that Blench (1962) formula could be the best predictor among those compared in this study for East-West Interconnector and Hardinge Bridge. It is also concluded that Breuser (1991) formula could be the best predictor for laboratory test data of groyne.

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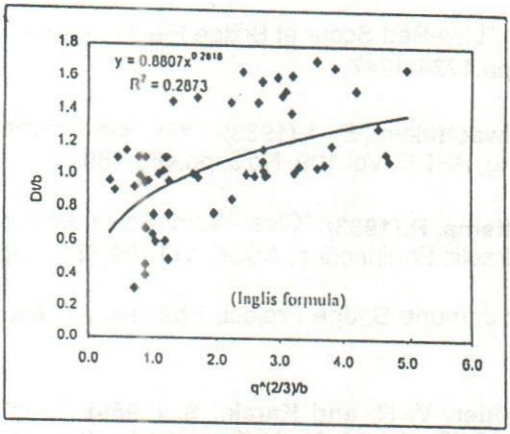

Fig. 1 : Relation between Dt/b and q^(2/3)/b for East-West Interconnector

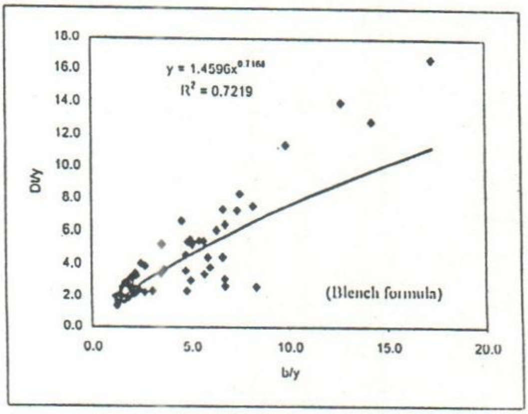

Fig. 2 : Relation between Dt/y and b/y for East-West Interconnector

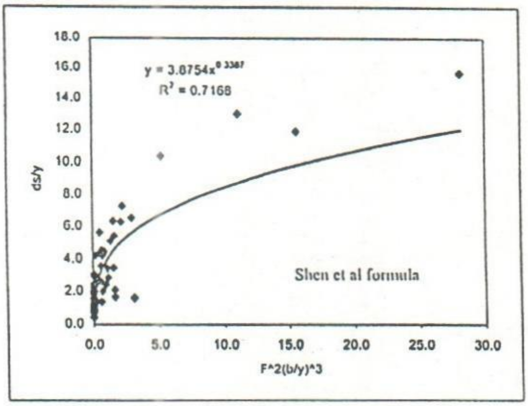

Fig. 3 : Relation between ds/y and F^2(b/y)^3 for East-West Interconnector

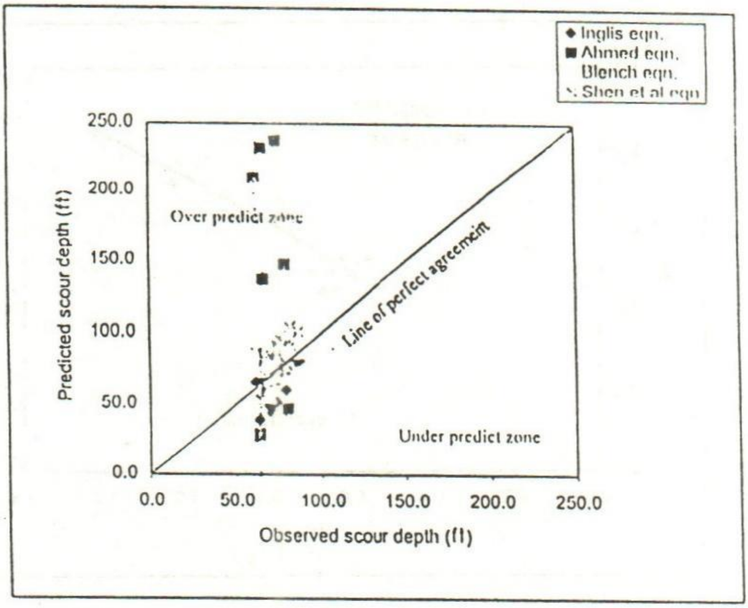

Fig. 4 : Comparison of predicted and observed value of scour depth for Hardinge Bridge

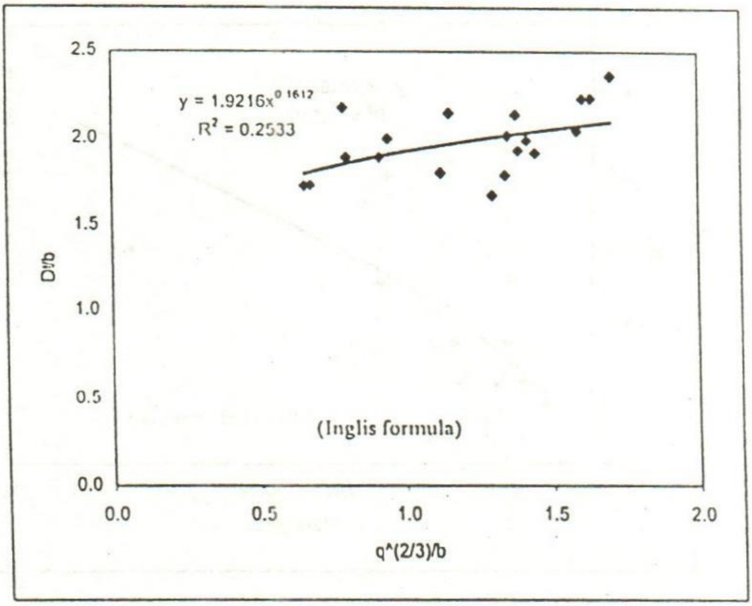

Fig. 5 : Relation between Dt/b and q^(2/3)/b for Hardinge Bridge

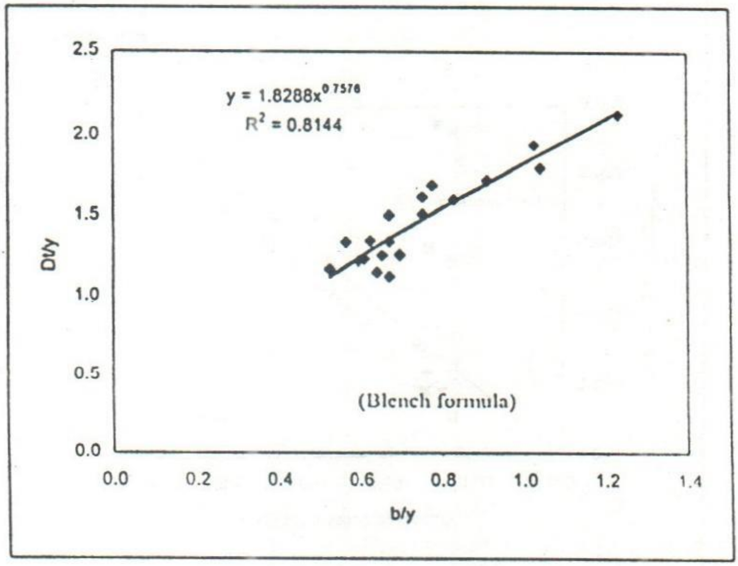

Fig. 6 : Relation between Dt/y and b/y for Hardinge Bridge

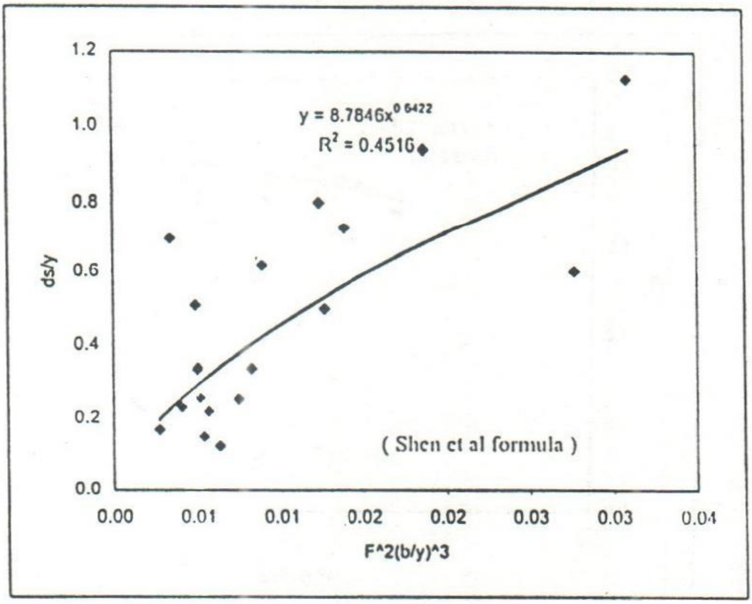

Fig. 7 : Relation between ds/y and F^2*(b/y)^3 for Hardinge Bridge

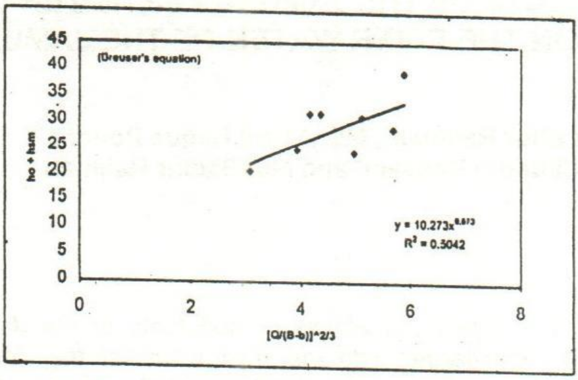

Fig. 8 : Relation between (ho + hsm) and [Q/(B-b)]^2/3 for groyne

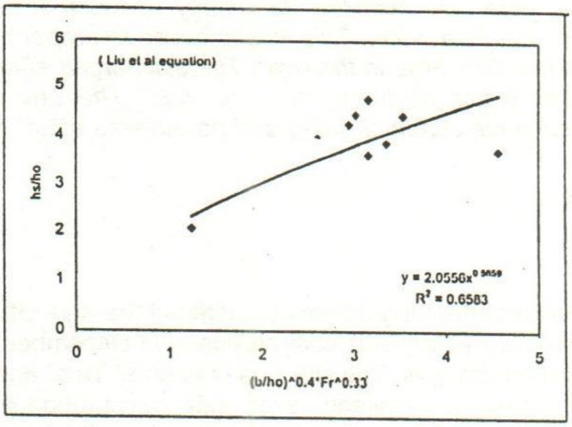

Fig. 9 : Relation between (hs / ho) and (b/ ho)^0.4* Fr^0.33 for groyne

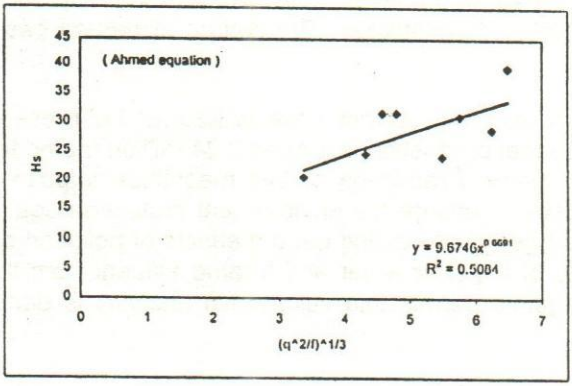

Fig. 10 : Relation between Hs and (q^2/f)^1/3 for groyne

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POLLUTION EFFECTS OF THE JAMUNA FERTILIZER COMPANY LIMITED ON THE RIVER WATER OF THE JAMUNA

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Abstract

A study carried out to investigate the effects of pollutants of the Jamuna Fertilizer Company Limited (JFCL) discharged into the river water of the Jamuna. For this purpose, the results of the chemical analysis of the water samples collected from the river during 1996-97 periods were studied. The different parameters of the effluents drained to the river water were also checked. The study shows that the pollutants in general had little effects on the river water. This was because of proper treatment of the pollutants effluent prior to the discharge in the river. The discharged effluents were also readily diluted in the river water all throughout the year. The environment in the surrounding area is natural since commissioning and no adverse effect reported due to pollutants discharge.

Introduction

Fertilizer is one of the most important ingredients to increase the agricultural production. With this aim Jamuna Fertilizer Factory was commissioned in December 1991. The raw materials are air, water and natural gas. The site was chosen at Tarakandi, Jamalpur on the left bank of the river Jamuna. Tarakandi is an extended portion of haor area of greater Jamalpur. During monsoon the whole area used to be under 3 m to 5 m depth of water. The whole area of 194 bigha has been reclaimed by dredging from the Jamuna and has been raised above flood level. The site is centrally located and well communicated by rail, road and waterways. The source of natural gas supply is also near the site (Figure -1).

Since commissioning the factory is producing urea fertilizer and at present its production amounts to about 25% (annual production is around 2.24 million metric tons) of the total urea production of the country. Production of this magnitude imposes great deal of responsibility on the industry to ensure the environment protection against pollution. In this paper an attempt has been made to find out the effects of pollution discharge to the river water. Water sample of the river water and treated effluent from the factory were analyzed and checked against permissible values. No analysis is done regarding air pollution.

Pollutants Effluent from the Factory

Like other fertilizer factories, Jamuna Fertilizer Factory has (i) utility unit (water treatment unit), (ii) ammonia plant unit (iii) urea unit (iv) bagging and finishing unit and

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(v) maintenance unit. These units discharge different type of pollutants containing ammonia, acid or alkali, wash water, mud and aluminum hydroxide sludge, natural gas condensate, lubricating oil, urea condensate, diethanol amine, potassium carbonate solution, vanadium oxides, urea dust etc. The type of emissions and nature of pollution of the above effluents is explained in Table-1.

Table-1: Emissions and effluents produced from the jamuna fertilizer factory limited and the nature of the pollution

Nature of Pollution	Type of Pollution
Emissions	Fumes, Hydrogen Sulphide
Air Pollution	Carbon monoxide, Carbon dioxide, Oxides of Nitrogen, Hydrocarbons, Chlorine, Ammonia, Urea, Formaldehyde, Dust mainly from Ammonia-Urea Plant
Effluents	Suspended Solids, Hydrochloric Acid, Sulfuric Acid, Sodium Hydroxide, Ammonium Hydroxide, Methanol, Urea, Formaldehyde, Water Pollution, NH3 Solution (ppm level), Hydrocarbons, Phosphates, biocides, used resins, oil and grease.

pollution from urea plant

About 15 metric tons of condensate is discharged from this unit. This contains about 200 ppm urea and 50 ppm NH_4. Before discharging into the river water, the condensate is treated by diluting with fresh water to bring the concentration of the urea to the desired level.

pollution from ammonia plant

About 70 metric tons of the condensate per hour is produced from this unit. Of this 50-55 metric tons of the condensate is made free from dissolved ammonia by stepping process and reused in the process. The rest 15-20 metric tons are drained to the river water. These pollutants have effect on the growth of fish flora and fauna in the river.

pollution from utility plants

The utility plants comprises (I) Water Treatment Unit, (ii) Waste Water Treatment Unit, (iii) Cooling Tower, (iv) Nitrogen Generation Plant and (v) Ammonia Storage and Bottling Unit. Resin of demineralized plant is washed with acid and alkali. The waste is discharged to the river water after proper treatment. Blow down cooling water to river contains phosphate-based chemicals and biocides. They increase the growth of unwanted plant and algae. When decomposed, they cause pollution of the river water and ultimately causes death to aquatic animals like fishes etc. Ammonia is vent (allowable range) from Ammonia and Urea plants in atmosphere. This polluted surrounding atmosphere. Sometimes chlorine contaminates air through leakage of valve of chlorine cylinder.

Pollution from bagging and finishing plant

The pollutants from this unit include (i) Urea dust from granular urea, (ii) Free ammonia formaldehyde and carbon dioxide and (iii) Dust from packing material – Jute and Pothene Bags. Workers working in this area are generally subjected to be affected more in respect of health and hygiene.

Sample collection

In order to avoid environmental pollution, the effluents discharged from the different units are treated in the neutralization pit and brought to desired concentration before they are finally discharged to the river water. This is explained in Figure-2.

The process is being practiced since commissioning of the factory till the middle of the year 1992 and complete tests of the waste disposal material (contaminate) were done to check the water quality of the river Jamuna. However, collection of about 1 litre of river water has been collected daily at 10:30 pm from the inlet point of the classifier for routine laboratory tests of the pH, Total Dissolved Solid (TDS) conductivity, turbidity, total hardness (as calcium carbonate), calcium and magnesium hardness in ppm. These tests result are recorded as shown in Table-2.

Table 2: Components showing significant seasonal variations (1996-1997)

Sl. No.	Components	November to April	May to October
01	Total Dissolved Solid (ppm)	150-250	55-100
02	Conductivity (micromos/cm)	220-400	100-120
03	T-hardness (ppm) as $CaCo_3$	150-220	40-50
04	Ca-hardness(ppm) as $CaCo_3$	100-140	30-45
05	Alkalinity(ppm) as $CaCo_3$	100-195	40-85
06	Turbidity (TNU)	5-20	100-200
07	Nitrite No_2 (ppm)	0.17-0.60	0.06-0.21
08	Nitrate No_3 (ppm)	1.24-1.8	0.62-1.039
09	Ammonium NH_4 (ppm)	0.35-1.0	0.3-0.8
10	Sulfate (ppm)	4.4-7.0	6.4-8.1
11	Potation (ppm)	1.8-2.5	1.5-2.02
12	Iron(ppm)	0.16-0.66	0.24-0.60
13	Chlorine(ppm)	1.48-4.0	1.0-2.5
14	Sodium (ppm)	3.7-5.6	2.5-3.6
15	Silicate (ppm)	5.3-15	4.9-7.4
16	p^H	7.4-8.0	7.5-8.2

From 1993 regular program for adequate tests have been taken. The effluent (waste) disposal point is about 100 meter downstream of the pump intake location for withdrawal of fresh water for the factory. Samples of water are collected from this disposal location and tests for pH value, conductivity in ppm, temperature, BOD, COD and ammonia content (as ammonium) are done daily and recorded as shown in table-3.

Table-3: Jamuna river water quality before and after commissioning of jamuna fertilizer company limited.

Components	Test result on 19-6-90	Test result on 22-7-90	At Discharge Point (96-97)	At 100 m d/s (96-97)	Remarks
pH	7.7				
TDS(ppm)	73.0	81.0			
Suspended Solids(ppm)	45.0	44.0			
Carbonates & Bi-carbonates(ppm)		450			
Chlorides(ppm)	4.5	4.8			
Sulfates(ppm)	N/T	1000			
Trace Organic matter	4	4			
Iron(ppm)	0.8	0.40			
Total N_2(TKN) excluding Nitrite(ppm)	0.4	0.8	0.96	0.06	
Ammoniacal Nitrogen(ppm)	N/T		5.3	0.27	
Nitrate(ppm)		1.15	1.2	3.0	0.68
DO(ppm)			6.3	6.8	
BOD p pm)	0F80	1.0	3.8	2.5	
COD(ppm)	1.0	1.2	5.37	3.58	
Temp(^{0}C)	26	24	23	22	

Testing

Modern equipment and standard practices are followed in physical and chemical tests. The pH value is determined by TOA-pH meter model HM-55. Measurement is possible with an accuracy of +0.1 pH unit. Specific conductivity is measured in TOA-conductivity meter model CM-405. Alkalinity, Hardness and Chloride are determined by titrimetric method. For the determination of turbidity silicate, sulphate, nitrite, ammonium and iron, Spectro-photometric method is applied. Spectro-photometer model UV-120-01 of Shimadzu Ltd. is used for this purpose. The accuracy range is + o.5. Sodium and Potassium are determined photometrically by Shimadzu Atomic absorption spectro photometer Model AA- 625-11. Following the universal standard practices, it was determined the Dissolved Oxygen (DO), Biological Oxygen Demand (BOD) and Chemical Oxygen Demand (COD). Since no radioactive materials are discharged as effluents no test is done in this respect.

The Jamuna river

As mentioned earlier the factory is located on the left bank of the river upstream of Jagannathganj Ghat. The river serves the purposes of the factory. Firstly it serves as a source of fresh water for the factory. Secondly as a pollutant disposal area of the factory and thirdly as one of the cheapest transport routes to different places of the country.

The Jamuna, like other rivers of the country is characterized by the monsoon floods between July and October. During dry season (November-May) the discharge is low and the river is non-tidal. A typical hydrograph of the river is shown in Fig 3.

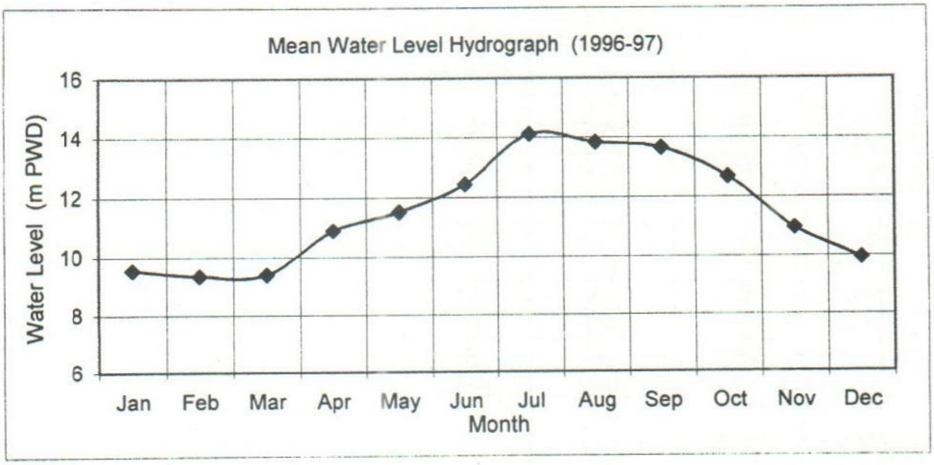

Figure 3: Average Water Level Hydrograph of the Jamuna River at Jagannathgonj Ghat

The highest and lowest discharge recorded at forward position of Jagannathganj ghat 19880 and 6627 cumecs respectively. The maximum discharge of the river is thus about three times than that of minimum discharge. In the rainy season the river overflows vast areas, creates high turbidity and carries large quantities of suspended sediments.

Results and discussions

Table 2 shows the seasonal variations of the various components and properties of river water used as raw material for the factory. Except p^H, and turbidity, ammonium (ppm), and sulphate (ppm) concentrations of all the components and properties decrease in the rainy season (May –October) due to higher dilution and turbidity. The values, except hardness and alkalinity, however are well within the allowable values both in the dry season and rainy season. The concentration of Specific Conductivity, Hardness and Alkalinity increases in the dry season due to the less quantity of water in the river. A lime treatment project to the clarified water is now under implementation to decrease the hardness, alkalinity and conductivity in the dry season.

Table-3 explains the effect of pollution of Jamuna Fertilizer Factory on river of the Jamuna. The p^H of the Jamuna River water varies from 7.4 –8.2 (table-2) throughout the year and this value is little affected by the effluent discharge. At 100 metro down-stream from the discharge point the average concentration of ammonia is 0.27 PPM which is much below the allowable value 0.5 ppm. The DO, BOD & COD values indicate that the treated effluents are not septic and organic fouling is minimum.

Test run 2, model is run with increased recharge rate (6.4E-09 $m^3/s/m^2$) at certain locations as such as Patnitola, Nachol, Tanore, Porsha and Gomostapur. Results appeared improved compared to test run 1.
Test run 3, water levels in the river Mohananda is adjusted little a bit. All other parameters remains same. Results relatively improved. Model heads at observation well points is shown in Table -3 and their comparison with initial heads is shown in Fig. 2. Water balance for the whole area is checked and found relatively better than that in test run 2.

Test run 4, the recharge values for the north-western part (Porsha, Patnitola) and Tanore is increased further by 120 percent i.e., from 6.64E-09 to 7.68E-09 $m^3/s/m^2$ and river water levels are checked and adjusted. All other parameters remain same as before. Results improved at almost all points as shown in Table -3 and Fig. 2. Groundwater contour lines observed in model is presented in Fig. 3.

Test run 5, in this test model is run with adjusted river water levels and found no significant response to model heads. Comparison of heads at 18 observation well points with initial heads at the corresponding points are shown in Table -3 and Fig. 2.

For dry season

Test run A, to adjust the model heads near observation well points river water levels in the Atrai, Mohananda and Ganges are changed by trial as well as recharges are adjusted correspondingly. Pumping rates increased as in dry season. The default initial head is taken as 14m.Maximum number of iteration and mean error in computation are selected as 3600 and 0.000001. Heads obtained in the model is shown in Table -4.

Test run B, in this test river water levels in the Mohananda near Nachol are further are further adjusted as well as recharges rate increased little a bit to 4.8E-09$m^3/s/m^2$ keeping all other parameters same as before and found the heads at steady state relatively fairly matched with the initial heads at 18 observation well points as shown in Fig. 4. But still significant head differences observed near well points 44, 45, 41. These well points are located at Nachol and Dhamairhat thana where ground levels near well points are at 39 and 26m (PWD) respectively.

Test run E, model is run with the increased recharge rate (from 4.8E-09 to 5.82E-09$m^3/s/m^2$ i. e., 30 percent increased) for the Nachol and adjusted river water levels. Heads in the model are found relatively close to observed heads as shown in Table -3 and Fig. 4.

Test run F, model is run with the reduced pumping rate for the Nachol as from data it is found that in Nachol pumping is relatively low. River water levels are adjusted slightly for more refinement. Heads in the model found more close to the observed heads as shown in Table -3 and Fig. 4. Groundwater contour lines found in the model is shown in Fig. 5.

Recharge required calibrating the model for different season and year is important. Weighted average recharge required to calibrate the model for 1981 is found equivalent to around (7.68E-09*0.55 + 4.48E-09*0.2 + 3.2E-09 *0.25) 190 mm and for 1993 it comes around (6.4E-09*0.80 + 3.2E-09*0.2) 182 mm.With the aquifer parameters and recharge rate finally selected for the fairly calibrated model, response of the Barind aquifer has been tested with the existing well distribution pattern. The model is run to find out the sustainable yield of the groundwater basin of Barind tract. The sustainable yield is defined here from the standpoint of progressive reduction of groundwater resources and economic pumping conditions.

Prediction criteria

Economic pumping

If the energy cost and water charge increase proportionately in future then the maximum allowable lifting head for the Barind Tract can be calculated based on the present electricity and water charge. For the constant discharge, energy required is directly proportional to the lifting head. Assuming combined efficiency to be 65%, the maximum allowable lifting head for the Barind is calculated and found to be 36m with some allowance. As though in most of the cases screening depth of deep tube wells are more than 70 meters, shallow tube wells are incapable of operating with high lifting heads(MPO, 1987).

Drawdown

The maximum drawdown is calculated with the consideration of position of the pumps, position of groundwater table and types of pump used. For the deep tubewells, the submersible pumps are used, so the ground water level must always be at the higher level then the pump position. Considering these points maximum allowable drawdowns for 6 selected well positions are calculated and found to be 16.34 m for Nachol and 16.12 m for Godagari considering the worst situations.

Estimation of sutainable yield

In Estimating sustainable yield for the Barind Tract economic pumping and maximum drawdown is considered. For this purpose entire area is divided into 12 parts and each part is to be individually tested for simplification. In this way total pumping for the Nachol is distributed within 15 lumped wells and out of these 15 equivalent lumped pumping wells, 3 are selected to observe the draw down against the time dependent dynamic response to the aquifer. These three wells are selected as uniformly as possible and considering the relative stress from the overall project area. In this area, in dry period most of the water demands are fulfilled by the ground water supply. In 1989 total groundwater abstraction was 715Mm3. Calibrated model is run first with this rate of pumping for dry period i.e., for 150 days and draw downs checked at the end of each pumping. Heads in the model are presented for scenario-1 in Figs. 6, 7 and 8. In the pumping wells P4 and P6, maximum drawdowns observed are within 5m at the end of 2000 A.D. In this test imposed abstraction rate is 139mm/year and its probable response upto 2000 A.D. is estimated. Variable recharge rate as found from water balance study are used for respective years. Model is run first for 1981 (calibrated with the recharge for 1981) and with the heads found after computation model is run for the year 1982 taking these heads as initial heads(importing the initial head file of 1981). In this way models are run for remaining years up to 2000 A.D. with the recharge for respective year and heads found at the end of run are listed. For the years 1996 to 2000 as there are no recharge data, average recharge obtained from water balance study is used in the computations. Maximum drawdowns found with this rate of pumping are 4.75 m, 5.72 m, and 1.60 m in wells- P4, P6 and P10 respectively (scenario-1). Actually these drawdowns would be different (less) in the field as in model lumped wells are used for simplification.

Model is run with the pumping rate 50 percent higher than the 1989 pumping rate as used in the first test. Unsteady with time dependent recharge conditions are imposed in the model as in previous test but the computation is done in more better way. For the total 20 years single model is run using pivotal weighted recharge technique. Results of the computation are presented in Figs. 6, 7 and 8 for the scenario-2. Maximum draw down found after 2000 A.D. is 9.47m in well P6. Draw downs in well P4 and P10 are found as 8.28m and 3.18m respectively. Maximum allowable draw down from economic as well as limiting draw down considerations for Nachol is 16.34m which is higher than the draw downs found in this test.

Test-3 , model is run with the increased pumping rate and in the same way as in test-2. In this test pumping is increased by 33 percent from previous test (test-2) and heads found in the model at the end of computations for 1981 to 2000 A.D. are listed and their comparison are presented in Figs. 6, 7 and 8 for the scenario-3. Drawdowns found in well P4, P6 and P10 for this rate of pumping as 11.37m, 12.76m and4.42m respectively (Figs.6, 7 and 8). Corresponding abstraction rate is (139*2 =) 278mm/year may be considered as sustainable yield up to 2000 A.D. with some safety margin. This yield is estimated on the basis of localized modeling for the Nachol thana and may be treated as the sustainable yield for the Barind as actually the model is prepared for the whole Barind but due to limitations of the model in estimating the sustainable yield small area have to used for relatively better result.

Similarly for the Godagari thana model is run with 15 selected wells. Out of these 15 wells, three wells are selected to observe draw down against pumping with time dependent recharge (different for individual year). In first test, model is run with pumping rate as in 1989 (2 m^3/s i.e., 63 $Mm^3/year$) for the period of 1981 to 2000 A.D. with recharges input as obtained from water balance study for 1981 to 1995 and for the period of 1996 to 2000 A.D. average recharge is used. Drawdowns found at the end of each year are presented in Figs. 9, 10 and 11 for scenario-1. Maximum draw down found is 7.41m in well G1 where as maximum allowable draw down for Godagari is 16.12 m. In 2nd test, model is run with the increased pumping rate (150 percent of 1989 pumping rate) and draw downs found at the end of each year are presented in above mentioned tables and figures for scenario-2. Maximum draw down found is 11.42 m which is also less than the maximum allowable.

In 3rd test, model is run with 133 percent of pumping as used in 2nd test and resulting draw down at the end of each year are presented in above mentioned tables and figures for scenario-3. Maximum draw down found is 15.44 m which is almost close to the maximum allowable.

As a trial in 4th test, model is run with 125 percent of pumping as used in 3rd test and the resulting draw downs are presented in Figs. 9, 10 and 11 for scenario-4. Maximum draw down found as 19.45 m after 2000 A.D. which is higher than the maximum allowable. On the basis of these test results the pumping rate used in 3rd test is selected as the sustainable yield for the Godagari thana which comes as around two times of the pumping rate in 1989. That is, 139*2 = 278 mm/year. From the results of these two typical thanas it is assumed that the sustainable yield for the Barind basin is around 278 mm/year up to 2000 A.D.

Conclusion

Following conclusion can be drawn from the study:

Results from aquifer simulation model indicated that long term safe yield i.e., sustainable yield for the Barind aquifer would be around 278mm/year which is higher than the abstraction rate in 1995 (171mm/year). That is, the abstraction rate may be increased further for irrigation purpose. A comprehensive plan for optimum utilization of the groundwater would be prepared for the long term. But before finalization of the utilization plan more detail investigation with more powerful mathematical model would be required.

Acknowledgement

The study was carried out in preparing Master of Engineering thesis at the School of Civil Engineering, Asian Institute of Technology, Thailand. The authors are grateful to Prof. Dr. A.D. Gupta, AIT, for his unfailing guidance in carrying out this study and to AIT & JJ/WBGSP for funding.

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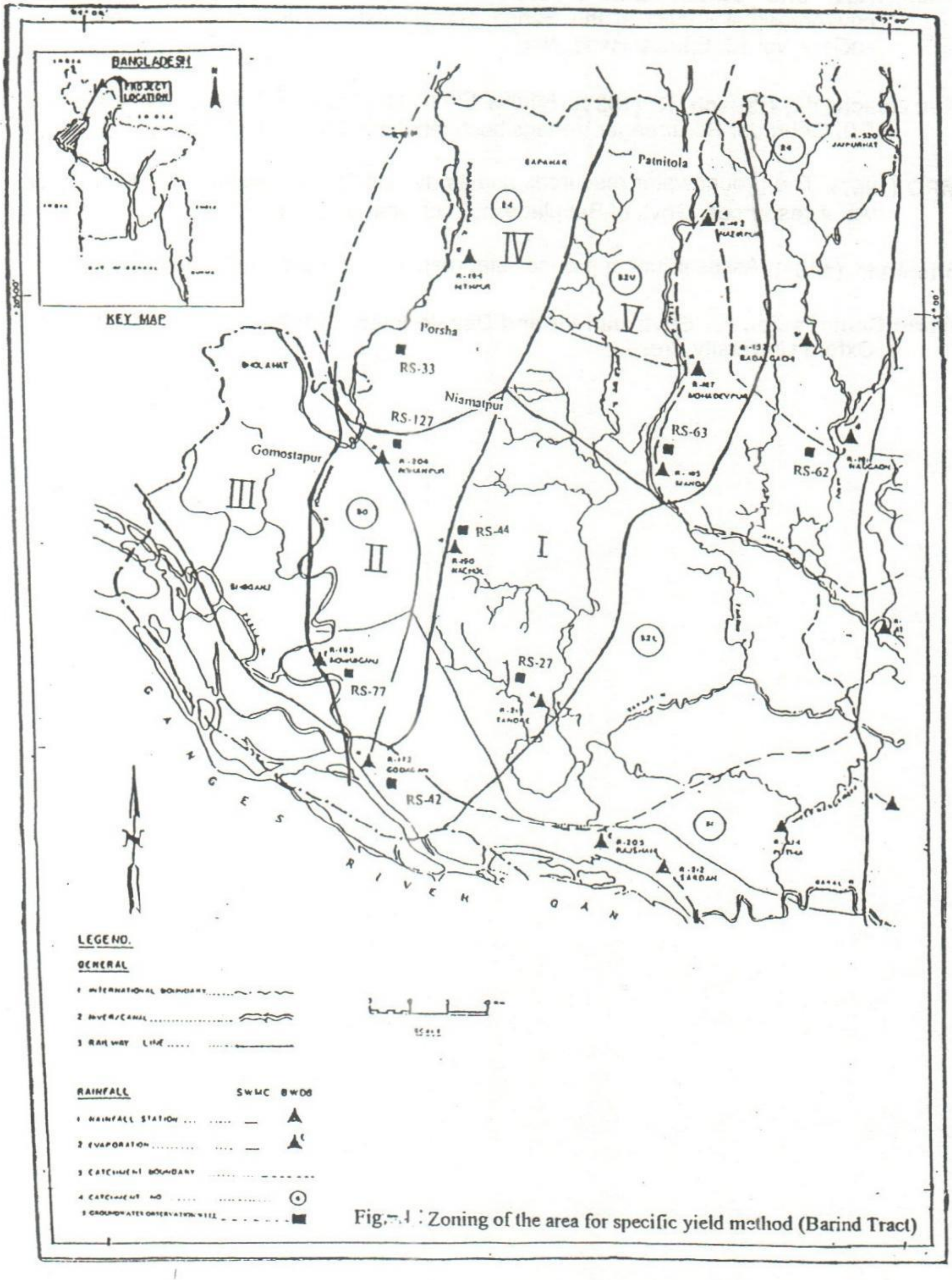


Fig. 4 : Zoning of the area for specific yield method (Barind Tract)

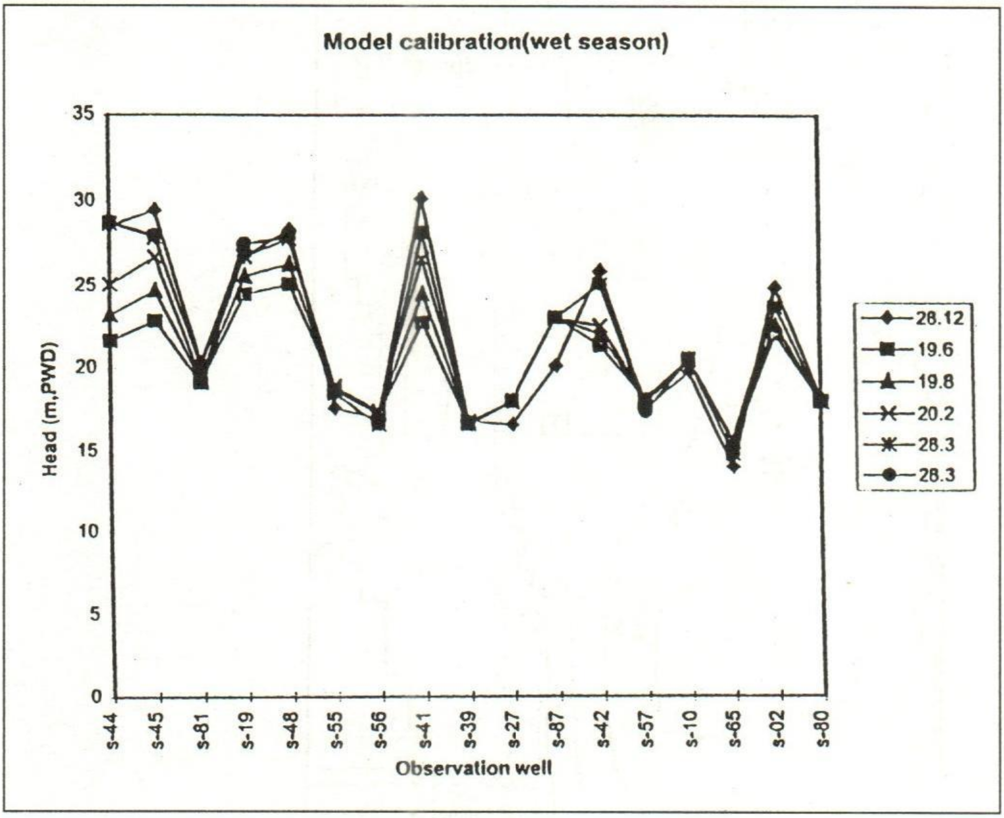
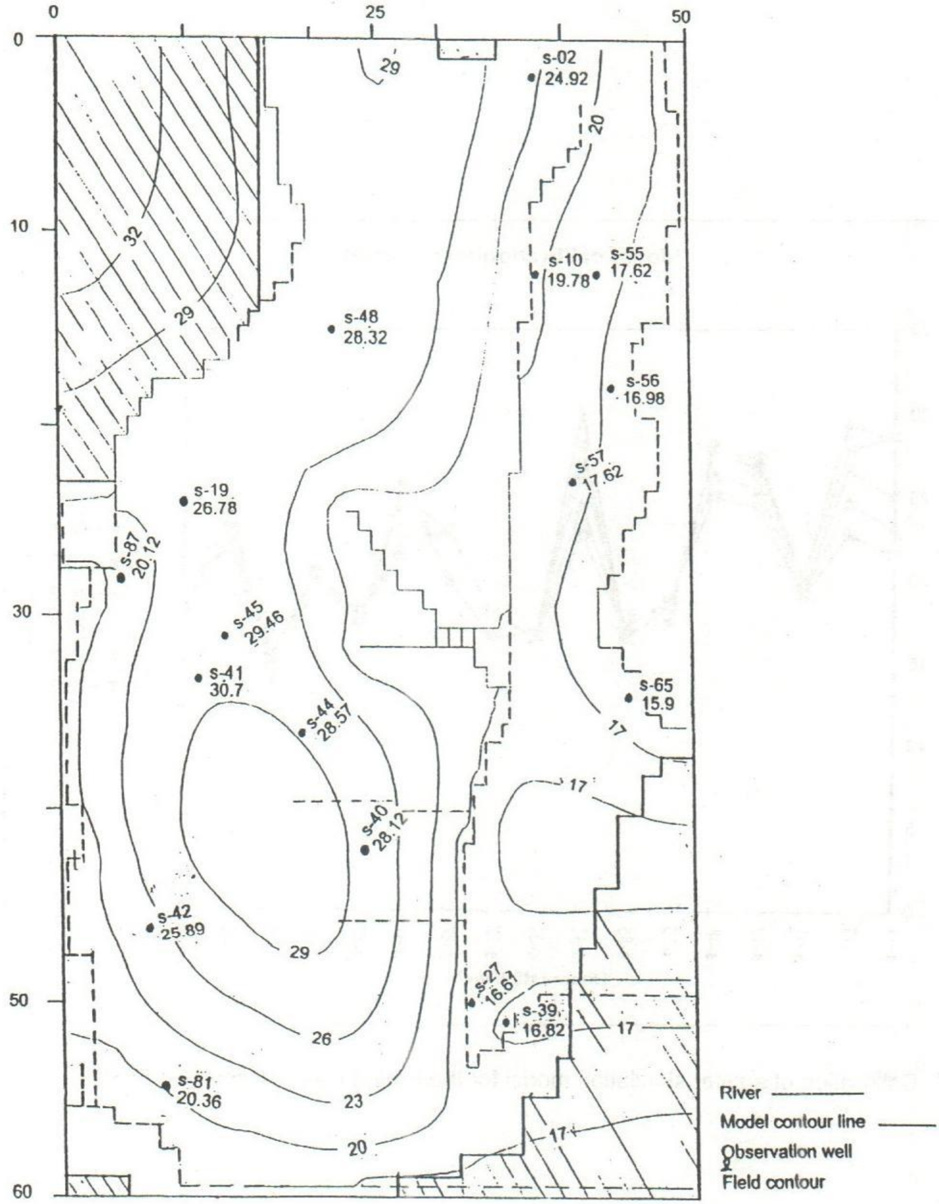


Fig. 2 · Calibration of aquifer simulation model for the Barind (wet season, Sept./1981)



File: BSP812.1 Heads at steady state

h-Min = 14 m, h-Max = 32 m, Delta-h = 3 m, Scale 1 : 600000

Fig.- 3 : Groundwater contour lines found in the model and observed heads in wet season (calibration)

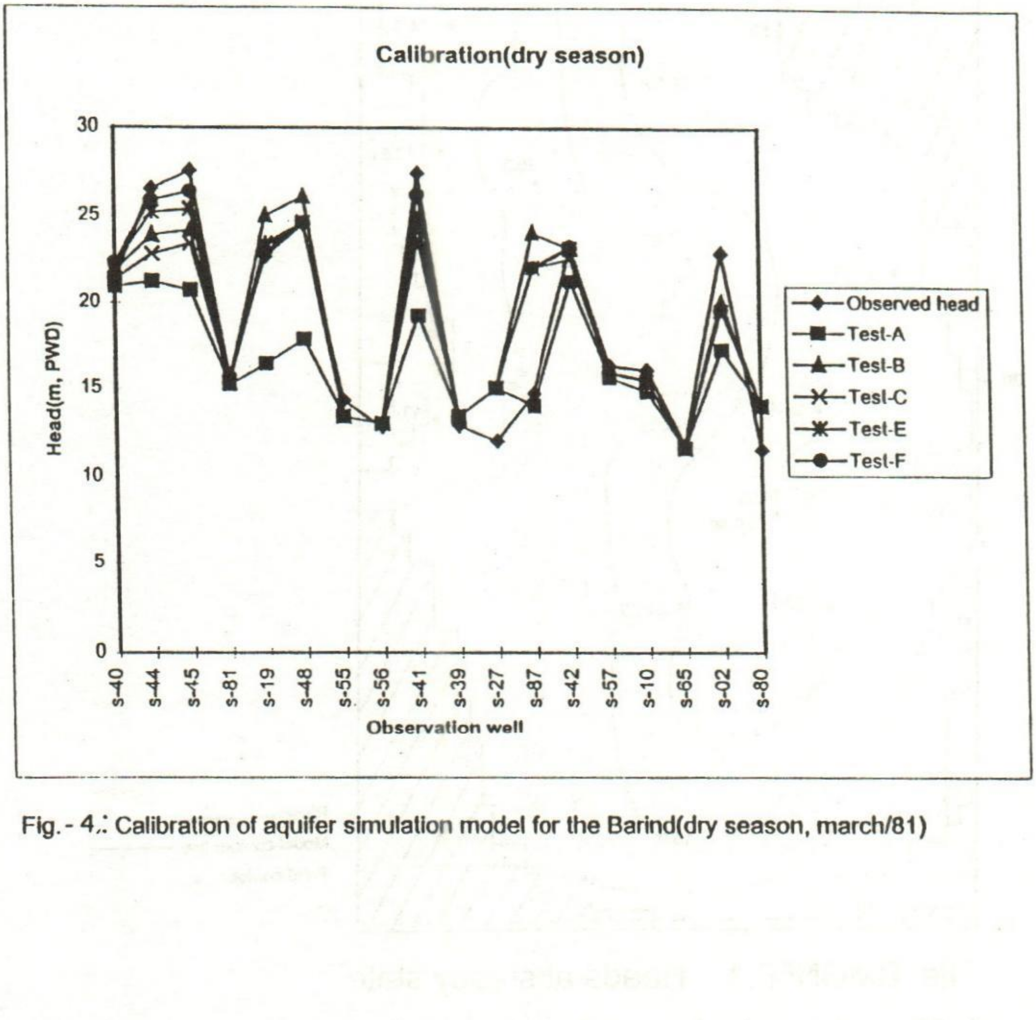
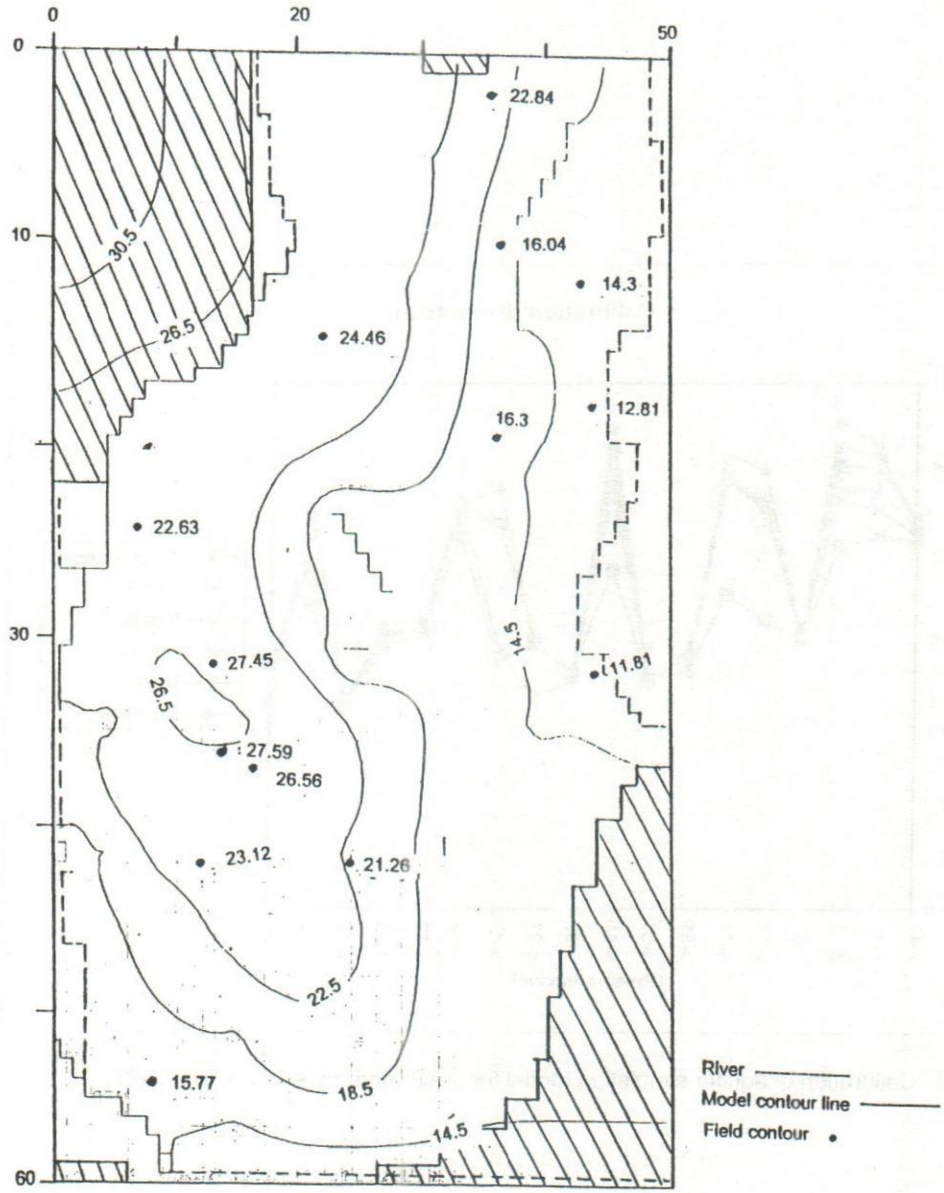


Fig.- 4.: Calibration of aquifer simulation model for the Barind(dry season, march/81)



File: BMGN5E.1 Heads at steady state

h-Min = 10 m, h-Max = 31.914 m, Delta-h = 4 m, Scale 1 : 6000

Fig.- 5 : Groundwater contour lines found in the model and observed heads in dry season (calibration)

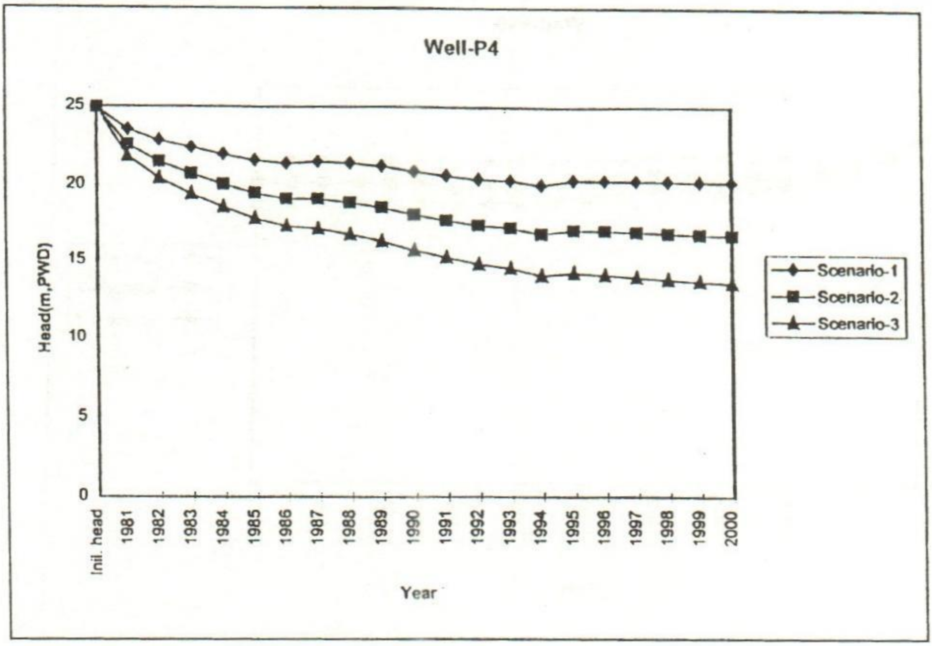


Fig.- 6 : Heads found (m,PWD) in pumping well at the end of pumping

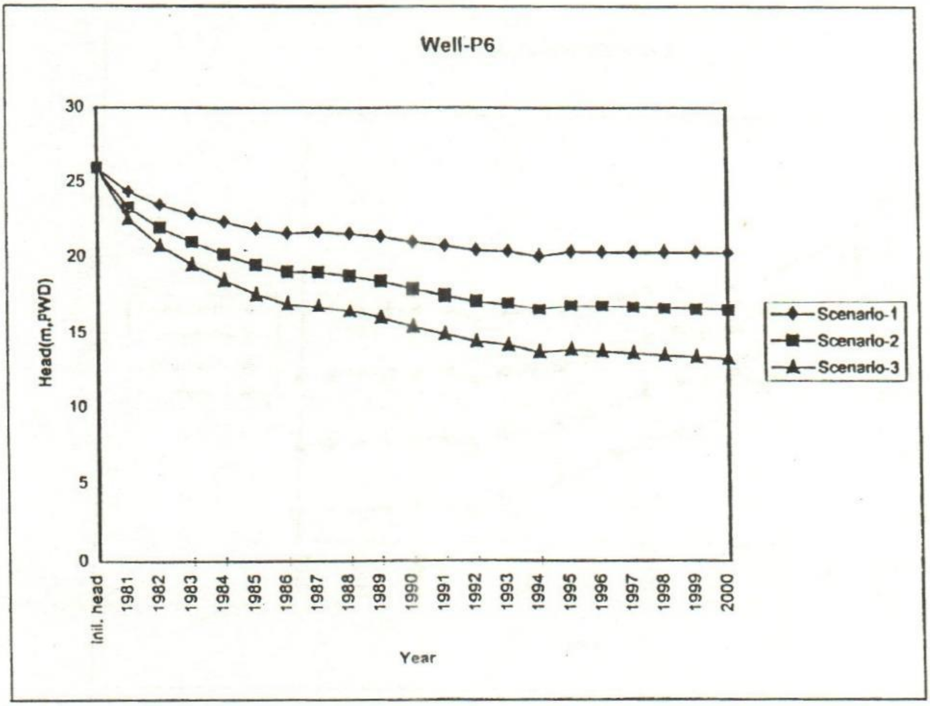


Fig.- 7 : Heads found (m,PWD) in pumping well at the end of pumping

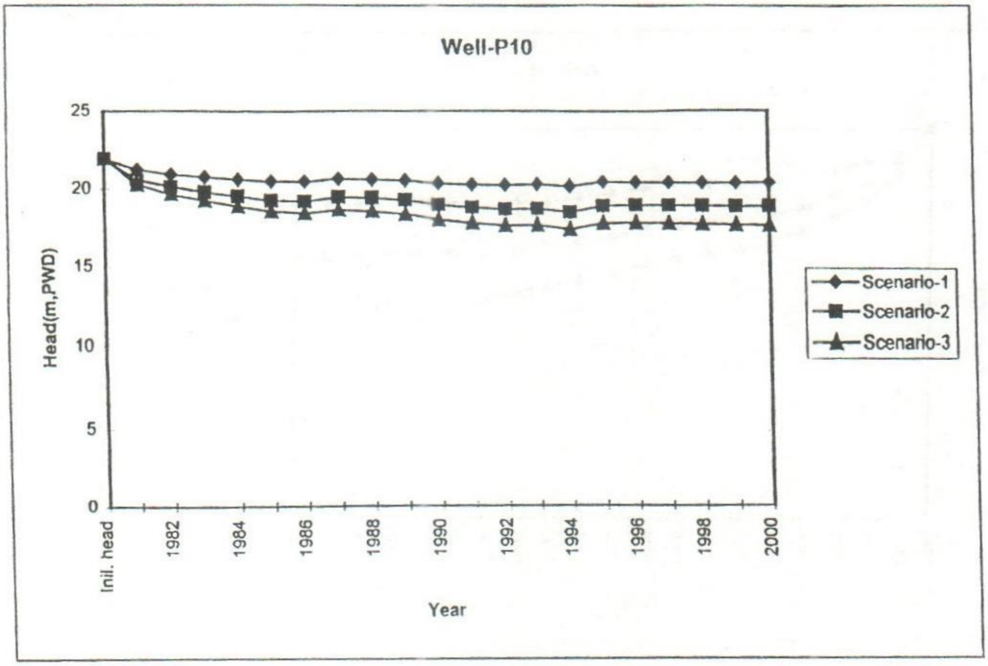


Fig.- 8 : Heads found (m,PWD) in pumping well at the end of pumping

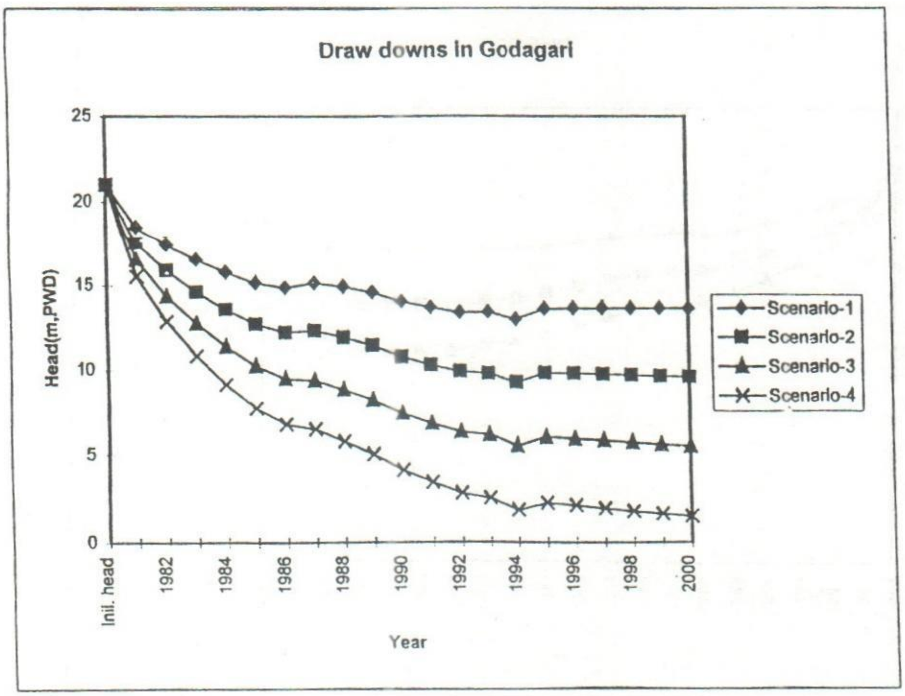


Fig. 9 : Heads found (m,PWD) in pumping well at the end of pumping (well-G1)

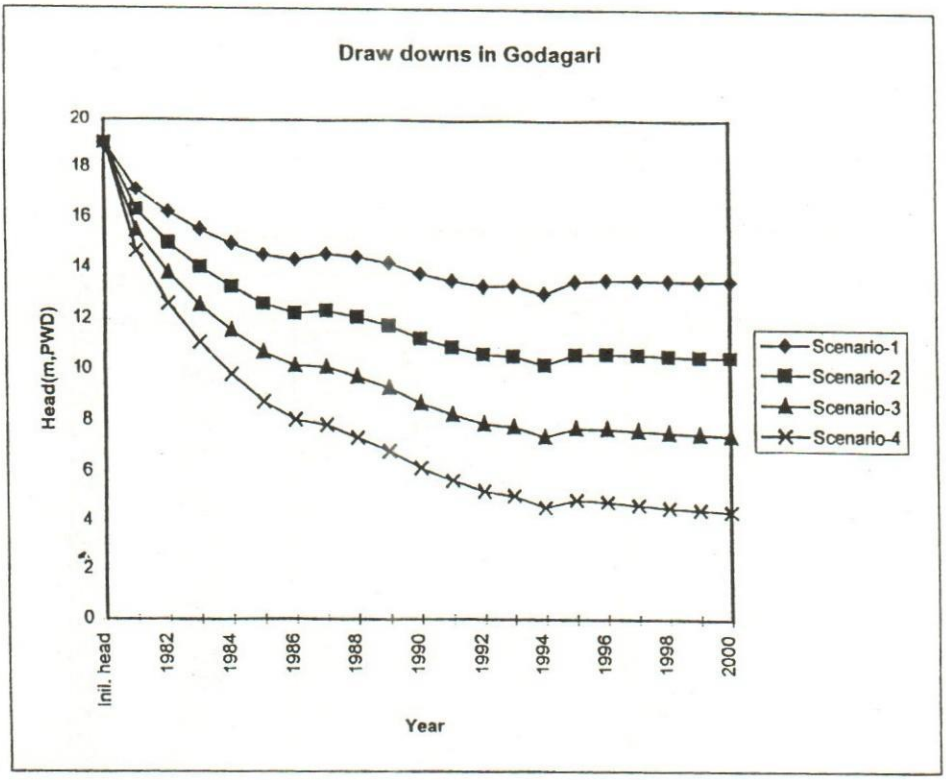


Fig.-10: Heads found (m,PWD) in pumping well at the end of pumping(G5)

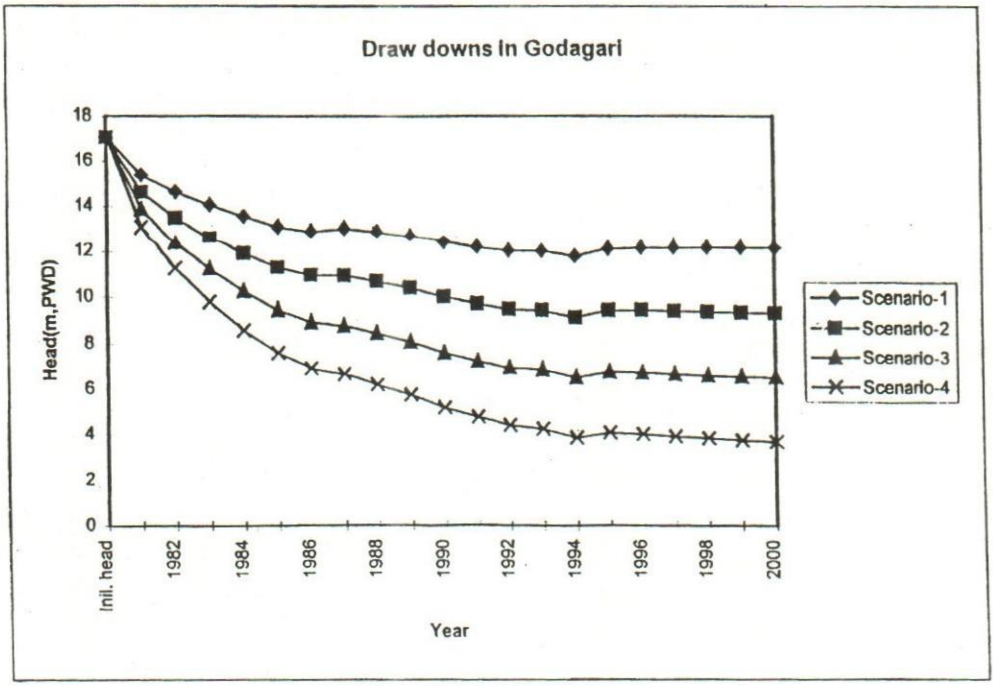


Table I : Deep and shallow tube wells in different years in different thanas

Thana	1981		1986		1989		1995	
	Deep	Shallow	Deep	Shallow	Deep	Shallow	Deep	Shallow
Sapahar	0	22	5	216	85	157	103	330
Badalgachi	0	0	0	0	102	869	176	1720
Niamatpur	2	82	28	367	146	128	155	500
Nawabgong	0	94	32	578	121	263	225	795
Manda	40	388	78	1059	193	688	364	1085
Bholahat	0	28	29	95	132	32	94	113
Dhamoirhat	9	102	10	295	198	624	197	1589
Godagari	14	73	60	531	193	642	189	1004
Tanore	33	91	52	511	186	225	223	392
Porsha	0	22	5	216	79	182	131	253
Shibgonj	17	35	72	361	219	221	231	649
Gomostapur	0	52	14	430	122	414	296	530
Patnitola	2	166	50	537	200	696	136	1993
Mohadevpur.	18	247	108	606	308	765	290	1871
Nachole	0	20	4	185	104	197	114	179
Total	135	1422	547	5987	2388	6103	2924	13003

Table 2 : Transmissivity and storage coefficients in different thanas within the Barind Tract (15 thanas included).

Thana	Average Transmiss (m^2/s)	Average st. co-efficient
sapahar	0.012	0.1
Badalgachi	0.011	0.08
Niamatpur	0.00578	0.05
Nawabganj	0.0109	0.13
Manda	0.0069	0.1
Bholahat	0.0115.	0.11
Dhamairhat	0.00925	0.09
Godagari	0.0104	0.075
Tanore	0.0081	0.085
Shibganj	0.0156	0.125
Porsha	0.0069	0.06
Gomostapur	0.0075	0.08
Patnitola	0.00839	0.075
Mohadevpur	0.00868	0.075
Nachol	0.0046	0.09
	Av 0.0087	Av 0.0883

Table 3 : Calibration of aquifer simulation model for the Barind (wet season, Sept./81)

	Obs.head	Test-1	Test-2	Test-3	Test-4	Test-5	Location
s-40	28.12	19.6	19.8	20.2	28.3	28.3	(24,42)
s-44	28.57	21.6	23.2	25.1	28.7	28.8	(19,36)
s-45	29.46	22.8	24.7	26.7	27.9	28	(13,31)
s-81	20.36	19.1	19.2	19.3	20.1	20.1	(8,55)
s-19	26.78	24.5	25.6	26.8	26.7	27.5	(10,24)
s-48	28.32	25.1	26.3	27.8	27.8	27.9	(22,15)
s-55	17.62	18.7	18.7	18.9	18.5	18.5	(43,12)
s-56	16.98	17.2	17.2	17.4	16.6	16.6	(44,18)
s-41	30.17	22.7	24.5	26.6	28.1	28.2	(11,33)
s-39	16.82	16.6	16.7	16.8	16.8	16.8	(35,51)
s-27	16.61	18	18	18	18	18	(32,50)
s-87	20.12	23	23	23	23	23	(5,28)
s-42	25.89	21.3	22	22.5	25.1	25.1	(7,46)
s-57	17.62	18.1	18.1	18.2	17.9	17.4	(41,23)
s-10	19.78	20.5	20.5	20.5	20.5	20.5	(38,12)
s-65	13.98	15.5	15.5	15.7	14.8	14.8	(45,34)
s-02	24.92	22	22.6	23.7	23.7	23.7	(38,2)
s-80	18.23	18	18	18	18	18	(6,57)

Table-4 : Calibration of aquifer simulation model for the Barind(dry season, march/81)

	Observed head	Test-A	Test-B	Test-C	Test-E	Test-F
s-40	21.26	20.9	21.8	21.3	22.1	22.3
s-44	26.56	21.2	23.9	22.8	25.2	25.9
s-45	27.59	20.7	24.2	23.4	25.4	26.4
s-81	15.77	15.2	15.3	15.3	15.3	15.3
s-19	22.63	16.4	25	23.2	23.4	22.9
s-48	24.46	17.8	26.1	24.6	24.6	24.6
s-55	14.3	13.4	13.4	13.4	13.4	13.4
s-56	12.81	13	13	13	13	13
s-41	27.45	19.2	24.4	23.5	24.9	26.2
s-39	12.88	13.4	13.4	13.4	13.4	13.4
s-27	12.01	15	15	15	15	15
s-87	14.71	14	24	22	22	22
s-42	23.12	21.2	23.1	22.5	23.1	23.2
s-57	16.3	15.6	15.9	15.9	15.9	15.9
s-10	16.04	14.8	15.5	15.4	15.4	15.4
s-65	11.81	11.6	11.6	11.6	11.6	11.6
s-02	22.84	17.2	20	19.5	19.5	19.5
s-80	11.52	14	14	14	14	14

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EVALUATION OF SOME SCOUR FORMULAE FOR PIERS AND GROYNES

Syed Md. Anwaruzzaman[1], Dr. M. Monowar Hossain[2]

Abstract

This study was under taken to obtain field data of scour depth of different piers of East-West interconnector, Hardinge Bridge, model test data for groynes and to compare it with the potential predictors of scour depth.

Empirical formulae of Inglis (1949), Blench (1962), Ahmed (1962) and Shen et al (1969) were used to predict the scour depth using the field data of piers of East-West Interconnector and Hardinge Bridge. The study revealed that scour depth prediction by Blench formula indicated better correlation compared to other formulae. Empirical formulae of Ahmed (1953), Liu et. al (1961) and Breuser (1991) were used to predict the scour depth using the model test data of groynes. Scour prediction by Breuser formula indicated better correlation compared to other formulae in this case.

The study reveals that depending on the situation, the prediction capability of a particular formula could differ.

Introduction

Scour refers to the removal of material by running water. This usually occurs when the velocity of flow in the waterway exceeds the velocity which will cause the bed material to move. Scour may occur progressively over the life of a structure or it may occur very quickly as a result of some relatively rare high flow. Many watercourses experience erosion and deposition and consequent changing of bed levels with time as part of their natural life cycle. The presence of a bridge can results in additional lowering of their bed at the bridge site, however, and it is this reduction in bed level which is generally meant by bridge scour. If the bridge has one or more piers sited within the waterway, or if the water level reaches one or both abutments, the structure then interacts with the river. The first effect will be a reduction in the natural breadth of the river channel at the bridge site. For a given flow, this reduction in breadth will result in a rise in water level upstream of the bridge and an increase in the mean flow velocity through the opening. The types of scour which may occur at a bridge site could be grouped as follows:

- General scour of the stream which would occur irrespective of whether the bridge was there or not.
- Localized scour (or constriction scour) which may occur because of the constriction of the waterway and rechannelling of berm flow by the bridge.

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The environment in the surrounding area is natural and fauna is better condition in the surrounding area and there has been no adverse effect due to presence of JFCL.

Living condition of nearby villagers have improved from the past due to JFCL. Agricultural production has increased and people have been enthusiastically participating in rural development and welfare activities like tree plantation, blood donation, educational activities etc.

In short JFCL has been maintaining a high standard in environment protection and pollution control.

Acknowledgement

The authors wish to thank Md. Tariquzzaman, Ex-thana Engineer, Local Government Engineering Department, Sarishabari, Jamalpur, Bangladesh for giving the related data of JFCL. The authors are also grateful to the authority of the Bangladesh Water Development Board (BWDB) for supplying water level data.

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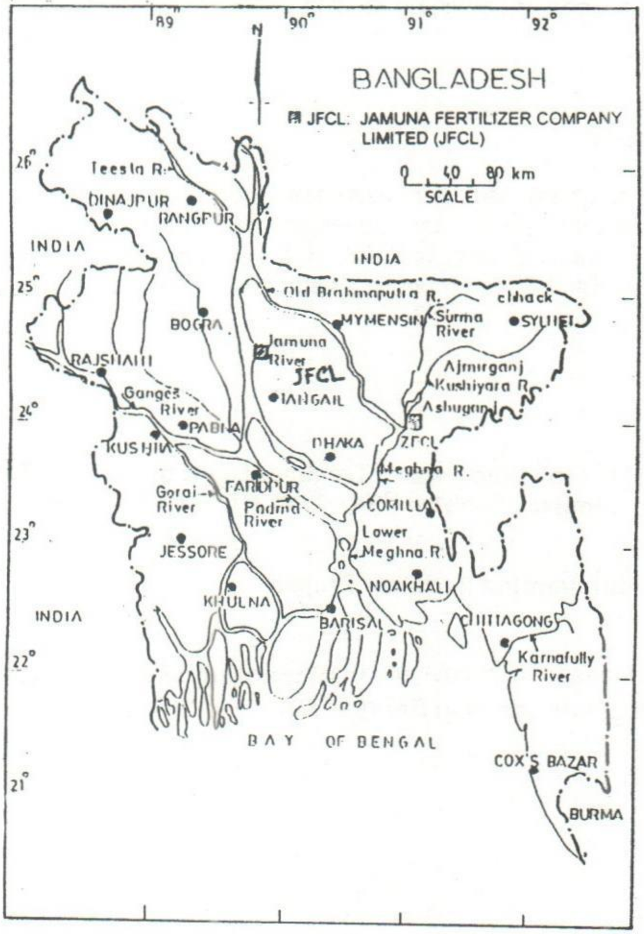

Figure 1: Location of the Jamuna Fertilizer Company Limited (JFCL)

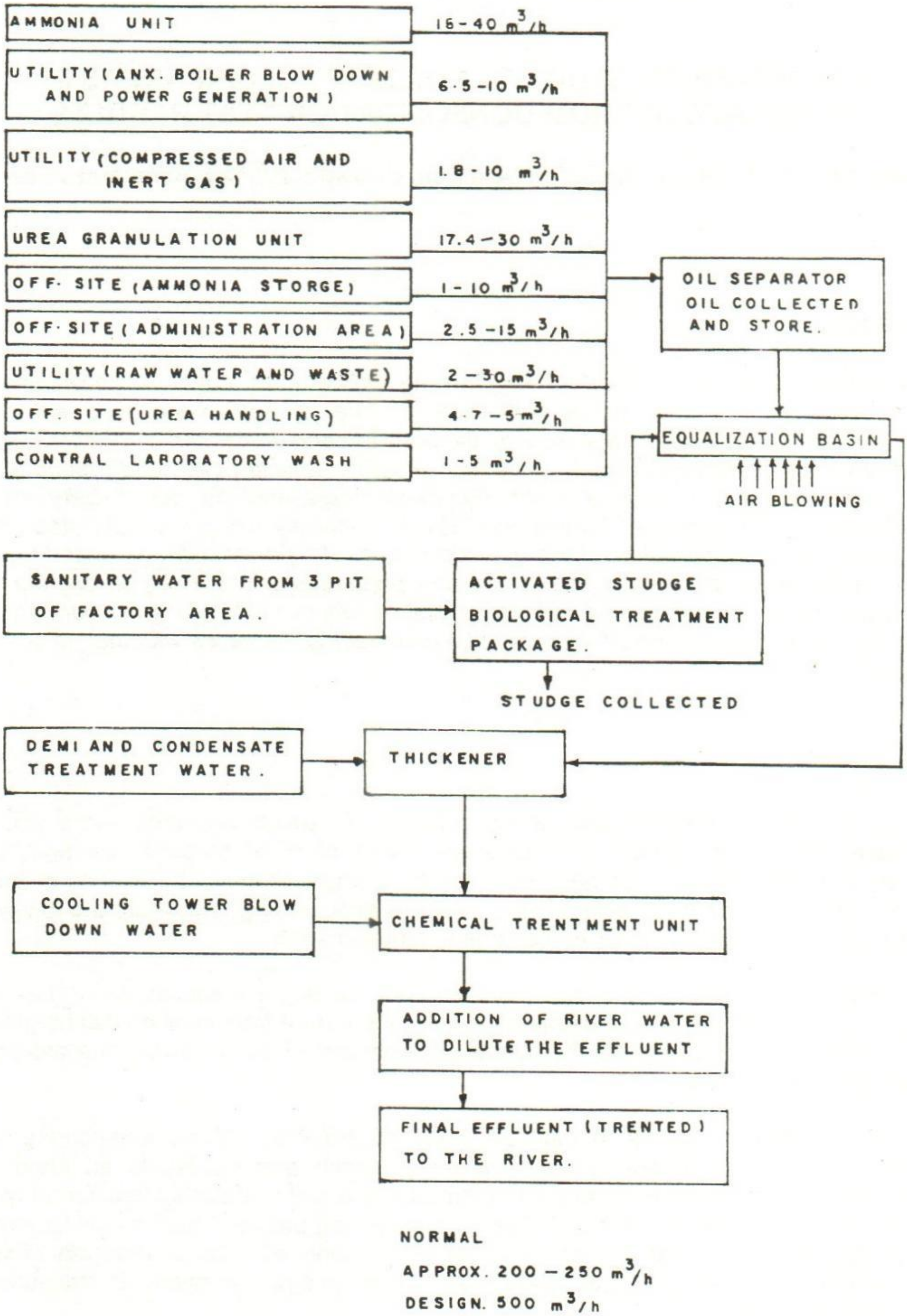

FIG. 2: EFFLUENT TREATMENT SYSTEM IN JFCL.

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PERMEABILITY STUDY OF SOILS OF POLDER AREAS IN BANGLADESH FROM CONSOLIDATION TEST RESULTS

Uma Saha [1], Md. Nurul Haque[2] Sheela Rani Chowdhury[1], Md. Hanif Mazumder[3]

Abstract

Permeability is a very important engineering property of soils. A knowledge of permeability is essential for determination of rate of settlement of a saturated compressible soil layer, seepage through the body of earth dams, and stability of slopes, uplift pressure under hydraulic structure & their safety against piping and ground water flow towards wells and drainage of soil. This paper represents the permeability of soils for different polder areas of Bangladesh. The permeability has been calculated from the consolidation test results of soils of polder areas. The permeability is related to void ratio, particle size, unit weight and many other properties of soil. Here an attempt has also been made to established the relationship between void ratio and permeability only.The key findings of this study is that the permeability increases with the increase of void ratio.

Introduction

Bangladesh is a riverine country which consists of several hundred rivers and its tributaries and distributaries. The extensive flood plain of these rivers and their numerous tributaries and distributaries is the main physiographic phenomena of these alluvial rivers. So, the land reclamation process is encouraging to develop the low-lying areas (polders) by constructing embankment, regulators etc.

The embankments around the polders prevent tidal flooding and salt intrusion. Drainage sluices constructed in the embankments to drain the run-off from local rainfall by gravity on surrounding tidal rivers. So, for overall development of polder areas, the aforesaid projects are undertaken.

For the construction works in different types of projects ,various engineering and physical parameters of soil is determined of which permeability is an important parameter .Therefore the knowledge of permeability of soil is essential in different types of constructions to get an insight of various engineering problem such as settlement of buildings, seepage through and below the earth structures etc. The permeability of soils also contributes in designing the filters used to prevent piping in hydraulic structures. (K.R. ARORA, 1992)

Permeability of soils is directly measured by two methods namely, (1) constant head method & (2) falling head method and indirect methods for the assessment of permeability by means of calculations based on other soil properties (Head, 1982).

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In the indirect method, the permeability of the soil is determined from consolidation test data, which is extremely useful for forecasting the magnitude and time of the settlement of the structure. The consolidation test, which is very important for all the foundation structure (ARORA, 1992).

Permeability test as well as the consolidation test of cohesive soil is time consuming, and as the consolidation test is to be conducted so in this study, the investigation on permeability has been carried out from the consolidation test data. The results of permeability tests are very important for the respective polder areas. The locations of polder areas have been shown in Fig. 1. The data and results of consolidation test have been collected from soil testing report of River Research Institute (RRI) of different years. Since the properties like void ratio affect the permeability (Murthy, 1993) and it is an important factor of consolidation test that's why equations have been established for different zones with void ratio and permeability which may help the construction works in the respective polder areas.

Literature Review

Geological background and soils of polder areas

Bangladesh forms a major portion of Bengal basin. Being located close to one of the world's major subjection fault in the north and a major transform fault in the east, the Bengal basin and its adjacent area from one of the most active tectonic regions of the world. Structural activity, primary faulting has significantly influenced the quaternary geology (Morgan, 1954).

The landscape of Bangladesh is mainly of a monotonous flat plain. From geological point of view, Bangladesh is located at the eastern part of Bengal basin which is an extremely flat delta consists primarily of a large alluvial basin floored with sediments deposited by three major rivers, the Ganges-Padma, the Brahmaputra-Jamuna and the Meghna and their tributaries and distributaries. Most of the part has been subsiding slowly due to tectonic forces responsible for building up the Himalayas and other hilly areas. Due to the iso-static balance of the earth surface some regions have been uplifted e.g. Barind tract and some regions have a distinct trend to subside on the other hand e.g. Sylhet haor areas, Chalan beel, Faridpur beel etc.

The stratification of materials around the site of polder areas consists predominantly of silty-clay and clayey silt mixed with varying amount of fine sand. From engineering point of view, the classification ingredients or constituents of soil is of utmost importance. Because different types of soil contain different ingredients in varying amount for which its properties varies (Jahan, 1996).

Simplified soil types of polder areas of Bangladesh with their general characteristics are given below.

Soils Characteristics of the polder areas of Bangladesh (Alam, M.K., 1990)

Types of soil	**General Characteristics**
Deltaic deposits	Tidal deltaic deposit-Light to greenish-grey, weathering to yellowish grey, silt to clayey silt with lenses of very fine to fine sand along active and abandoned stream channels, including crevasse splays. Contain some brackish-water deposits. Numerous tidal creeks crisscross the area ; large tracts are submerged during spring tides.
Paludal deposits	Marsh clay and peat- Grey or bluish- grey clay, black herbaceous peat, and yellowish - silt. Alternating beds of peat and peaty clay common in bils and large structurally controlled depressions ; Peat is thickest in deeper parts. Thin beds of peat and clay are interbedded with alluvial silt in the north-central Sylhet depression. Chains of linear lakes north of the Ganges River and south of the Shillong Plateau in the Sylhet depression suggest these areas are subsiding.

Permeability depends on soil types

The permeability of soil depends on the size of the constituents of the soil. In 1967, Terzaghi & Peck (After Casagrande and Fadum, 1940) developed a chart for the permeability of soil depending on the category of soil as shown in Table 1 (Terzaghi, 1967)

Table 1: The permeability k in cm/sec (log scale) for different category of soil.

<table>
<tr><th></th><th>10^{2}</th><th>10^{1}</th><th>1.0</th><th>10^{-1}</th><th>10^{-2}</th><th>10^{-3}</th><th>10^{-4}</th><th>10^{-5}</th><th>10^{-6}</th><th>10^{-7}</th><th>10^{-8}</th><th>10^{-9}</th></tr>
<tr><td>Drainage condition</td><td colspan="6">Good</td><td colspan="3">Poor</td><td colspan="3">Practically Impervious</td></tr>
<tr><td rowspan="2">Soil types</td><td colspan="3" rowspan="2">Clean gravel</td><td colspan="3">Clean sands, clean sand and gravel mixtures</td><td colspan="4">Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.</td><td colspan="2" rowspan="2">"Impervious" sols, e.g., homogeneous clays below zone of weathering</td></tr>
<tr><td colspan="2"></td><td colspan="5">" Impervious" soils modified by effects of vegetation and weathering</td></tr>
</table>

Methodology

There are many methods for computing the permeability. These are constant head method, falling head method and indirect method from consolidation test data by calculation. In this study the permeability has been calculated from the consolidation test data using the formulae given below

$$k = (c_v a_v \gamma_w)/(1+e)$$ -------------------(ARORA, 1992)

Where, k = permeability cm/sec
Cv = Co-efficient of consolidation cm^2 /sec
e = void ratio
γ_w = the unit weight of water kg/m^3

and $a_v = (0.435\ Cc)$

in which, Cc = compression index
p = pressure for the increment of load in kg /cm^2

In this connection, co-efficient of consolidation, compression index, void ratio and pressure for the increment of load have been taken from consolidation test and using these parameters from above equation the permeability has been calculated.The void ratio e of the samples have been calculated using the formulae

$$e = (h-h_s) / h_s$$

Where, h = initial height of the sample
h_s = solid height of the sample

Laboratory study and presentation of results

A large number of soil samples have been tested in consolidation test apparatus in Soil Mechanics Laboratory of River Research Institute which were collected from different polder areas of Bangladesh.The polder areas from where the samples were collected have been shown in **Fig. 1**.The soil types and the range of some soil parameters of polder areas have been shown in tabular form in **Table 2.** The results of permeability k vs.void ratio e has been plotted graphically and shown in **Fig. 2.** The correlation between permeability k and void ratio e of different polder areas obtained from this study has been shown in tabular form in **Table 3**.

Discussions

It is seen clearly from the **Table 2** that the soils of different polder areas consist of silt or clay and occasionally it is silt or clay with trace organic matter. The consistency of the soil varies from very soft to soft. The permeability range of the soil of most of the polder areas varies between 10^{-3} cm/sec to 10^{-5} cm/sec but in some areas its range is between 10^{-3} cm/sec to 10^{-6} cm/sec . So comparing permeability values presented in **Table 2** to the values of **Table 1** it is clear that the drainage characteristics of the soils of different polder areas varies in between good to poor. From the graphical representation of permeability with the void ratio as shown in **Fig 2** that the permeability increases with increase of void ratio for the soils of all polder areas. It is also observed from the graph that for the same void ratio the permeability value is different for soils of different polder areas. This is due to the fact that soil ingredient of different polder areas are not same.

Table 3: Correlation equations of the permeability k and void ratio e established from the study of soils of different polder areas

Name of the polder	Established equation	Correlation coefficient, R
Khulna	k=.0065e-0.0047	.808
Patuakhali	k=.0554e-.044	.672
Borguna	k=.0682e-.0545	.616
Cox'sbazar	k=.0007e-.0004	.700

Conclusion and recommendation

The soil parameter permeability and void ratio are most important factors. These are very essential for determination of rate of settlement of soil layer, seepage through the earth dams, uplift pressure under the hydraulic structure, ground water flow towards the wells and drainage condition of soil. The permeability values obtained by investigating soils of polder areas may help in engineering works mentioned above . From the equations, established by studying the soils of different polder areas it is seen that permeability increases with the increase of void ratio. But the permeability varies with types of soil. For detail information about the soils of the polder areas further investigation is required.

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Table 2: Soil types and range of some soil parameters of different polder areas

Name of the polder project	Soil type	compression index, C_c	co-efficient of consolidation, C_v in cm^2/sec	void ratio, e	permeability, k in cm/sec
1.Polder No. 34/1, Khulna	Soft sandy SILT	.130-.133	$2.16\text{-}3.56 \times 10^{-2}$	.6965-.8710	$1.0 \times 10^{-4}\text{-}.5 \times 10^{-3}$
2. .Polder No. 28/2, Khulna	Soft sandy organic CLAY	.357	$.79\text{-}2.54 \times 10^{-3}$	.7188-1.169	$3 \times 10^{-5}\text{-}1.1 \times 10^{-3}$
	Very soft ORGANIC CLAY with trace organic	.510	$.81 \times 10^{-3}$	.7209-1.356	$9 \times 10^{-5}\text{-}4.2 \times 10^{-3}$
	Soft sandy CLAY with trace organic	.405-.455	$.069\text{-}1.7.9 \times 10^{-3}$	.7641-1.9155	$1 \times 10^{-6}\text{-}7 \times 10^{-3}$
3..Polder No. 34/2, Khulna	Soft sandy CLAY	.241-.392	$.73\text{-}3.33 \times 10^{-3}$	.7075-1.1208	$3 \times 10^{-5}\text{-}1.1 \times 10^{-3}$
	Soft sandy ORGANIC CLAY	.750	1.77×10^{-4}	.8904-1.992	$4 \times 10^{-6}\text{-}8.8 \times 10^{-3}$
	Soft sandy SILT	.285	$2.56\text{-}6.23 \times 10^{-2}$	.6617-.9635	$8 \times 10^{-5}\text{-}6.6 \times 10^{-3}$
	Very soft sandy CLAY	.237-.334	$.5\text{-}5.73 \times 10^{-3}$	.5988-1.3871	$3 \times 10^{-5}\text{-}5.1 \times 10^{-3}$
4.Polder No. 47/5, Patuakhali	Soft sandy CLAY	.264-.274	$2.56\text{-}4.94 \times 10^{-2}$	.6994-1.006	$2 \times 10^{-5}\text{-}8.7 \times 10^{-3}$
	Medium stiff sandy CLAY	.200-.296	$2.35\text{-}6.04 \times 10^{-2}$	.6613-1.2708	$3 \times 10^{-5}\text{-}1.7 \times 10^{-3}$
	Soft sandy CLAY	.150	$2.48\text{-}4.81 \times 10^{-2}$	1.1207-1.2436	$1 \times 10^{-3}\text{-}4.8 \times 10^{-2}$
5.Polder No. 55/1, Pataukhali	Very soft sandy CLAY	.290	$4.22\text{-}5.71 \times 10^{-3}$	.7660-1.0065	$3 \times 10^{-4}\text{-}2.1 \times 10^{-2}$
6. Polder No. 55/2D, Patuakhali	Medium stiff sandy CLAY	.160	$1.26\text{-}6.57 \times 10^{-3}$	.8125-.9838	$4 \times 10^{-4}\text{-}4.2 \times 10^{-3}$
	Medium stiff sandy SILT	.215-.310	$2.45\text{-}5.82 \times 10^{-2}$	.7568-1.0706	$1.5 \times 10^{-3}\text{-}1.8 \times 10^{-2}$
7.Polder No.55/3, Patuakhali	Soft sandy SILT	.205-.360	$2.46\text{-}5.29 \times 10^{-2}$	.7007-1.1651	$2.3 \times 10^{-5}\text{-}5.8 \times 10^{-2}$
	Medium stiff sandy SILT	.200	$2.87\text{-}4.18 \times 10^{-2}$	.8934-1.0746	$1.7 \times 10^{-4}\text{-}1.9 \times 10^{-4}$
8. Polder No.47/4, Patuakhali	Soft sandy CLAY	.334	$.95\text{-}2.25 \times 10^{-2}$	.7475-1.0781	$2.4 \times 10^{-3}\text{-}4\text{-}7.5 \times 10^{-3}$
	Soft sandy SILT	.205-.313	$1.02\text{-}8.77 \times 10^{-3}$	.8153-1.1675	$5.9 \times 10^{-5}\text{-}4.5 \times 10^{-2}$
10. Polder No.39/1,(B&D), Borguna	Soft sandy CLAY	.280	$3.73\text{-}6.28 \times 10^{-2}$	.8153-1.1675	$5.9 \times 10^{-5}\text{-}4.5 \times 10^{-2}$
	Stiff sandy CLAY	.250	$2.86\text{-}7.95 \times 10^{-3}$	.6640-.8950	$6.3 \times 10^{-3}\text{-}9.8 \times 10^{-4}$

(Continued)

Name of the polder project	Soil type	compression index, C_c	co-efficient of consolidation, C_v in cm^2/sec	void ratio, e	permeability, k in cm/sec
11. Polder No. 41/7A, Borguna	Soft sandy and clayey SILT	.245-.350	$2.84\text{-}6.25 \times 10^{-2}$	.7632-1.1835	$2.6 \times 10^{-4}\text{-}2.2 \times 10^{-2}$
	Soft sandy CLAY	0.210	$1.52\text{-}2.51 \times 10^{-3}$	0.6964-.9173	$3.5 \times 10^{-5}\text{-}9.3 \times 10^{-4}$
	Soft sandy SILT	0.205	$2.86\text{-}7.43 \times 10^{-3}$	.7986-.9778	$5.6 \times 10^{-5}\text{-}2.0 \times 10^{-3}$
12. Polder No. 39/2A, Borguna	Very soft sandy SILT	.192-.287	$.405\text{-}8.26 \times 10^{-2}$	.8507-1.1816	3.7×10^{-6} 6.6×10^{-4}
	Very soft sandy CLAY	.310	$2.22\text{-}6.33 \times 10^{-3}$	0.8999-1.222	2.1×10^{-4}
13. Polder No. 39/1 & 41/7B, Borguna.	Medium stiff sandy CLAY	.277	$1.89\text{-}2.68 \times 10^{-3}$	0.6219-.9061	$2.5 \times 10^{-5}\text{-}7.4 \times 10^{-4}$
	Soft sandy CLAY	.213-330	$.975\text{-}7.61 \times 10^{-3}$	.7220-1.1227	$1.5 \times 10^{-4}\text{-}2.7 \times 10^{-3}$
	Very soft sandy SILT	.258	$3.96\text{-}13.06\text{-}10^{-3}$	.7580-1.097	$1.3 \times 10^{-5}\text{-}3.3 \times 10^{-4}$
14. Polder No. 39/2A, Borguna.	Soft sandy SILT	.210	$.85\text{-}2.612 \times 10^{-3}$	.6587-1.1758	$6 \times 10^{-5}\text{-}6.8 \times 10^{-4}$
		.330	$3.03\text{-}9.36 \times 10^{-3}$	.6362-1.1321	$6 \times 10^{-6}\text{-}4.2 \times 10^{-3}$
	Stiff sandy SILT	.215	$4.17\text{-}5.76 \times 10^{-3}$	.6362-.8233	$3.5 \times 10^{-5}\text{-}1.8 \times 10^{-3}$
	Soft sandy CLAY	.185-.295	$1.34\text{-}9.75 \times 10^{-3}$	.6606-9893	$4 \times 10^{-5}\text{-}1.1 \times 10^{-3}$
15. Polder No. 66/2 Cox's bazar	Very soft sandy ORGANIC CLAY	.580	$.86\text{-}1.28 \times 10^{-3}$	.9773-1.552	$1.7 \times 10^{-5}\text{-}9.5 \times 10^{-4}$
	Very soft sandy CLAY	.162	$2.6\text{-}9.1 \times 10^{-4}$	.5306-.7234	$6.7 \times 10^{-6}\text{-}3.7 \times 10^{-5}$
16. Polder No. 65/A-1, Cox's bazar	Very soft sandy CLAY	.291	$3.65\text{-}8.76 \times 10^{-3}$	.7097-1.0574	$1.0 \times 10^{-4}\text{-}3.1 \times 10^{-3}$
17.Polder No. 68, Cox;s bazar	Very soft sandy CLAY	.232	$8.69\text{-}37.19 \times 10^{-4}$	.4734-.7226	$7 \times 10^{-6}\text{-}1.4 \times 10^{-4}$
18. Polder No. 67/B, Cox's bazar	Very soft sandy CLAY with trace organic	.412	$.85\text{-}1.19 \times 10^{-3}$	.7357-1.2573	$1.5\text{x}10\text{-}5\text{-}7.4 \times 10^{-4}$
	Very soft sandy CLAY	.305	$1.03\text{-}2.25 \times 10^{-3}$	.6333-1.003	$2.9 \times 10^{-5}\text{-}8.6 \times 10^{-4}$

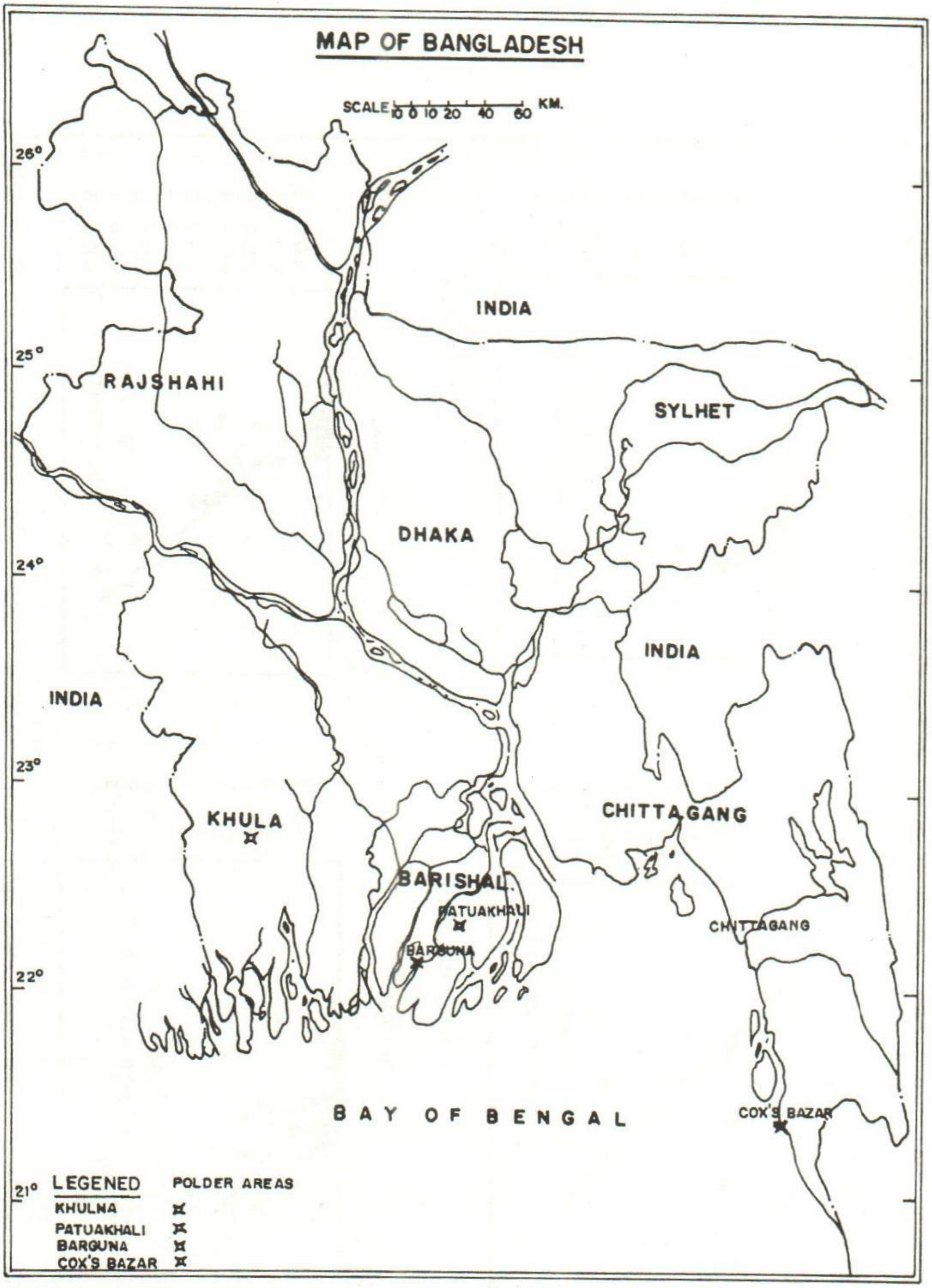

FIG. I : LOCATION OF POLDER AREAS IN BANGLADESH

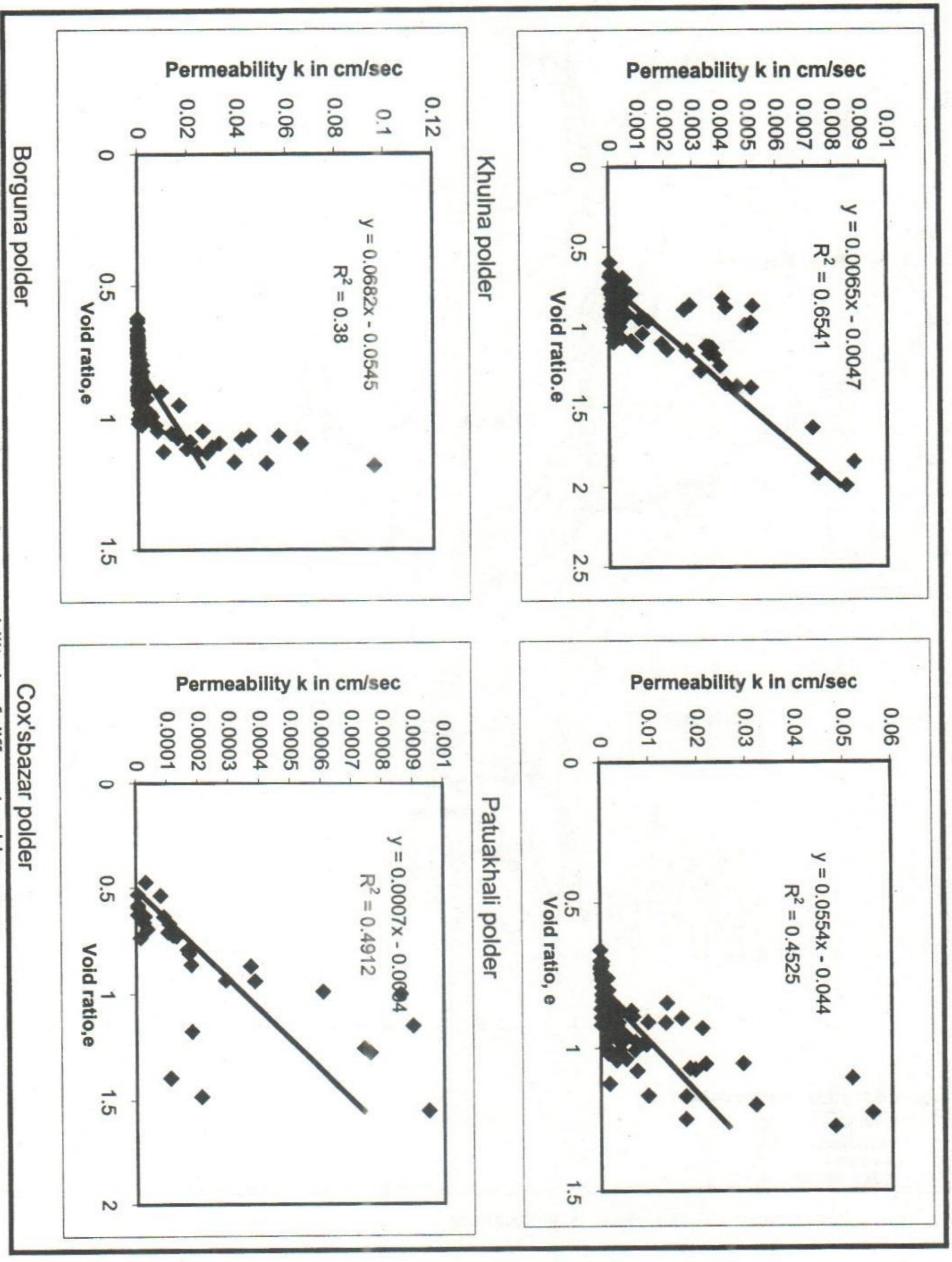

Fig 3 : Relation between void ratio e and permeability k of different polder areas

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A STUDY ON ENVIRONMENTAL CONSEQUENCES FOR A FLOOD CONTROL PROJECT IN BANGLADESH

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Abstract

The Chalan Beel Polder D is a flood control project situated in the northwestern Bangladesh. It is one of the four water resources development projects in the general Chalan Beel Area. The Polder D is enclosed by the river Atrai, and its distributaries Fakirni on the eastern side (Ref. Fig. 01). The total project area is 53,055 ha of a net cultivable area is 37,235 ha. Bangladesh Water Development Board (BWDB) initiated the Chalan Beel Polder D to provide drainage and flood control facilities with limited irrigation empoldering the project area with a view to achieve food self sufficiency, expansion of productive employment, acceleration of economic growth and to promote self-reliance. A study was conducted to evaluate the project performance with a view to have an idea about the project impacts incurred due to intervention of the polder. Water resources development projects have several positive and negative potential impacts on the surrounding environment and vice versa. This contribution describes the physical, biological and socio-economic impacts due to this project.

Introduction

Chalan Beel Polder D is one of the four water resources development projects with flood control as a key element in the general Chalan Beel Area. The Polder D as shown in Figure-1 has been chosen as one of the sites for in depth evaluation of the environmental consequences of a flood control and drainage project under Flood Action Plan -12 (FAP-12). Agricultural, economic, engineering , fisheries and social institutional aspects have been considered in this evaluation. Issues on livestock and nutrition also have been investigated. A number of sites under the project and localities around the project were visited and discussions were held with farmers, shopkeepers, transport workers, petty traders, owners of the husking machines and laborers and also with various concerned officials for collection of information on and understanding the project area and project impacts.

Due to the implementation of the FCD project the potential positive impacts on human and physical environmental issues, in increasing land availability, monsoon season cropping and harvested monsoon season yield, and in improving communication network, were limited and have been further offset by a number of serious negative impacts, mainly associated with the impacted area with the annual public cuts and breaches.

The main negative impacts are the decrease in wetlands leading to a decline in capture fisheries, the marked deterioration in social cohesion and equity, the failure to develop any public participation in project operation and the threat to the cultural traditions of the

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largely Hindu capture fisherman. Outside the project area the project has had major negative impacts, on conditions in adjacent areas which suffer higher flood levels and downstream where the combination of the Chalan Beel D polder with other middle atrai embankment systems leads to threads of catastrophic flooding. The retreat of the wetlands has caused more significant biotic impacts than the pre project situation. Fish ecology and aquatic micro-biota in particular have suffered.

Methodology of the Study

The study was conducted using the Rapid Rural Appraisal (RRA) and Project Impact Evaluation (PIE) techniques with the help of structured questionnaires. In the following sections, at first the pre project Environmental conditions for different physical, biological and socio-economic issues have been described. Next the post project environmental impacts on various factors have been analyzed and compared to without project situation.

Project structures as implemented

The Chalan Beel Polder 'D' sub-project as implemented composed of a good number of structures. The major structures are described in the sub-sections below.

Embankment

In the original project proforma and in the Feasibility Study the total required length of the embankment was 133.63 km which was later revised in the project proforma to 132.28 km. The embankment has been found to be constructed as designed up to March 1989 except at three places in Murshidpur and Chauapara village under Bharso Union. It had been reported by the Union Parishad (U.P) Chairman of Bharso that the embankment was not constructed at these points due to land acquisition problem.

Cuts and Breaches

During the floods in 1986/87 there were all together 33 natural breaches and public cuts. Six public cuts were made at Gopalpur, Mansinghpur, Birloya, Basudebpur, chak Kesab and Balubajar and two natural breaches occurred at Sankarpur and Saildha during the flood of 1988. It is here noted that during these floods the river stage at the gauge stations Jotebazar, Noahata and Bagmara on the rivers Fakirni, Sib-Barnai and Barnai respectively exceeded the highest river stage considered in 100 years return period. Two public cuts were made in 1990 at Tangrapara (Ratandanga) and Madhupur though there was no abnormal flood in this year. Conflicting interests of fishermen and farmers and among farmers led to some such cuts.

Condition of the embankment:

Discussion with the Bangladesh Water Development Board (BWDB) Staff, interviews with local people and on-site observations indicated that about 40% of the embankment along with its length was damaged due to rain needs to be repaired/rehabilitated for resectioning.

Regulators

In the original project proforma there was a provision for 9 regulators only. Later in the revised project proforma 4 additional regulators were proposed. The ventage of the regulators varies from 1 to 8 and was constructed as designed. However, some of the regulators appeared to be under designed and therefore were found to be in defective condition. The gate operating arrangement i.e the Gear Box for some regulators needs minor repair while 3 regulators were found to be in need of protective works. Four additional flushing cum drainage regulators are proposed by concerned BWDB officials.

Irrigation inlet/drainage outlet

In the third revised project proforma there was a provision of 77 nos. of irrigation inlets/drainage outlets. These were constructed as designed. All are in good condition except two which need to be reconstructed as these fell in disappear due to public cuts in the embankment.

It was understood that farmers feel the necessity of more inlets for irrigation water from the river by low lift pump (LLP). The concerned BWDB people has proposed 17 irrigation inlets in addition to the existing ones to meet the demand.

Flushing sluice/drainage outlet

In the third revised project proforma 8 nos. of flushing sluices/drainage outlets were proposed. All these structures appear to be functioning fairly. However, these do not seem to be sufficient to meet the total requirement. Accordingly 4 additional flushing sluice/drainage outlets have been proposed by the BWDB.

Drainage channel

The original project proforma (PP) provided for excavation of 193.08 km of drainage channels which was brought down to 1378.49 km in the revised PP and split into excavation of 123,58 km of existing channels to be funded under Annual Development Programme (ADP) and excavation of 10.70 km of drainage channels to be under Food For Works (FFW) programme. The 3.21 km would not be required as was reported by field offices. The drainage channels are now partly silted up and re-excavation of 22.87 km of drainage channel need to be conducted as was reported by concerned BWDB people.

Roads

In the original PP there was a provision of construction/reconstruction of 48.27 kms of main road and 96.54 kms of village road. The respective figures in the latest PP were 76.09 kms and 25.65 kms. Both the main and village roads are in good condition except in some places that there is a need for repair. An additional 53.25 km of main road (HBB) is proposed by the local people and the concerned BWDB people to improve the road communication network in the project area.

Project implementation and project costs

The construction works of the Chalan Beel Polder 'D' was officially started in 1981/82 and completed in 1988/89. All the project components were constructed during this span of time except for repair and rehabilitation due to some damages during 1987/88 to 1990/91.

The initial cost of the project as estimated in the Feasibility Report had been Tk. 2,850 lakh while the sum approved by the Government had been much lower, Tk.2,261.28 lakh. The Final Revised PP in October 1988, put the estimated cost at tk. 3,732.26 lakh of 65% more than the initially approved budget.

Pre - project environment

Physical environment

The General topography of the project area is rather flat sloping from the north west to the south-east. The micro-topography is quite complicated, however. The project area is low old or young alluvial delta with irregular relief in elevation from 10.2 to 16.7 m above the mean sea level. Referring to UNDP/FAO Agro-ecological regions of Bangladesh consists of two physiographic units, namely the " High Ganges River Flood Plain" in the lower part of the project and the " Tista floodplain" in the upper part of the project. Both physiographic units have a complex relief of broad and narrow ridges and inter-ridge depression, separated by areas with smooth, broad ridges and basins. The upper parts of the high ridges belong to 'high lands' while the lower parts of ridges and basin margins are mostly classified into 'medium highlands' . The basin centers are below 12.2 m and classified as 'Low land'.

Physiography

The soil of "High Ganges River Flood Plain" may be characterized as clayey with acidic in reaction and calcareous, while the soil of the " Tista Flood Plain" is coarse in texture with low organic matter and low moisture holding capacity.

The project area belongs to the Bangladesh's " dry zone" where the annual rainfall is lower than in other parts and highly variable. The rabi season temperature is moderately suitable to grow temperature zone crops such as wheat, potato, mustard and lentils. However conversely the temperature may retard vegetative growth in case of HYV paddy.

Surface Water Hydrology

Historically the project area had been susceptible to annual flooding during the monsoon. The project area experienced inundation due to flooding from river water and drainage congestion due to the combined effect of floodwater, rainfall and topography. About 53% of the project area went under water with a maximum inundation depth of one and half meter. Shallowly inundated land (about a quarter) was the next most important category. The project area abounds in many large and small Beels. Perennial water bodies and surrounding areas which used to be deeply flooded, therefore, had been quite important, These Beels and their surrounding areas play a very important role in the economy and environment of the area.

Agriculture

In about 58 percent of the cultivated area, a single crop like the Boro local varieties (LV) in low land and the Broadcast aman in medium low land dominated in the project area . Some double and triple crop could be observed in highlands or higher parts of the medium highland.

Under pre-project condition the input use levels in respect of quality seeds, chemical fertilizers, pesticides etc, were rather low because of insufficient and uncertain return. The crop management of major crops like b. aman and boro (LV) was inevitably extensive, as these were subject to flooding during monsoon or pre-monsoon period and also drought hazards.

Groundwater Hydrology

Of all the four rivers around Polder - D only Atrai flows perennially. So Atrai flow were used for surface water irrigation. Therefore surface water irrigation was not a major feature of the project area

The characteristics of the ground water hydrology remains at best an uncertain one. The surrounding rivers do influence the ground water reserve but little is known about the recharge capacity within the project area. Despite the recharge uncertainty irrigation in pre-project period depended for a few years on ground water. Ground water was pumped by small capacity hand pumps, shallow tube wells, etc.

Fisheries

Every year the flood within the project area enriched the water bodies with different species of fish which found aboundant feed in them. The dominant species involved included the major carps (Catla calta, Labeo rohita, Cirrhina mrigala), snake heads (chunna punctatus, C, striatus marulias), Anabas testadineus and anadroumous species. The Beels and Khals along with the flood plains were thus rich sources for a variety of fish production.

Both bengali culture and food habit and the shortage of livestock within the polder meant that fish from project area constitute an appreciable part of animal protein in the diet of the local population.

Livestock

The total number of cattle and bullocks at 130,000 heads, some 80,000 of which were termed as work animals those used for draft purposes and in transport. Some 22,000 had been estimated to be cows and 28,000 as youg stock. In addition there were an estimated 79,000 goats and sheeps and 245,000 poultry birds.

Social and Institutional

Farming, fishing or farming and fishing were the major occupations and cultivation of Betel leaves (pan) has been a prominent farming activity in the Polder - D area. The incidence of seasonal out-migration of labor from the project area was high, but in some cases, especially in the betel leaf growing areas there was seasonal in-migrating as well. Social and physical infrastructures such as schools, madrasas, hospitals, roads and transports systems were poorly developed and the level of literacy was eventually very low. There were very few social organizations such as formal or informal cooperatives, land less groups, youth clubs or fisherman societies. But the village societies used to play important roles in litigation, conflict resolution and social festivals.

Post - project environmental impacts

Various potential positive and negative impacts were investigated in the Evaluation such as Hydrological impact, agricultural impact, impact on livestock and fisheries, socio - economic impacts.

Hydrological impact

Based on the areas cultivated by sample household, the FCD infrastructure has had a limited impact in transforming land to shallower normal flood levels (increase of 17 percent of protected area); associated with this is a similar reduction in normal inundation period. However, in the unprotected impacted area the area of shallow flooded land has decreased and inundation period has increased. Thus there has been a negative off-side negative impact. The control area showed no significant changes in normal flooding., implying that changes in the impacted area are the project effect. The adverse impacts are attributed in part to inadequate drainage structures provided at both ends of the active channel (Kompo river) which passes through the middle of the project, connecting the Sib and Fakirni rivers. This lead to increased flood depths in adjacent unprotected areas and has exacerbated drainage congestion inside the project. The project has also increased flooding downstream.

Unfortunately the project has also suffered from regular breaches and public cuts, sometimes due to embankment failure during high floods, but mainly due to conflicting interests of insiders and outsiders and of farmers and fishermen . These cuts have been followed by sudden rapid inundation of supposedly protected areas and have caused intense dissatisfaction. There have also been substantial drainage congestion problems in some areas of the project, possibly due to inadequate capacity of drainage structures, and in some cases this has been the cause of public cuts.

Agricultural impacts

The project was expected to lead to a very substantial increase in cropping intensity (eventually to 235 percent) and to reduce crop losses and increase yields. In practice much of the project area and the control area is mono cropped. The most important crop is HYV Boro, which is cultivated to a greater extent in the control area and has been stimulated by expansion of ground water irrigation, and not by the FCD infrastructure.

However, overall paddy yields are slightly higher in the project area. T. L. aman and T. aus yields in particular are higher in the protected area, though in the peak flood years of 1987 and 1988 aus and aman yields were less than in control area. Overall there is some positive project impact on Agriculture, but it is far smaller than was anticipated by the project feasibility study.

Livestock impact

A comparison between livestock holdings in the protected area and in the control area revels little evidence of project impact. There are however significantly larger holding of bovine animals and poultry in the project area and incomes from livestock are slightly higher.

Fisheries impact

In Chalan Beel the polder has led to a reduction in the number of fishermen, a fall in the number of days a year the remaining fisherman spend fishing, and is a fall in their daily catch. As a result production of all capture fish species has dropped. This has been caused by the reduced area annually flooded, by the drying up of Beels and the blockage to normal fish migration routes, although general over fishing, fish disease and illegal fishing (non-project causes) have also contributed to the decline in output. A comparison of the project and control areas noted that far fewer of the non-farm households in the project area owned fishing nets, an independent confirmation of impact on capture fishing.

The project area has the largest number and area of fishponds than any of the projects studied in detail, but a high proportion of these are still vulnerable to flooding, and their productivity is low about half that normally expected. As a result increased fishpond output following protection is very limited, perhaps 430 mt a year compared to an annual loss of capture fisheries which is estimated to be between 1900 to2500 mt.

Socio-economic impacts

There appears to be no major differences in occupations or sources of income between project and control area, although secondary occupations are more common in the project. There is no difference in agricultural employment at household level. Although the survey data suggest that the cropping pattern inside the project implies about one-third more days per hectare are required than in the control area, wage rates in the project are slightly lower than in the control area.

There appears to have been negligible project impacts on secondary economic activity. There has been hardly any growth in numbers of rice mills, input and grain trading enterprises, light engineering workshops, while the number of oil mills has decreased. The latter, however, is primarily due to the replacement of rabi oilseeds by Boro and is not a project effect. It was noted that in the project area household heads were very likely to have a secondary occupation far more likely than in control area.

It was also reported that the amount of women's work in farm related activity was declining in the area. A decline in paddy husking, associated with expanded use by men of STW engines for this purpose, is a common phenomenon, but a general decline is not expected and again is an indicator of increasing poverty. Cautions must be exercised here, however since responses may have been covered by loss of paddy in the year 1991 floods.

Despite these indications, reported incomes were slightly higher in the project area. However, this may have been unreliable, since housing quality, sources of water, sanitary facilities and food availability all show negligible differences with the control area.

Inequality is still extreme (per capita incomes of large landholders are 5.8 times those of the land less) and landholding pattern have not changed relative to the control area. Local people noted disbenefits to fishermen, boatmen and the inhabitants of the neighboring areas, and are concerned about declining soil fertility and soil moisture, loss of fisheries and drainage congestion. Land acquisition was a serious problem. Although only 4 percent of house holds lost land, in 56 percent of the cases there was no compensation, and compensation was paid only slowly and after payment of bribes.

Conclusion

Chalan Beel Polder - D is a very large project with primarily FCD objectives and subsidiary provision for irrigation, which has successfully reduced flood depth and duration in years when the embankment remains intact, but failure to consider external impacts at the planning stage has led to regular public cuts by disbenefitted people outside, and the project has produced far smaller agriculture benefits than expected. Fishery disbenefits have been large, and economic performance is very poor, EIRR being negative based on agricultural and fishery impacts, but excluding the substantial external disbenefits.

Recommendation

To achieve the targeted benefit from an intervention like a Chalan Beel Polder D which is located in a complex topographical setting, the following salient features to be considered:

- Detailed hydrological studies should be performed;
- Hydraulic modeling to determine the backwater effect on the control area.
- Implementation of all the project components as identified and designed
- Routine and emergency maintenance fund provision should be made in the project document which can be used at the time of urgency.
- Operation and maintenance manual for project structures.
- Participation of the beneficiaries from the project identification stage through out the project life should be considered.
- Proper environmental studies should be carried out and appropriate environmental mitigation measures to be implemented to achieve environment friendly development option.s

- To determine the exact EIRR based on reality that dictates the project implementation strategy.

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Interview of farmers in selected mauzas, the proportion of each category of land as well as soil type. The farmers in the selected mauzas for transect survey are also interviewed;

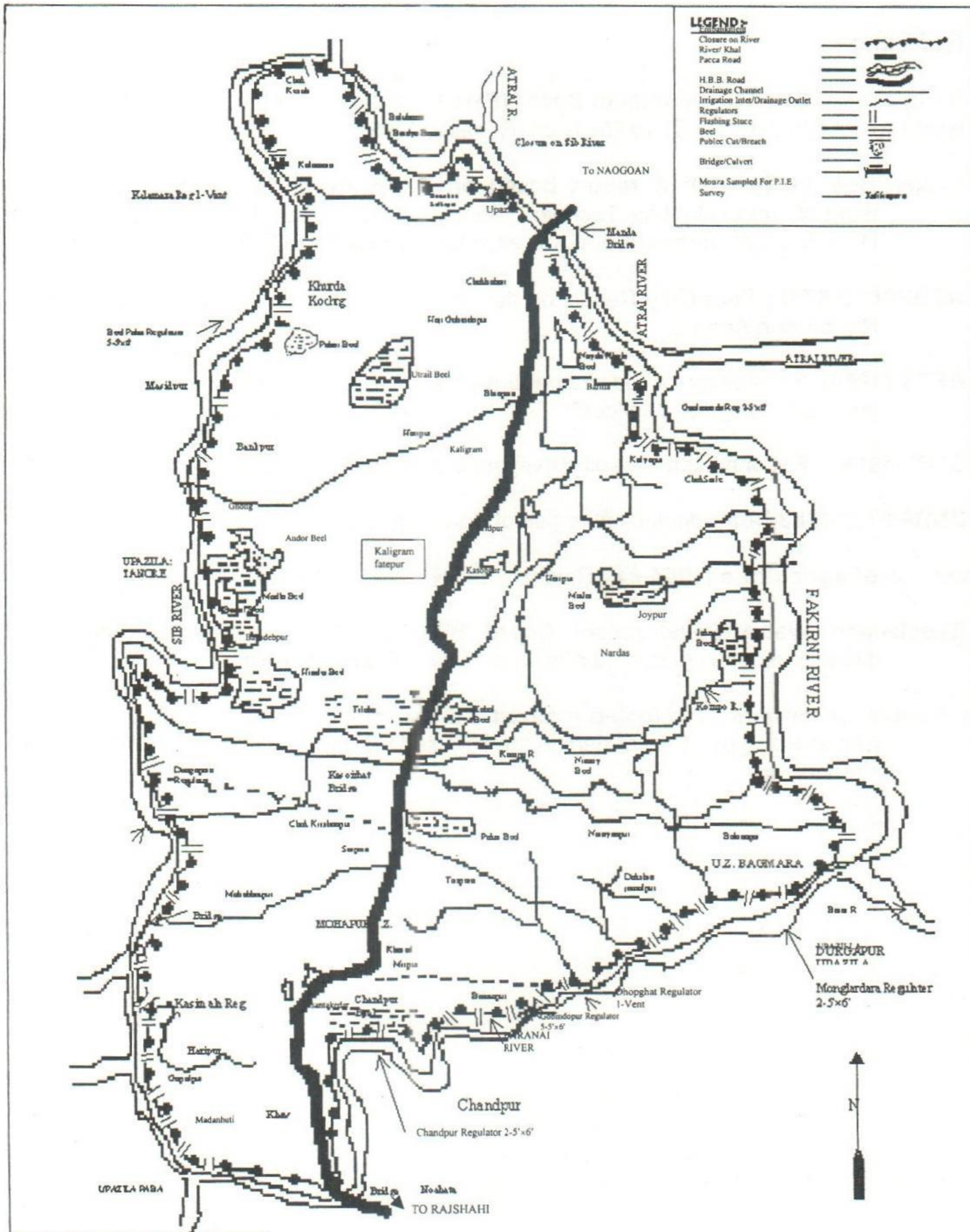

Figure no. 1: CHALAN BEEL
POLDER D. RAJSHAHI

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COMPARATIVE SEDIMENT STUDY OF TEESTA RIVER BEFORE CONSTRUCTION AND AFTER OPERATION OF BARRAGE

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Abstract

Extensive sediment study is most essential before construction of any hydraulic structure like barrage, bridge, irrigation canal, drainage channel, reservoir, sluice or any river Engineering works. Teesta Barrage project is one of the biggest project of Bangladesh Govt. Long term extensive sediment study has been done before construction and after operation of this barrage. In this paper an attempt has been made for comparative study of sediment characteristic of the Teesta River before and after barrage operation, which may help for proper barrage operation. Sediment transport rate between the U/S station Dakhin Kharibari and D/S station Kaunia after barrage operation and the sediment transport rate before barrage operation between the U/S station Dalia and D/S station Kaunia have been compared graphically. The D_{50} of suspended sediment has also been compared graphically separately between the above mentioned stations, before and after barrage operation. Year wise comparison of percentage of sand, silt and geometrical standard deviation of suspended material has been presented in tabular form of above stations, before and after barrage operation. This paper also provides information about deposition of sediment in silt trap constructed between the main canal and Canal Head Regulator, the channel characteristics of the Teesta River and information regarding Teesta Barrage Project and its objectives.

Introduction

The sediment load, the characteristics of the bed, bank and suspended material are the most important parameters of an alluvial river. The size and other characteristics of sediments ultimately determine many properties of the rivers. The sediment transport load is related to the size of the material. The coarser the bed material, the smaller the sediment transport. Even the sediment deposition depends on its size. The smaller the particles, the smaller the settling velocities and slower the particles settle. The bar deposits, the composition of bank material, and flood plains, the channel geometry and the planform of the rivers are influenced by the sediment characteristic. Hence a good knowledge of the sediment characteristics of the rivers is very essential for a better understanding of sedimentological and morphological processes of the river. As, for proper planning and design of any hydraulic structures sediment studies are most essential. Hence, for proper planning and design of Teesta Barrage the sediment studies of the Teesta River were started from 1961.

A large number of sediment samples were collected from the Teesta River at Dalia by EPWAPDA during the period from 1961 to 1968 and the samples were analysed in the

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then Hydraulic Research Laboratory (now River Research Institute). During the period from 1969 to 1972 the sediment samples has not been possible to collect due to non-cooperation movement and liberation war in Bangladesh. Then from 1973 to 1995 sediment sampie were collected from the river Teesta at Dalia, Dakhin Kharibari and Kaunia at random and the samples were analyzed in sediment laboratory of River Research Institute.

The main barrage over the river Teesta was completed in August 1990. After completing the construction of other necessary structure in connection with barrage such as canal head regulator, silt trap etc. the barrage came to operation in 1993. Teesta River carries a large amount of sediment load both in suspension as well as bed load during the period of flood and dry season. The sediment study along the vertical profile of the Teesta River shows that the sediment load is much higher near the bed than the middle or top. A large quantity of sediment in the bottom layer can enter into the main canal under sluice of the Canal Head Regulator. As a result, this sediment will reduce the capacity of the main canal and create problem of Canal Head Regulator and other regulators of main and secondary canals. In order to solve this problem BWDB made an agreement with BRTC of BUET to design a silt trap to review the optimization of planning and design of irrigation and drainage system of Teesta Barrage Project. The sediment entry into the main canal was calculated by BRTC of BUET and BWDB reviewing the sediment and discharge data of the year 1981 to 1987 available in RRI and design the silt trap for sediment arrest and removal. A physical model study was performed by RRI for the effectiveness of the designed silt trap. In this regard, a series of model tests was carried out with the different guide canal angle and flow condition and sediment deposition in the silt trap was observed and measured for different tests. From these model study it was recommended that guide canal with an angle of 80° would deposit about 80% of the sediment in the silt trap before entering into the main canal by producing a large semi-circular thalweg with reducing velocity. The design diagram of the silt trap has been shown in **Fig. 1** (RRI report no 112, 1994). Now the objective of the paper is to compare the sediment characteristics of the Teesta River before and after Barrage operation. So, in this paper, for the purpose of comparative sediment study, the two stations Dalia at the U/S and Kaunia at the D/S of the barrage site before barrage operation and the two stations Dakhin Kharibari at the U/S and Kaunia at the D/S of the barrage after barrage operation have been selected. The comparative study has been done taking the sediment data of 1989 to 1991 and 1993 to 1995 before and after the barrage operation respectively.

Literature review

A short description of channel characteristics of the teesta river

The Teesta River rises in the Himalayan in Sikkim, the highest peak in the catchment being Kangchanjunga (8585m). It flows for approximately 172 km through a mountainous area before emerging at Sivok on to the alluvial plains of North Bengal across which it flows in a braided course for a further 96 km before crossing the Bangladesh – India boarder. After a further length of 124 km it joins the Brahmaputra River near Chilmari. The catchment area upstream of the border is as follows:

Sikkim	7428 sq. km	mountainous	8981 sq. km
India	3136 sq. km	plains	1583 sq. km
Total			= 10564 sq. km

(Source: Report of Water investigation branch of EP, WAPDA, 1967)

The catchment area includes those of a few small tributaries rising in the foothills and joining the Teesta in the plains. The total catchment area in India and Bangladesh border is approximately 108,890 sq. km. The river flows in a bed of medium to fine sand. The general slope of the terrain, and therefore of the river, from Jalpaiguri, in India to Kaliganj, 29 km below Dalia is 0.76 m per mile. The slope exceeds 0.91 m per mile immediately upstream of Dalia. Teesta is the only river, which has the considerable streamflow throughout the year. Its runoff is supplied both by rainfall and summer snowmelt. The annual precipitation in the mountainous part of the catchment ranges from 254 cm to 508 cm with an average of about 343 cm per year for the catchment upstream of the border (Binnie, 1969).

Teesta barrage and its objectives

The idea of irrigation from the Teesta River was conceived since 1935. Most of the area found suitable for gravity irrigation falls in Bangladesh. Due to the portion of India, implementation of the project was slowed down. However from then both the countries started to formulate the project in their own parameters.

In our territory M/S Haigh Zinn and Associates prepared the preliminary feasibility study report of the project in collaboration with ACE Ltd. in 1960 (Pakistan) and M/S Binnie and Partners Ltd. prepared second one during 1968 to 1970. M/S Haigh Zinn and Associates proposed barrage site at Goddimari while M/S Binnie and Partners Ltd. proposed at Dalia. The project was only the survey, investigation and studies during Pakistan period. After independence of Bangladesh the Government take necessary action for understanding the project. In the meantime, India constructed a barrage at Gozaldoba over the Teesta, which is 100 km upstream of our present barrage site. Under this situation Engineers of BWDB and BUET reviewed the previous survey, investigation, planning and detailed engineering (Teesta Barrage Project Information, BWDB, 1993).

The project is bounded by the Teesta on the North, the Atrai on the West, Santahar-Bogra Railway line on the south and Bogra - Kaunia Railway line on the East. The purpose of the project is for irrigation, flood control and drainage for command area of 750,000 hectares of which 540,000 hectares are irrigable. The project covers seven districts of northern side of Bangladesh.

The main objective of the project is to increase the agricultural production and to attain the self-sufficiency in food in national level by irrigation and thereby create employment opportunities for the jobless people.

The vast area of North Bangladesh suffers from acute shortage of water every year. Drought is regular phenomenon in the project area even during pre-monsoon and post monsoon periods. The aim of the project was to supply irrigation water by diverting the flow of the river Teesta by heading up the water level in front of the barrage through a net-work of canal systems with construction of barrage across the river at Doani in Lalmonirhat mainly for supplementary irrigation during monsoon. Its purpose was also to cover the requirements as possible during lean period by crop diversification and irrigation by rotation. Teesta Barrage project now is a blessing to the distressed people. Now the farmer can grow three crops a year. Due to developed communication, marketing system and various agro-based cottage industries have established. Hydrological and ecological balance, positive impact on human resources development and poverty alleviation have been possible for construction of Teesta Barrage.

Trees of different kinds have been planted on both sides of the canal dykes and structure site. Water shed of reservoir, and canals, green crops and trees along the canal dykes have brought a positive impact on the climate in the barrage area. A vast land in the northern region of the country was suffering from desertification. Now it has been transformed into green cropland. On the whole, the socio-economic conditions of the people have been changed immensely due to the blessing of the barrage. The Salient features of the Teesta Barrage Project are given below.

Sl. No.	Description of Item	Quantity	Remarks
1.	Total benefited area	750,000 Ha	The Project was completed by different Phases
2.	Irrigable area	540,000 Ha	
3.	Barrage length – 615 m	1 No.	
4.	Canal Head Regulator – 110 m	1 No.	
5.	Colosure Dam – 2470 m	1 No.	
6.	Flood by pass – 610 m	1 No.	
7.	Silt trap for decantation of silt	1 No.	
8.	Flood Embankment	80 km	
9.	Main Canal	34 km	
10.	Branch Canal	275 km	
11.	Secondary Canal	1450 km	
12.	Tertiary Canal	2720 km	
13.	Drainage Channel	5000 km	
14.	Irrigation Structures	1512 No.	
15.	Drainage Structures	2320 No.	
16.	Turn out	1500 No.	

In this region, even the whole of the country has little space for recreation. The green parks, flower gardens and around the gigantic magnificent beautiful barrage and its head works and the lagoons in the old course of Teesta and silt trap attracts the tourist and many visitors to enjoy the beauty of nature. The migratory birds from the nearby Himalayans have made a natural Sanctuary in the reservoir in front of the barrage. The snow capped Kanchanjhanga is visible from the barrage site in the autumn. Many visitors and tourists come to this beautiful spot everyday specially in winter to enjoy the charming beauty of the nature.

Methodology

There are various methodologies for determination of sediment concentration and particle size distribution. Displacement method (field method), evaporation method and filtration method are used to determine the sediment concentration. In this study evaporation method has been used for determination of sediment concentration. In this method sediment concentration in mg/l is determined using the formulae

$$C_s = (W_s/V_{ws}) * 10^6$$

Where, C_s = Sediment concentration in mg/l
W_s = Wt. of sediment in gm
V_{ws} = Volume of water sediment mixture in ml.

The filtration method is suitable for low sediment concentration. It is a very quick method for determination of concentration. In this method filter papers of different pore dia. are used. The vacuum pump is used for suctioning water from the sample. But in case of the smallest clay particles and colloidal particles, the particles pass through the filter paper and these are not included in account of concentration. But evaporation method offers some advantages in simplicity of equipment and technique over that offered by the filtration method. For special studies where the amount of sediment is enough and consist mostly of sand sizes then the displacement method is used. The formulae for computation of sediment concentration in displacement method is

$$W_s = W/\{1-(d_s/d_w)\},$$ (Ref: ASCE Report No. 54, 1977)

Where, W_s = wt. of sediment (sand) in gm
W = Difference in wt. of tube with water only and water plus sediment
d_s = Specific gravity of sediment
d_w = Specific gravity of water
When $d_s/d_w = 2.65$
then the formula becomes
$W_s = 1.606\ W$

Similarly in case of particle size distribution of sediment, various methods are used according to the size of sediment. These are VA tubes, sieve-pipette, sieve-hydrometer, Bottom Withdrawal Tube (BWT) or settling tube methods. The VA tube method is fast, economical and accurate means of determining the size distribution of sediment in terms of fundamental hydraulic properties of particle and the fall velocity. This method is applied particularly for sample composed of sand i. e., sand size greater than 0.053 mm and smaller than 2 mm. Pipette, BWT and hydrometer methods are used in case of particle size distribution of fine sediment (SILT or CLAY). In this study sieve hydrometer and sieve pipette method have been used for particle size distribution of sediment. The formulae for particle size distribution using pipette method on the basis of Stoke's law is given by

$$W = (gd^2/18\nu) * (d_s - d_w)/d_w,$$ (Ref: ASCE Report No. 54)

Where, W, g, ν, d, d_s and d_w are settling velocity, acceleration of gravity, kinematic viscosity, diameter of particle, sp. gr. of sediment and sp. gr. of water respectively.

The Geometrical Standard Deviation for the suspended material has been calculated using the following equation:

$$\sigma_G = 0.5\{(D_{84}/D_{50})+(D_{50}/D_{16})\},$$ (Ref: DHL Manual, 1986)

Where,

σ_G = Geometrical Standard Deviation.

D_{84}, D_{50} and D_{16} are the dia. of the particle at 84, 50 and 16 percent by weight respectively.

In case of sediment classification, MIT standard classification method has been followed.

Bed load sampler, Bed load Transport Meter Arnhem (BTMA) is used for collection of bed load samples (the materials rolling or sliding over and near the bed). The formulae for computing bed load collected by BTMA is given by

$S_b = 2G/(0.085T)$, (Ref: DHL Manual, 1986)

Where, S_b = Bed load transport in Kg/Sec/m

G = Average dry wt. of sediment in Kg

T = Sampling time in Sec

The width of the intake opening of BTMA is 0.085 m.

In this study, for collection of bed load and computation of the same BTMA has been used.

A large number of bed material and bed load samples were collected from the only one station Dalia (barrage site) and tested in RRI. So, it has not been possible to compare the data in this study.

Laboratory sediment study and presentation of results

A large number of suspended sediment, bed material and bed load samples were collected by BWDB from the river Teesta starting from the year 1961 to 1995 before and after construction of barrage across the river Teesta. The samples were analyzed in the then Hydraulic Research Laboratory now River Research Institute. The suspended sediment samples were analyzed for determination of concentration and grain size distribution. The suspended sediment transport rates were also calculated from the sediment concentration using water discharge. The bed material samples were tested for only grain size distribution. The bed load samples were analyzed for determination of grain size distribution, dry weight. Bed load transport expressed in Kg/Sec/m was calculated. In this paper, comparative sediment study has been done taking the data of suspended sediment only for the years 1989 to 1995 before and after barrage operation in 1993. Results of comparative study of suspended sediment transport rate between Dalia (U/S) and Kaunia (D/S) for the years 1989 to 1991 before barrage operation has been presented graphically in **Fig. 2.** Sediment transport rate between the two stations Dakhin Kharibari (U/S) and Kaunia (D/S) for the years 1993 to 1995 after barrage operation has been presented graphically in **Fig. 3**.

The D_{50} of suspended sediment collected from the above-mentioned stations has also been compared and presented in **Fig. 4a & 4b** before construction and after barrage operation respectively.

Comparison of percentages of SILT, SAND and Geometrical Standard Deviation of suspended material has been shown before and after barrage operation in tabular form in **Table 1 & Table 2** respectively.

Table 1: Comparison of percentages of Sand, SILT and Geometrical Standard Deviation of the suspended sediment samples of U/S station Dalia (Doani) and D/S station Kaunia before barrage operation.

U/S Station: Dalia (Doani)					D/S Station: Kaunia				
No. of sample	Year	Percent by wt.		Geometrical Standard Deviation	No. of samp.	Year	Percent by wt.	Geometrical Standard Deviation	
		Sand	Silt				Sand	Silt	
192	1989	70-90	10-30	1.53-2.62	176	1989	60-90	10-40	1.72-2.67
160	1990	75-90	10-25	1.51-2.38	188	1990	65-85	15-35	1.58-2.63

Table 2: Comparison of percentages of Sand, SILT and Geometrical Standard Deviation of the suspended sediment samples of U/S station Dakhin Kharibari and D/S station Kaunia after barrage operation.

U/S Station: Dakhin Kharibari					D/S Station: Kaunia				
No. of sam.	Year	Percent by wt.		Geometrical Standard Deviation	No. of sam.	Year	Percent by wt.		Geometrical Standard Deviation
		Sand	Silt				Sand	Silt	
104	1993	78-85	15-22	1.60-4.42	88	1993	70-80	20-30	1.48-2.62
114	1994	70-85	15-30	1.58-3.74	94	1994	80-90	10-20	1.09-2.79

Discussions of results

From the results of the comparative study of sediment and some sediment parameters of the Teesta presented in **Table 1 & 2** specially it is seen that there is a wide variation of the percentages of sand, silt and geometrical standard deviation before construction and after barrage operation. It is seen clearly from **Table 1** that the percentages of sand and geometrical standard deviation ranges in the U/S station Doani is higher than that of the D/S stations Kaunia before barrage construction. But from the **Table 2** it is observed that the percentages of sand and range of geometrical standard deviation in the U/S station Dakhin Kharibari is less than the D/S station Kaunia. Perhaps this due to the fact that before construction of barrage the flow carries a large amount of sediment from the U/S station and a large portion of it settles down before reaching to the D/S station Kaunia. But after operation of barrage the percentages of sand and geometrical standard deviation range is less due to that fact that large size portion of sediment that carries the flow, deposits due to reduced flow velocity in U/S station for the obstacle of the barrage. In D/S station Kaunia the percentages of sand and Geometrical Standard deviation is higher after barrage operation, this due to the fact that the flow velocity increases due to the control of flow by the gates of the barrage. As a result, erosion occurs in the banks as well as bed scour takes place and the flow carries these bank and bed materials, which increases the percentages of sand and range of σ_G value in the D/S Kaunia.

It is seen clearly from the results of the comparative study of the suspended sediment transport rates versus days of the Teesta presented in Fig. 2 & 3 that the transport rate is higher in U/S station Doani than the D/S station Kaunia before construction of barrage. But after the barrage operation the sediment transport rate in the U/S station Dakhin Kharibari is less than D/S station Kaunia. This due to the fact that after barrage operation the larger particle of the sediment settles due to the reduction of flow velocity for the control of the flow by the gates of the barrage. As a result the sediment transport rate decreases. On the other hand, the water level is lower in D/S than that of the U/s of the barrage. Hence the flow velocity increases which causes bed scour and bank erosion. This flow carries a large amount of sediment, which increases the transport rate in the D/S station. From the comparative study of D_{50} for the above mentioned same U/S and D/S stations as shown in **Fig. 4a & 4b,** it is observed that the D_{50} values in the U/S station is much higher than that of the values of the same in D/S station before barrage construction and after barrage operation. The D_{50} are also higher in U/S station than that of the values of the same in D/S station. This is due to the fact that D_{50} doesn't depend on the quantity of the material but it depends on the size of the material transported along the river channel.

Conclusion and recommendation

Teesta Barrage Project is one of the biggest projects of Bangladesh Government and huge amount of money has been spent to implement this project. So, concerned authority must take proper care for barrage operation to get the fruitful benefit from the project for the purpose of which it was constructed. It is seen from the comparative sediment study that a large amount of sediment coming from U/S of the barrage is entering into the silt trap along with the water for irrigation through the Canal Head Regulator (CHR). From the model study result for designing the silt trap for the Teesta Barrage Project, it was recommended by RRI that 80% of the incoming sediment after passing through CHR would arrest in the silt trap. This is due to the reduction of flow velocity in this large semi circular area of thalweg in the silt trap as shown in **Fig. 1**. The rest fine sediment will enter in the main canal with water for irrigation. It was also suggested by RRI to remove the sediment from the silt trap time to time.

Acknowledgement

The authors like to express their heartfelt gratitude to the field personnel of the northern hydrology measurement division of BWDB and the technicians who were involved in testing of the samples.

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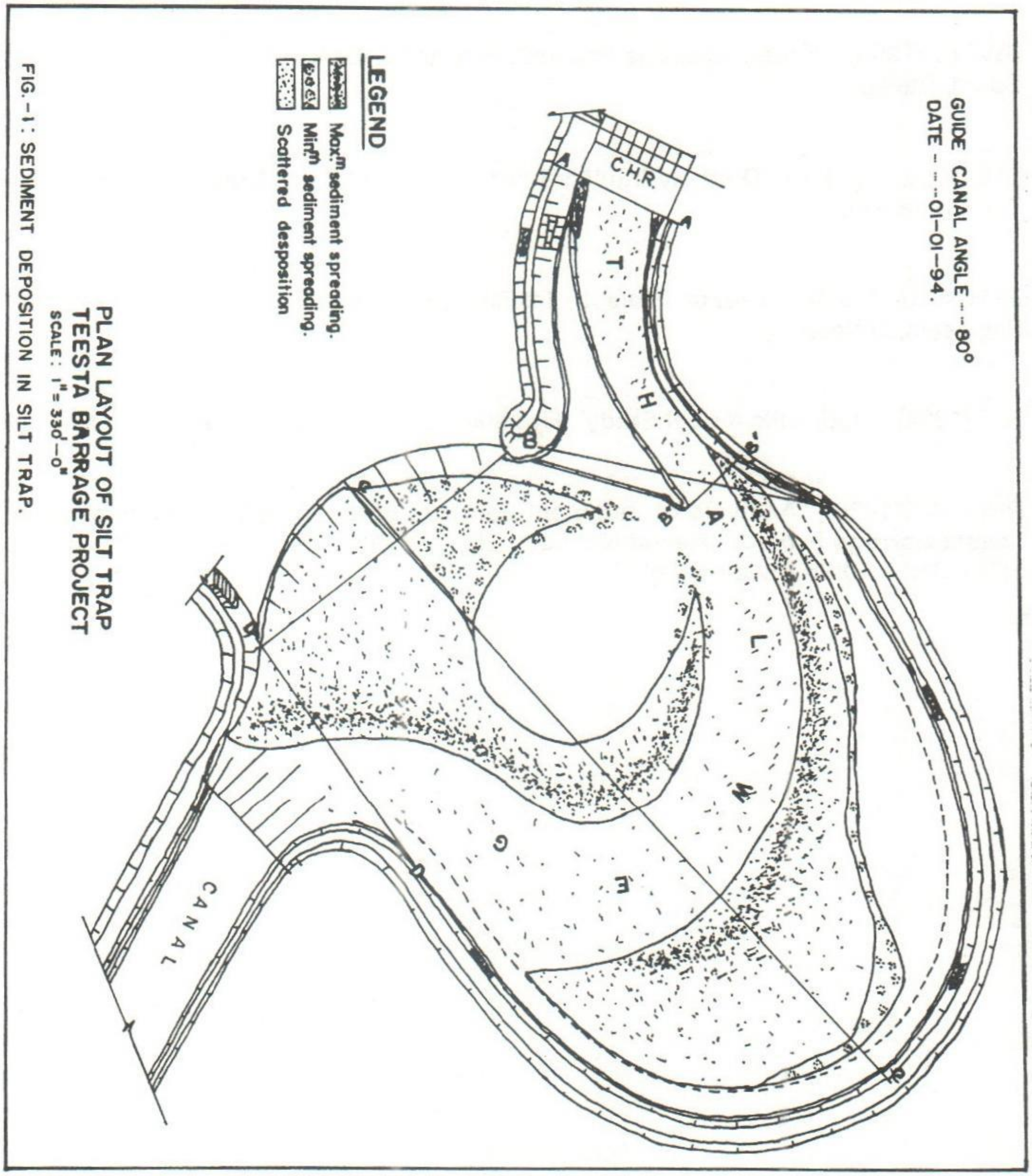

FIG.-1: SEDIMENT DEPOSITION IN SILT TRAP.

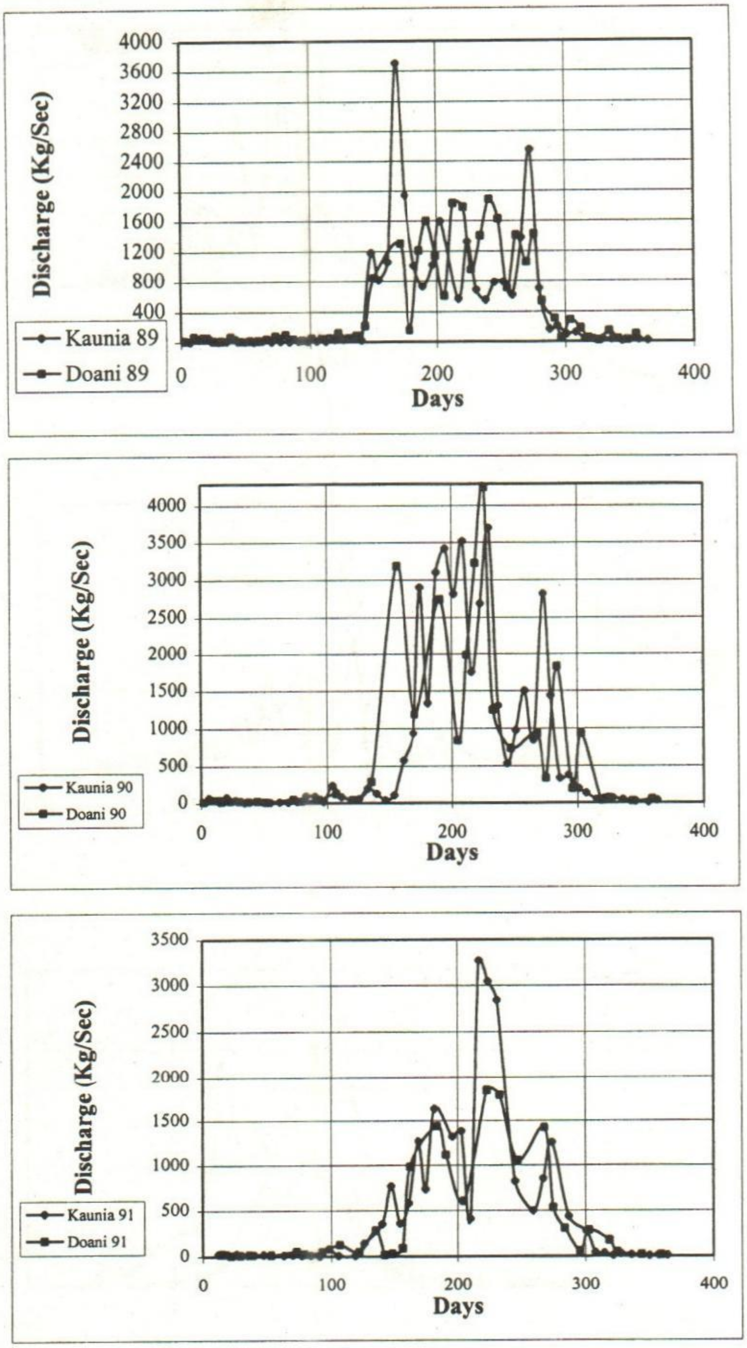

Fig. 2: Comparative study of sus. sediment transport rate at Doani (U/S) Vs Kaunia (D/S) before barrage operation for the years 89, 90 & 91 respectively.

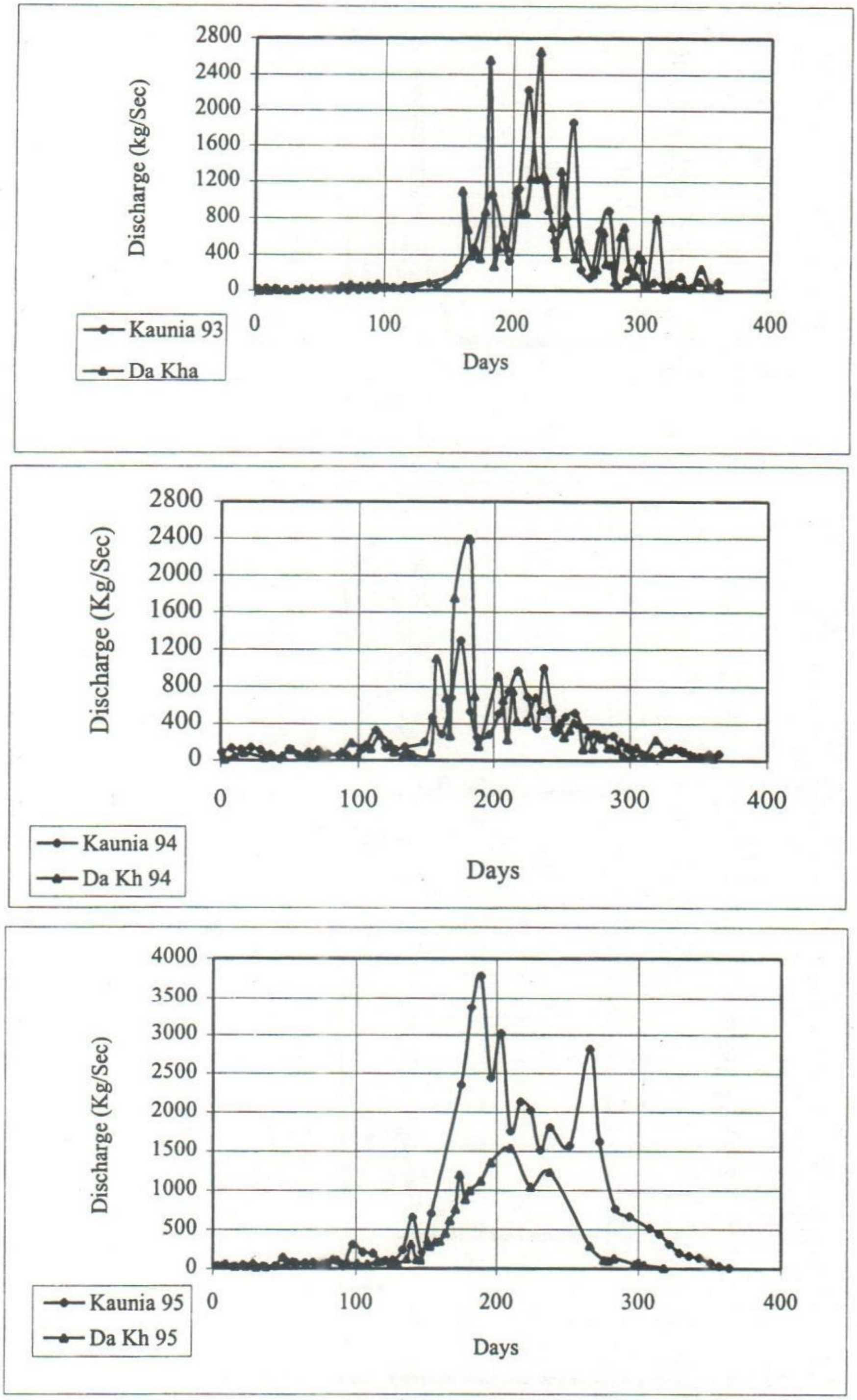

Fig.3: Comparative study of sus. sed. transport rate at Dakhin kharibari (U/S) Vs Kaunia (D/S) after barrage operation for the years 93, 94 & 95 respectively.

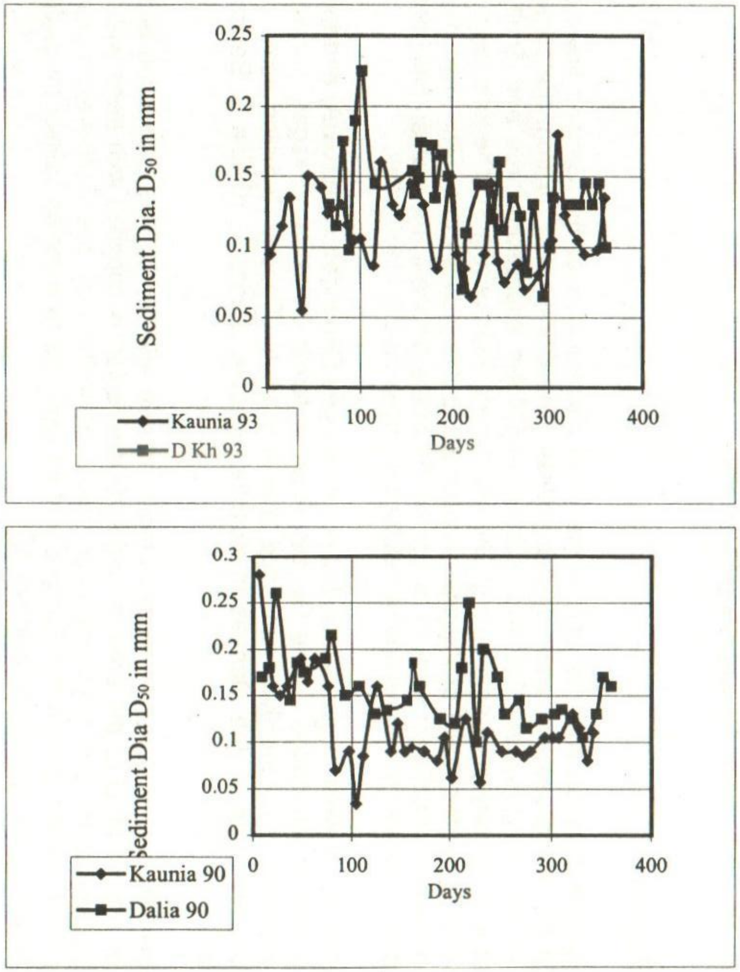

Fig. 4a: Year wise comparison of D_{50} (mm) of Da. Kha. (U/S) and Kaunia (D/S) after barrage operation.

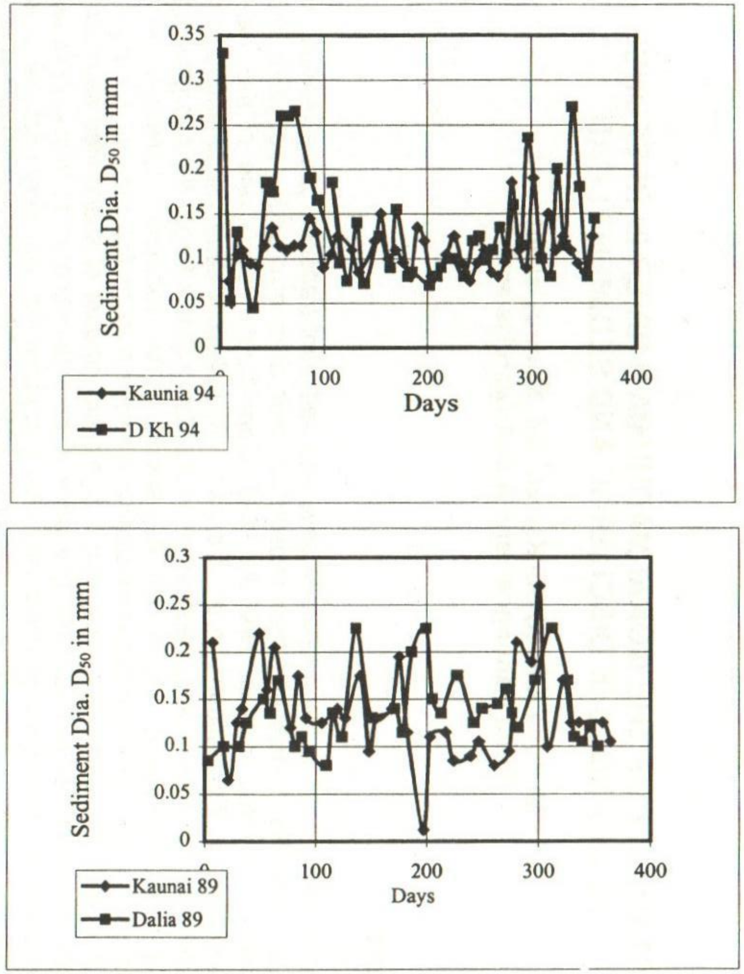

Fig. 4b: Year wise comparison of D_{50} (mm) of Doani (U/S) and Kaunia (D/S) before barrage construction.

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INSTABILITY OF THE GORAI-MADHUMATI RIVER IN RESPONSE TO CHANGES IN DISCHARGE AND SEDIMENT LOAD

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Abstract

This study describes the hydrological regime and sediment transport characteristics of the river Gorai-Madhumati and their impacts on river channel changes based on the very recent collected data of 1995 to 1998. Gorai-Madhumati is one of the main distributaries of the mighty river Ganges. Hydrological and sedimentological characteristics have been analysed based on the data obtained from the two gauging stations at Gorai Railway Bridge and Kamarkhali. In the monsoon, extensive river erosion takes place and huge suspended sediment flows through the channel. From this study it reveals that almost in every year a significant amount of suspended sediment deposits between these two stations. The ratios of the both maximum to minimum water and sediment discharge in a year are very high that make the river very unstable. It is likely that the results of this study may be helpful for the Gorai river restoration project and any measures taken for stabilizing the river.

Introduction

The Gorai-Madhumati River is one of the main distributaries of the Ganges flowing through the south-western region of Bangladesh. The Gorai takes off from the Right Bank of the Ganges near Talbaria, Kushtia and flows south-eastwards and finally empties into the Bay of Bengal through the Haringhata estuary (Fig.1). The total length of the river, upto the down of Patgati, is about 258.5 Km (161.5 miles). The river has different names at different parts of its course. From the offtake to the Kamarkhali ghat the river is known as the Gorai and thereafter it is known as the Madhumati.

The entire south-west region is included in the Ganges Depended Area (GDA) and the area is directly influenced by the Ganges River in terms of historic linkages, current direct impact and potentially commanded land. It consists of about 10 million acres of land located south of the Ganges in Kushtia, Jessore, Faridpur, Khulna & Barisal districts.

The southern part of the south-western region is also affected by tidal inundation and saline intrusion. Most of the area in south-western region is supplied with fresh water from the Ganges through the Gorai-Madhumati river. In this way, the south-west region is very much dependent on the Ganges flow through this river for its salinity intrusion problem as well as agriculture (Hannan, 1981).

The Gorai is a Right Bank distributary of the Ganges River debauching independently into the Bay of Bengal after capturing some local flows. Historic records indicate that the

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river was once a major possible course of the Ganges River. At present the Gorai is fast loosing its conveyance function as the average discharge of the river is only about 1400 m^3/sec representing some 13% of the Ganges River flow. Even this represents only the wet months from June to October. During rest of the year, the offtake remains almost dry. This is primarily due to the Right Bank sand bar of the Ganges River that virtually chokes the Gorai mouth. Water level analyses at specific discharge indicate that the river is going through a secular period of aggradation. The flow in a river varies widely over the year as well as from year to year. The river carries both water and sediment and they together control the slope and characteristics of the stream. The sediment discharge also varies widely with the variation of water discharge (Islam, 1998).

Flow in this river varies very widely. The ratio of maximum to minimum discharge for both water and sediment is very high because of very low flow in the dry season. But to maintain an equilibrium condition for alluvial channels, it is widely recognized that this ratio for both water-sediment discharge should be as low as possible (Islam, 1998). Under the above circumstances, the following objectives are set forth in this study:

- To observe the latest hydrological behavior of the river.
- To determine a feature that gives an idea about sediment transport and deposition characteristics of the river.

Literature review

Distributaries of the Ganges

The river Gorai is the largest distributary channel of the Ganges. The other major distributary channels are the Bhairab, the Arial Khan, the Mathabanga, the Chitra and the Nabaganga. Formerly these distributary systems played an important role in supplying freshwater to the tidal zone and were the key to surface water development in the south-west region. At present the distributary systems in the west (the Bhairab, Mathabanga, Chitra and Nabaganga) have become completely disconnected from the Ganges and collect only rainfall. Their channels have shrunk to mere traces of their former size.

The Gorai is also facing almost no flow situation in the dry season due to formation of sandbars at the Gorai mouth. Each year, the distributaries of the Ganges carry huge discharge and sediment load and the river course has a general tendency of instability. Rivers are constantly migrating laterally which results information of many meander loops, latter cutoffs and consequently many oxbow lakes along the channel (Shafiqul et al, 1997).

Flow and sediment regime of off-take channel

The flow of the Gorai-Madhumati river largely depends upon the hydrodynamic and morphological conditions of the Ganges near the Gorai off-take. With the help of some theory the flow from the parent Ganges River can be explained. The wandering pattern of the major boundary rivers has particular implications for the hydrology, hydraulics and morphology of the regional rivers which are distributary channels. By definition, the channel mouth of a spill channel is located at the edge of the active corridor. Statistically, it is more likely to be in an embayment than in a nodal reach, because nodal reach is much shorter than embayment.

The amount of water sediment spilled into the distributary depends primarily on the position of the mouth within the pattern of the wandering river. In turn, the channel characteristics and morphology of the distibutary depend on the inputs of water and sediment and their annual variations are controlled firstly by channel dynamics in the parent river and secondly by conditions of the off-take mouth. Recognizing the morphologically dynamic nature of a wandering pattern river, it is to be expected that large variations in inputs to distributaries will occur, to which the size and the morphology of the distributary river constantly adjust (Navera, 1996).

Previous flow conditions

During the last three decades, diversion of the Ganges flow to the Gorai has been highly variable due to moving sandbars that seriously obstruct the flow into Gorai periodically. The average annual discharge at Gorai Railway Bridge and Kamarkhali are 1409 and 1255 cumec respectively. Annual average discharge at Gorai Railway bridge before 1975 was 1538 cumec and after 1975 it is 1302 cumec.

The average minimum flows at Gorai Railway Bridge and Kamarkhali before 1975 were 87.32 and 108.5 cumec respectively and those after 1975 were 28.83 and 25 cumec respectively. The minimum flows were 0.057 cumec and 0.764 cumec at Gorai Railway Bridge and Kamarkhali respectively. The average discharge in the winter at Gorai-Railway Bridge was 475.42 cumec and in rainy season 6035.35 cumec and in the year 1967 the maximum discharge was 7568 cumec.

In 1988, the year of devastating flood in Bangladesh, the highest discharge was 8490 cumec at Gorai Railway Bridge site. On the basis of the data available from 1946 to 1990, BWDB, the recorded highest discharge at Gorai Railway Bridge is 8490 cumec in 1988 and at present even no flow occurs during the lean period. Actually in winter no water of the Ganges falls into the Gorai and hence the upstream parts remain almost dry. But in the rainy season huge amount of water from the Ganges passes through the Gorai. The Minimum discharge at Gorai-Railway Bridge in 1988 was only 0.286 cumec on the contrary the maximum discharge in same year was approximately 8490 cumec, that is the ratio of maximum discharge to minimum discharge was app. 30000, which is too high. That is, discharge variation through out the year is as high as possible (Islam, 1998).

Sedimentological characteristics

According to RSP (1996) average grain size of bed material of Ganges and Gorai river are 0.12 and 0.179 mm respectively. Based on sediment transport data measured by BWDB in the period 1965-1970 average yearly volume of suspended sediment transport of the Gorai river was 47 million tons of which 29 million tons occupies fine sediment particles i.e., 62% of the total yearly volume. The remaining 18 million tons occupies coarse sediment particles. Gorai river receives only about 8.58% of the total volume of sediment transport of the parent Ganges river (RSP, 1996).

Methodology

Surface Water Hydrology-II (SWH-II), BWDB are measuring hydrological parameters at several gauging stations along the main rivers. Hydrological data of the two stations Gorai Railway Bridge and Kamarkhali have been collected from SWH-II. BWDB measures also suspended sediment transport at a few selected stations in the main rivers. In this river suspended sediment transport is considered to be more important than bed load transport. Therefore, attention is focused on suspended sediment transport measurements.

Normally coarse suspended sediment particles are examined at the field by separating it from the fine particles (also called as bulk suspended sediment particles) and these fine sediment particle are sent to River Research Institute, Faridpur for laboratory analysis. Average monthly representative value have been determined by averaging the weekly data that have been quoted from the previous sediment testing report of Geotechnical Research Directorate of RRI, Faridpur. Based on the available monthly average data, present hydrological behaviors of the river and sediment deposition characteristics between these two gauging stations have been studied.

Results and discussion

Present hydrological behavior

Monthly mean water level and average water discharge from the year 1995 to 1998 at the station Kamarkhali for the month of January to June and December as the low flow period of a year appear in Table.1. It is obtained from these data that monthly mean water level varies from 0.96 to 2.56 m (PWD Datum) and average water discharge varies from 5.97 to 159.8 cumec. The variations of monthly mean water level and average water discharge for the residual months of July to November as the comparatively high flow period of a year at the gauging stations Gorai Railway Bridge and Kamarkhali have been given in Table.2.

Table 1: Monthly Mean Water Level and Water Discharge Variation at Kamarkhali

Hydrological Parameter	Year	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Dec.
Mean Water Level in meter (PWD Datum)	1995	0.96	**1.01**	1.06	1.21	1.68	3.31	1.8
	1996	1.16	1.18	**0.99**	1.11	1.56	1.96	1.97
	1997	1.2	1.12	**1.05**	1.13	1.22	1.8	2.35
	1998	1.85	1.12	**1.02**	1.37	1.58	2.51	2.56
Water Discharge (cumec)	1995	19.31	**10.52**	10.68	18.45	62.42	486.15	74.66
	1996	17.64	10.14	**5.97**	7.53	35.89	80.45	31.67
	1997	14.71	**12.09**	12.14	15.75	19.57	53.06	68.61
	1998	55.31	38.98	**26.63**	56.32	80.22	159.8	97

Table 2. Monthly Mean Water Level and Average Discharge Variation at the Gauging Stations Gorai Railway Bridge and Kamarkhali

Month & Year	Gorai Railway Bridge		Kamarkhali		Difference in Water Level (m)	Difference in Water Discharge (cumec)
	Mean Water Level in meter (PWD Datum)	Average Water Discharge in Cumec	Mean Water Level in meter (PWD Datum)	Average Water Discharge in cumec		
Jul-95	10.50	2341.86	7.04	2387.36	3.46	-45.50
Aug-95	11.25	3186.11	8.02	**3051.99**	3.23	134.12
Sep-95	11.28	**3043.99**	7.64	2894.35	3.64	149.64
Oct-95	9.34	1471.51	5.74	1342.88	3.61	128.64
Nov-95	7.10	526.72	3.83	351.14	3.27	175.58
Jul-96	10.35	2315.12	6.56	2072.22	3.79	242.90
Aug-96	11.75	3887.75	7.98	3154.51	3.77	733.23
Sep-96	11.80	**3926.79**	8.07	**3307.49**	3.73	619.30
Oct-96	9.04	1375.08	5.72	1134.42	3.32	240.66
Nov-96	6.73	400.18	3.40	369.03	3.33	31.15
Jul-97	9.57	2088.68	6.03	1883.18	3.54	205.50
Aug-97	11.26	**3385.18**	7.60	**3125.49**	3.66	259.69
Sep-97	11.23	3278.36	7.44	2617.91	3.79	660.45
Oct-97	8.79	1119.34	4.61	808.07	4.18	311.27
Jul-98	10.71	2688.44	6.83	2660.12	3.88	28.32
Aug-98	12.52	**4560.13**	8.50	**3572.70**	4.02	987.43
Sep-98	12.15	4133.50	7.59	2813.33	4.56	1320.17
Oct-98	9.63	1330.77	5.72	998.31	3.91	332.46
Nov-98	7.93	651.22	4.68	568.58	3.25	82.64

It is found from table.2 in the months of July to November mean water level and average water discharge varies from 2.55 to 8.5 m (PWD Datum) and 351.14 to 3572.7 cumec respectively at the gauging station Kamarkhali.

The months in which maximum flow occurs in a year are August and September and the months in which minimum flow occurs are February and March (Table. 3). The ratio of maximum to minimum monthly average flow at the station Kamarkhali ranges from 134.16 to 554.01 in which 554.01 was in the year 1996. The mean water level in the year 1996 varies from 0.99 to 8.07 m (PWD datum). It has also been mentioned that in the catastrophic flood year 1998 the calculated ratio of maximum to minimum flow is decreased to the value as 134.16

Table.3: Maximum and Minimum Monthly Water Discharge Variation with the Year at the Gauging Station Kamarkhali.

Year	Month of Max. Flow	Max. Flow (m^3/sec)	Month of Min. Flow	Min. Flow (m^3/sec)	Ratio of Max. to Min. Flow
1995	August	3051.99	March	10.68	285.77
1996	September	3307.49	March	5.97	554.01
1997	August	3125.49	February	12.085	258.63
1998	August	3572.7	March	26.63	134.16

Sedimentological characteristics

Based on the available data from 1995 to 1998 at the station Kamarkhali, it is certain that there is very low amount of suspended sediment discharge in the months of January to June and December (Table.4). During the time from the months of January to June and December may be called as the period of low water discharge and also low suspended sediment discharge. So it is seen that suspended sediment discharge directly related to water discharge.The ratio to Maximum and minimum average monthly-suspended sediment discharges in the different years have been given in Table 5. This table represents that there is highly variation of suspended sediment discharge in a year at the gauging station Kamarkhali. It may be mentioned from the recently collected data that highly variation of water discharge as well as suspended sediment discharge make the river very unstable. This type of highly variation of suspended sediment flow causes a river into severe bank erosion as well as sediment deposition problem and finally also a complication situation which is very difficult to face.

Table.4: Average Monthly Suspended Sediment Discharge Variation at the Gauging Station Kamarkhali from the year 1995 to 1998

Hydrological Parameter	Year	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Dec.
Average Suspended Sediment Discharge (kg/sec)	1995	0.24	**0.17**	0.28	0.44	0.45	66.32	1.09
	1996	0.33	0.03	**0.01**	0.085	0.16	1.07	2.83
	1997	0.15	0.10	**0.08**	0.19	0.099	0.57	0.43
	1998	0.15	1.64	**0.13**	2.41	0.65	1.85	6.96

Table 5: Maximum and Minimum Suspended Sediment Discharge Variation with the Years at the Gauging Station Kamarkhali

Year	Month of Max. Sed. Discharge	Max. Sed Discharge (kg/sec)	Month of Min. Sed. Discharge	Min. Sed. Discharge (kg/sec)	Ratio of Max. to Min.
1995	August	1818.83	March	0.17	10699
1996	September	5991.27	March	0.01	599127
1997	August	2880.336	March	0.08	36004.2
1998	August	922.80	March	0.13	7098.46

Table.6 shows that significant amount of suspended sediment discharge observed in the months of July to November. From the available data at the stations Gorai Railway Bridge and Kamarkhali, sediment deposition rate and total annual sediment deposition between these two stations have been calculated as the most of the suspended sediment discharge occur in the months of July to November in a year (Table.6). It is observed that huge amount of suspended sediment deposits throughout its course that comes with the monsoon high flows. Table.6 also clearly shows that a significant amount of suspended sediment deposits in every year between these two stations Gorai Railway Bridge and Kamarkhali especially in the monsoon.

This is evident from the Table 6 which shows that total annual sediment deposition between these two stations varies from 7.34 to 53.04 million tons of which bulk suspended sediment (fine sediment particles i.e. silt fraction) is 70 to 80%. Fig.2 shows the average monthly-suspended discharge as the percent of the months July to November in the different year at the gauging station Gorai Railway Bridge.
It has been observed that in the year 1997 total inflow volume of suspended sediment at Gorai Railway Bridge was very much higher than that in other study year including in 1998 which was one of the catastrophic flood year of the last decade (Fig.3).

Conclusion

It is clear from the available hydrological and sediment transport data for the time period considered that both discharge and sediment load are highly variable in the Gorai-Madhumati river. The ratio of maximum to minimum discharge and sediment load at Kamarkhali station can be as high as 554 and 599127 respectively. It is also evident from the analysis of data that every year huge amount of sediment deposition (maximum 53.04 million tons) occurs between the Gorai railway bridge and Kamarkhali gauging stations. All these explain why the river is so unstable in terms of bank erosion and deposition.

When a river is affected by a change in discharge or sediment load, it may develop sediment transport discontinuities.

Table 6.Bulk and Coarse Suspended Sediment Flow Characteristics at the Gauging Stations Gorai RailwayBridge and Kamarkhali

Month & Year	Gorai Railway Bridge			Kamarkhali				
	Bulk Suspended Sediment Discharge (Kg/sec.)	Coarse Suspended Sediment Discharge (Kg/sec)	Total Suspended Sediment Discharge (Kg/sec.)	Bulk Suspended Sediment Discharge (Kg/sec.)	Coarse Suspended Sediment Discharge (Kg/sec)	Total Suspended Sediment Discharge (Kg/sec.)	Average Sediment Deposition Rate (Kg/s)	Average Sediment Deposition Per Month (Million tons
Jul-95	686.30	356.11	1067.41	897.02	102.58	999.59	67.82	0.18
Aug-95	2418.31	478.16	2896.48	1487.36	331.47	**1818.82**	1077.66	2.79
Sep-95	4147.17	432.83	**4635.04**	1149.79	305.04	1454.83	3180.21	**8.24**
Oct-95	1319.89	307.32	1502.21	357.14	81.39	438.53	1063.69	2.76
Nov-95	333.15	44.81	677.96	26.46	*	26.46	651.50	1.69
Total =								15.66
Jul-96	1055.02	475.17	1530.20	665.72	47.26	712.98	817.22	2.12
Aug-96	3437.67	1043.73	4481.41	1313.83	524.17	1838.00	2643.41	**6.85**
Sep-96	4173.63	1225.37	**5399.01**	1556.87	645.12	**5991.27**	-592.26	-1.54
Oct-96	2889.12	292.47	3181.59	3636.67	152.61	3789.28	-607.69	-1.58
Nov-96	565.28	37.72	603.01	32.51	*	32.51	570.50	1.48
Total =								7.34
Jul-97	2195.85	875.07	3070.92	547.06	195.17	742.22	2328.70	6.04
Aug-97	3700.57	1610.57	5311.15	2481.98	398.35	**2880.34**	2430.81	6.30
Sep-97	12136.71	3555.51	**15692.22**	1012.34	340.56	1352.90	14339.32	**37.17**
Oct-97	778.58	727.02	1505.60	105.27	36.08	141.29	1364.31	3.54
Nov-97	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
Total =								53.04
Jul-98	1839.69	1161.11	3002.43	304.17	222.75	527.32	2475.11	6.42
Aug-98	3916.69	1962.89	**5879.59**	547.90	374.89	**922.80**	4956.79	**12.85**
Sep-98	2930.88	1559.63	4490.50	410.67	273.26	683.95	3806.55	9.87
Oct-98	742.59	344.06	1086.65	77.63	28.00	105.63	981.02	2.54
Nov-98	200.65	129.45	330.11	73.64	*	73.64	256.47	0.66
Total =								32.34

The discontinuities move along the channel resulting in erosion and deposition which changes channel morphology and gradually restores stability of the system. The maximum discharge during 1995-98 is 4560 cumec whereas it was 8490 cumec in 1988. The minimum flow data during 1995-97 indicate a marked decrease in the lean period flow compared to the pre Farakka (before 1975) and even post Farakka situation. This bears explicit testimony of the fact that the Gorai mouth has been narrowed by sediment deposition. However, increase in the both maximum and minimum flow in 1998 may be caused by the excavation of the Gorai mouth under the Gorai river restoration project.

It is interesting to note from the analysis that although the maximum discharge occurred in 1998, the total suspended sediment discharge and deposition in the year 1997 were still higher than that of 1998. This again points to the fact that the sediment discharge is not only dependent on the flow discharges rather it depends on the sediment supply from the upstream. Here, decrease in the sediment discharge resulted from the upstream excavation of the river. It can be concluded from the analysis that maintaining a required minimum flow in the Gorai river i.e., decreasing the both maximum to minimum discharge and sediment load ratio many present hydro-morphological and other related problems can be encountered up to a certain extent.

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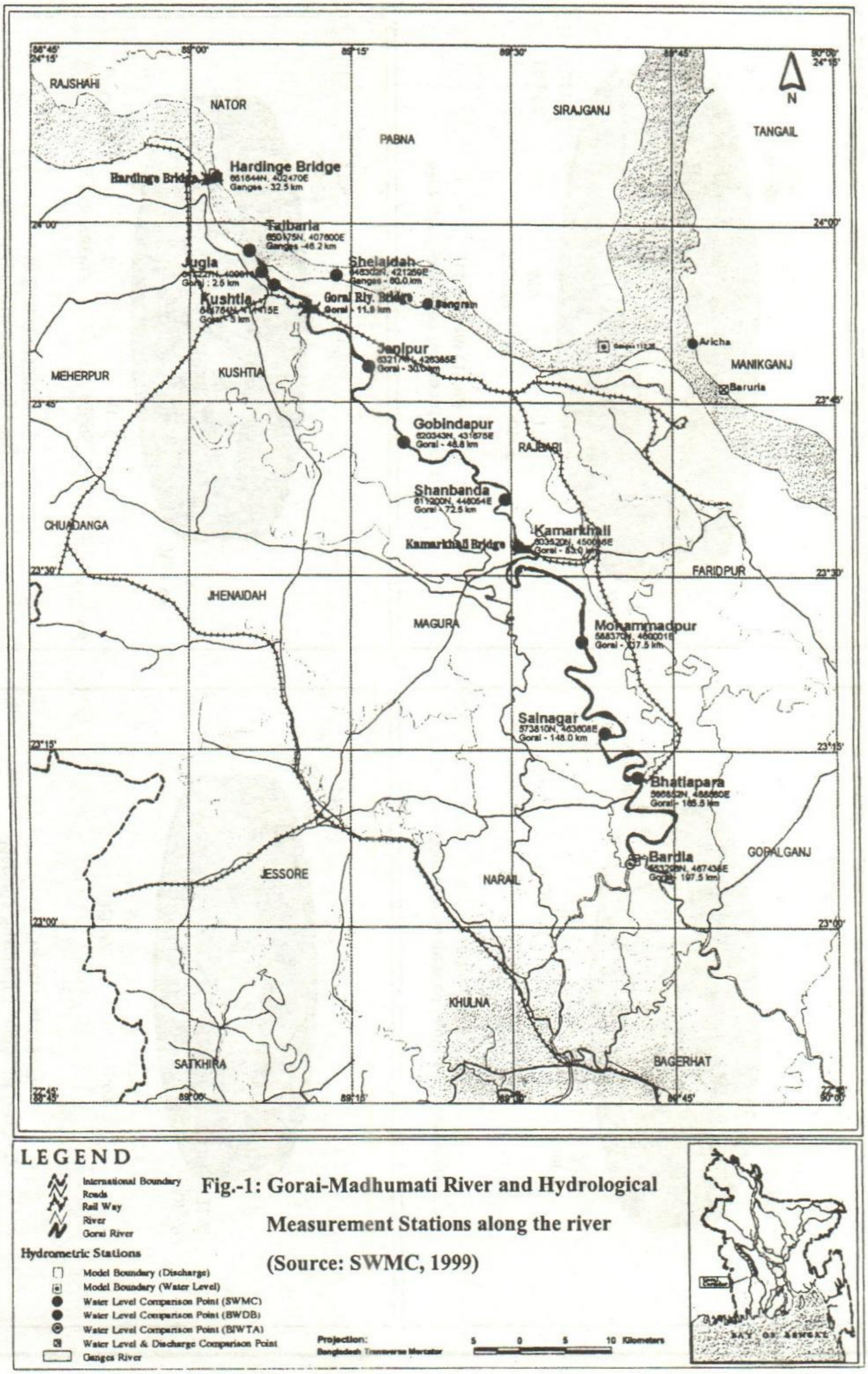

Fig.-1: Gorai-Madhumati River and Hydrological Measurement Stations along the river (Source: SWMC, 1999)

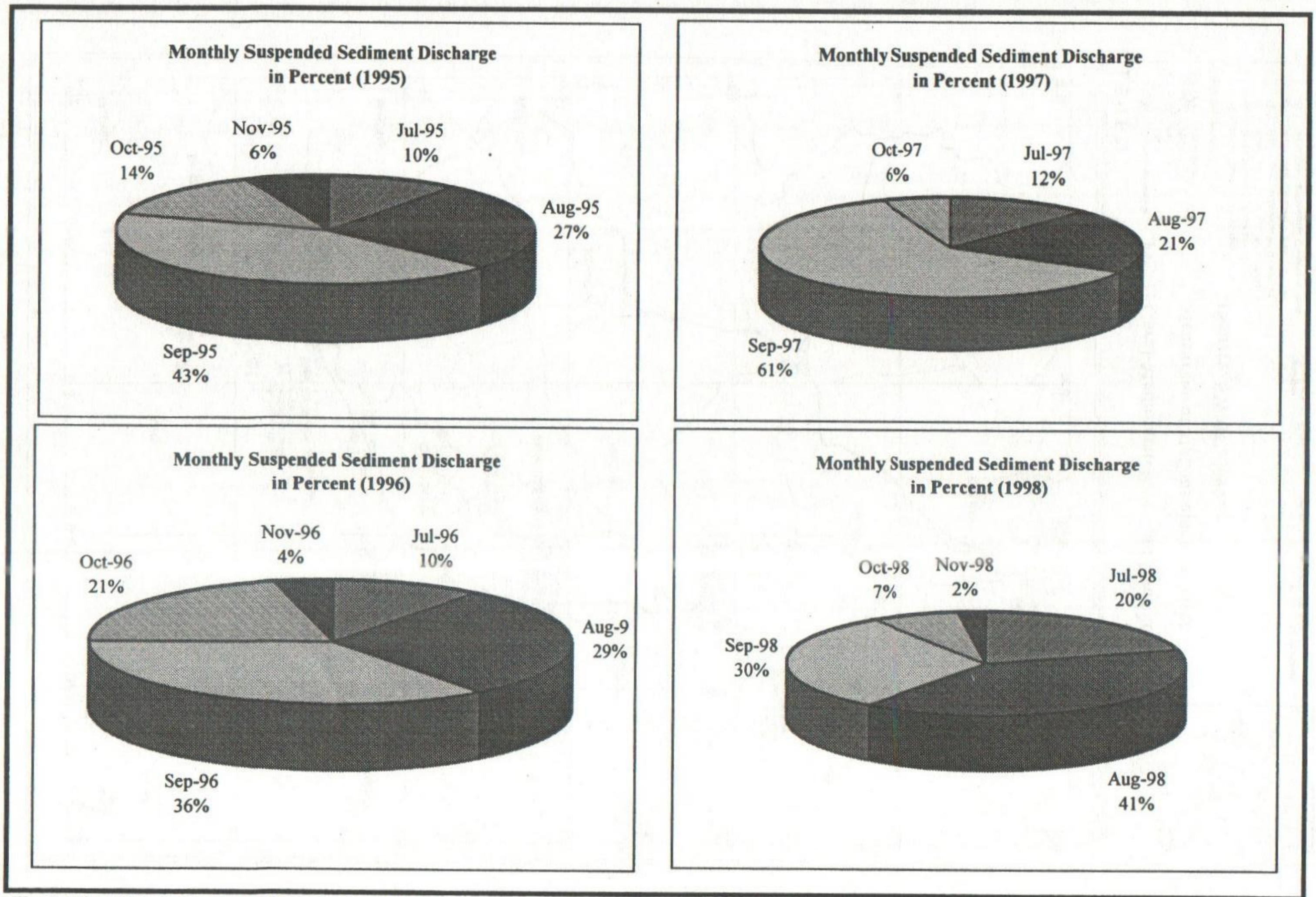

Fig.2: Monthly Suspended Sediment Discharge as the Percentage of July to November at the Station Gorai Railway Bridge

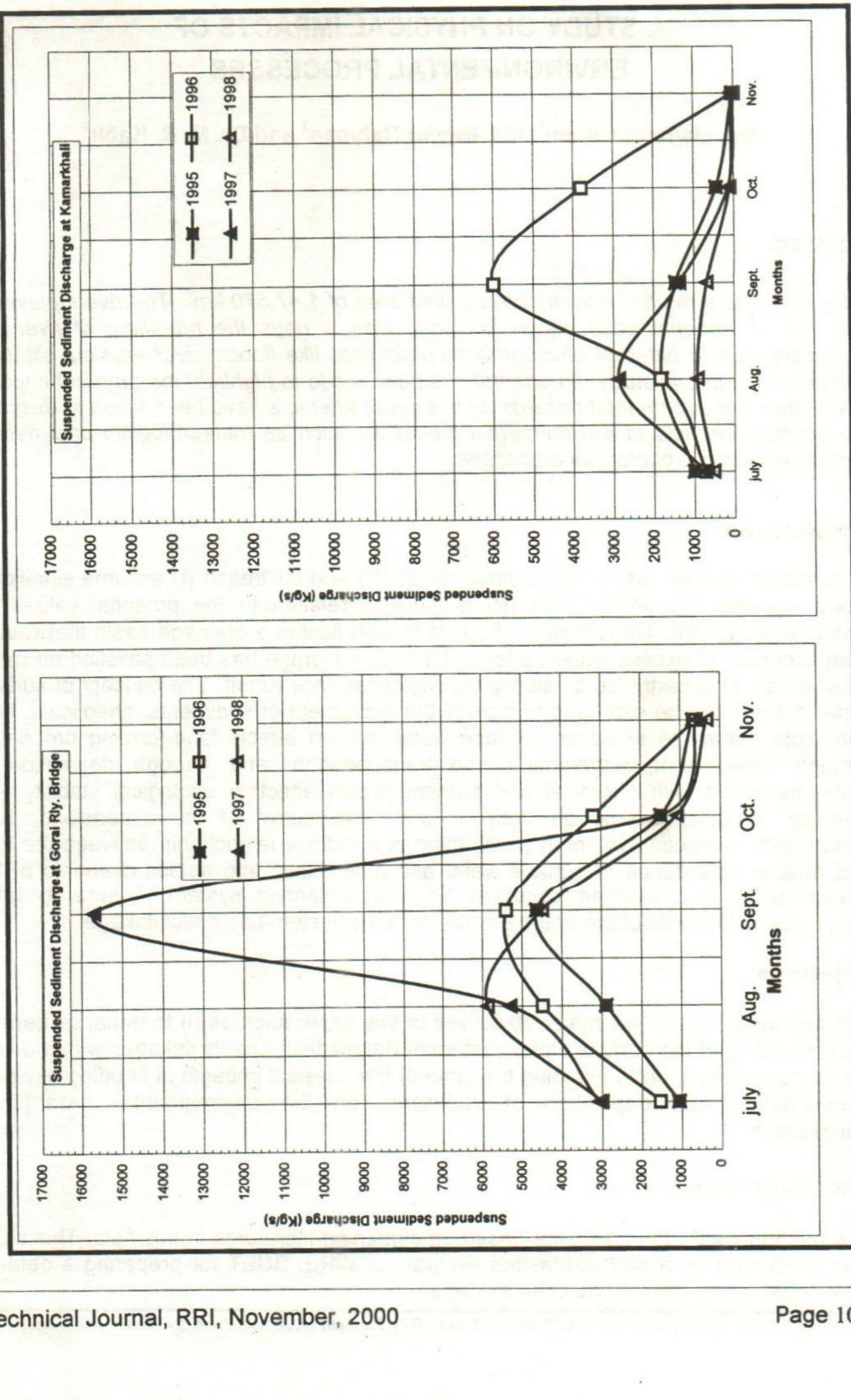

Fig 3: Average Monthly Suspended Sediment Variation through the Years at the Gauging Stations Gorai Railway Bridge and Kamarkhali

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A STUDY ON PHYSICAL IMPACTS OF ENVIRONMENTAL PROCESSES

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Abstract

Bangladesh is a riverine country having land area of 1,47,570 km^2. The rivers played a vital role in national economy in the past. Now a days the problems of river are increasing due to different environmental processes like floods, river erosion, siltation, accretion etc. In this study, an attempt has been made to highlight the physical impacts due to these environmental hazards. In this paper attempts have been taken to describe the physical impacts of environmental processes such as meteorological parameters, surface waters morphological processes.

Introduction

In humid watershed areas, the surface runoff (R) and infiltration (I) assume significant value because the precipitation (P) is large in relation to the potential values for evaporation (E) and Transpiration (T). R is stream flow in a drainage basin that results from accretion of excess water I after soil moisture storage has been satisfied on many small areas. R is partly fed by slowly moving subsurface runoff. The concept of surface water system can be extended to include the movement of sediments, chemicals, heat and biota contained in water. Surface water system affects land-forming processes through detachment, entrainment and transportation, and through deposition of materials linked with chemical and nutrient cycles affecting ecological stability and diversity. In other words, an area of land, irrespective of the dimensions, is a dynamically balanced system in which there is a definite relationship between the kind and relative abundance of surface water and other biotic and abiotic elements of the environment. Any imposed condition on such balanced system is sure to bring repercussion throughout the entire environment the system has direct linkage.

Objectives:

The following two are the major objectives of this paper such as (i) to enhance general understanding of the surface water system of Bangladesh and its linkages with the land forming processes, (ii) to examine the state of the physical impacts of flooding, erosion, transportation and deposition of sediments on the environmental hazards of Bangladesh.

Literature review

The methodology of this study is based on searching literatures in this field. This study was conducted as a part of Masters Program in WRE, BUET for preparing a detailed term paper from where it has been extracted.

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Surface water system of Bangladesh

Bangladesh, where two biggest rivers converge on forming the largest delta of the world, is marked by seasonal variations of surface water from over abundance to scarcity. Major components of her surface water system are:

- Stream, rivers,
- Haors, baors and beels
- Estuary and coastal channels
- Canals, ponds and tanks.

Standing water bodies:

Rain, extra-territorial inflow, tides and subsurface flows are principal sources of water to the system. The yearly inundation connects most of the components to a single system for up to four months. A separate system with streams, rivers and lakes exists in Chittagong and the Chittagong Hill Tracts. During dry season (January-February), the surface water system of Bangladesh covers an area of about 11,457 sq. km of which rivers account for about 4,797 sq. km, river and estuary cover 5,518 sq. km and beels make 1,142 sq. km (SPARRSO, 1985). The above estimates excludes ponds (1,625 sq. km), baors (55 sq. km) and coastal water bodies (518 sq. km), and the Kaptai Lake (688 sq. km) (MPO, 1985). Streamflow varies from maximum of 1,02,000 m^3/sec in August to a minimum of 7,030 m^3/sec in February. These are also the outflows to the Bay of Bengal. Inflows from India account for 90% in flood season and 85% in dry season. The excess inflow and the monsoon rainfall over Bangladesh cause seasonal flooding of about 26,000 sq. km annually, but in years of high inflow this may be over 52,000 sq. km. In a typical year 30% of Bangladesh is flooded from major river over bank spill while 5% of the country is inundated by tidal surges in the coastal belt.

Rainfall:

Rainfall in Bangladesh varies widely in both time and space. About 90% of annual rainfall occur in five months (May through September). During other remaining months drought is common. Mean annual rainfall increases rapidly towards northeast of the country to a maximum of about 5,690-mm at Lallakhal. Rainfall also increases towards southeast to about 3,600 mm near Cox's Bazar. Lower rainfall occurs in the west with a low of about 1,110-mm at Chapai Nawabganj. The mean rainfall is about 2,320 mm.

Streams and rivers:

Bangladesh lies at the confluence of three great rivers of the world, the Brahmaputra-Jamuna, the Ganges-Padma and the Meghna, and their numerous tributaries and distributaries. River Brahmaputra-Jamuna, rising in the northern most ranges at the Himalayas in the southwestern part of Tibetan plateau, covers a length of about 2,580 sq. km up to its confluence with the Ganges, and has a drainage area of about 5,80,000 sq. km. River Jamuna is the lower portion of the Brahmaputra, passing through Bangladesh (up to Goalando), has a length of about 300 km and a drainage area of about 47,000 sq. km. Tributaries in Bangladesh are Dudhkumar-Dharla-Teesta and Atrai-Gumati-Karatoa-Hurasagar on the right bank. Distributaries and spill channels are old Brahmaputra, Jhenai, Lohajang, Dhaleswari, and Ghior on the left bank. Principal spill channel on the right bank is Bangali.

The average discharge of the river is about 19,200 m^3/sec with average annual silt runoff 1,370 metric tons/sq. km. The average slope of the Brahmaputra is about 1:11,400; but the gradients in different localities vary considerably from this average value. For example, in Bangladesh the average slope is 1:11,340 for the 60 km reach between Nunkhawa to Kamarjani and 1:12,360 from Kamarjani to Serajganj (106 km). The slope flattens down to 1:27,200 for a distance of 92 km between Serajganj to Goalando (ECAFE, 1966).

River Ganges-Padma originates at the Gangotri with the name Bhagirathi and meets another branch of the Ganges, in the origin, the Alakananda that starts at the Garhwa-Tibetan border. The 2,527 km long Ganges enter Bangladesh about 16 km below Farakka. The total drainage area of the Ganges is about 9,90,400 sq. km of which only 38,800 sq. km lies in Bangladesh.

Tributary Mahananda meets the Ganges on the left bank in Rajshahi district. From the Indian border up to the point where the distributary Mathabhanga takes off from the Ganges on the right bank in Bangladesh, the midstream of the Ganges forms the international boundary between India and Bangladesh. From the mouth of Mathabhanga it flows for about 128 km before it meets the Brahmaputra at Goalando. The combined flow of Brahmaputra-Jamuna and the Ganges takes the name Padma until it meets the Meghna at Chandpur for about 105 km in a southeastern direction. Below Chandpur the river is known as the Meghna.

The average discharge of the river is about 11,610 m^3/sec with average annual silt runoff 492 metric tons/sq. km. The Ganges – Padma displays considerable variations in gradient between reaches. The average gradients for a reach between Allahabad to Benares is 1:10,500, from Farakka (India) to Rampur Boalia (Bangladesh) 1:18,700, from Rampur Boalia to Harding bridge (71 km) and from Harding bridge to Goalando (112 km) 1:28,000. The slope flattens to 1:37,700 for a distance of 1056 km from goalando to Chandpur (ECAFE, 1966).

River Meghna – the third largest river in Bangladesh – drains one of the heaviest rainfall areas of the world. River Barak bifurcates into two rivers near Bangladesh border, e.g., the Surma and the Kushyara, which again join at Markulia above Bhairab Bazar and takes the name Meghna. The Barak-Meghna has a length of about 950-km of which 340-km lies in Bangladesh. Total drainage area of the Meghna at Bhairab Bazar is about 80,200 sq. km of which 36,200 sq. km lies in Bangladesh.

The average discharge of the river is about 3,515 m^3/sec. The river has steep slope while flowing in the hills of India. At flood stages the slope of the Meghna downstream of Bhairab Bazar is only about 1:88,000.

Hydrological region:

The three major rivers divide the country hydrologically and for drainage broadly into four regions. The main rivers and the surface water availability in each region are summarized below (cf. MPO, 1985).

Northwest Region: Hydrologically the Northwest Region can be divided into three sub-regions, e. g.

- Dhepa-Punarbhaba-Tangon-Mohananda basin which drains the western part
- Karatoa-Atrai-Gur Gumani and Deconai Charalkata-Jamuneswari-Karatoa basins which drain central part of the region

- Teesta, Dharla, Dudhkumar and Ghagat basins that drain northeastern portion of the region independently.

Mean monthly available stream flow in the region varies from about 390 m^3/sec in February to about 6,250 m^3/sec in July. Mean monthly inflow from India to the region in February is about 420 m^3/sec. The major shares of stream flow during the driest month (February), are concentrated in the Teesta river (38%), Dudhkumar river (35%) and Dharla river (21%). Available static water is about 167 Mm^3 and in stream storage potential is about 103 Mm^3.

Northeast Region: Broadly speaking rivers in this region may be classified as (i) tributaries originating from the hills which fall into the Meghna river system and (ii) distributaries from the Brahmaputra with their spill channels which also fall into the Meghna river. Mean monthly stream flow varies from about 140 m^3 /sec in February to about 17,910 m^3/sec in July. Mean monthly inflow from India to the region in February is about 229 m^3/sec. In addition, the region receives about 75 m^3/sec from the Brahmaputra River in February. Inflow exceeds outflow in February by about 164 m^3/sec. Major sources of stream flow during the driest month (February) are the Manu-Kushiyara river (52%), Surma river (19%), Dhaleswari river (13%) and old Brahmaputra river (12%). Available static water is about 374Mm^3 and in stream potential is estimated to be about 275 Mm^3.

Southeast Region: Rivers in the region have the characteristics that they originate from the hills and discharge independently into the Bay of Bengal. The principal rivers in the region are Muhuri, Little Feni, Dakatia, Karnafuli, Sangu, Matamuhuri and Bakkhali. Mean monthly available stream flow varies from 400 m^3/sec in March to 4,630 m^3/sec in July. About 87% of the stream flow during the driest month (March) is the augmented flow in the Karnafuli river by the Kaptai reservoir. Other significant sources of stream flow for the month of March are the Gamti River (4%), Feni River (4%), Sangu River (2%) and Matamuhuri River (2%). The stream flow in about 9% of the total area of the region (coastal area) has salinity greater then 2,000 micro-mhos. In stream storage potential is about 57 Mm^3.

Southwest Region: Principal rivers which pass through this area are Gorai-Madhumati, Arial Khan, Kobadak, Kumar, Nabaganga, Bishkhali, Buriswar, Rupsa, Pussur, Sibsa and numerous other tidal creeks and channels in Barishal and Khulna. Mean monthly available stream flow varies from 2,820 m^3/sec in March to 44,000 m^3/sec in August. Over 70% of the available stream flows for the dry season enter the region through the Lower Meghna River. Other significant sources of stream flow during the driest month (March) are the Gorai River (4%) and Arial Khan River (2%) which receive stream flow from the Ganges and Padma Rivers. Available static waters and in stream storage potential is about 70 Mm^3 and 68 Mm^3, respectively.

Main River System: Approximately 85% of the total dry season streamflows in Bangladesh are in the main river system (Brahmaputra-Ganges-Padma-Meghna River). The primary surface water development potential in bangladesh is, therefore, in the main rivers. Mean monthly available streamflow in the Padma River at Baruria varies from 6,110 m^3/sec in February to 76,200 m^3 in August. The streamflow in the Padma River represents the combined streamflow of the Brahmaputra and Ganges River.

Haors, Baors and Beels:

Several large and small depressions (haors/beels) formed during the delta building process are located in larger Bogra, Pabna, Faridpur district and most important of all, in the larger Mymensingh-Sylhet districts. These depressions are zones of active subsidence where deposition by the tributaries and distributaries is not rapid in filling them. One exception to this is the 'Bhar Basin' (Pabna-Bogra), much of which is silting up (Rashid, 1977). There is no agreement as to their origin. Most common belief is that they are simply areas where delta development was left incomplete when silt-laden river shifted to a new location. Strickland (1940) hypothesized that a seaward ledge and its blocking of inland deposition caused the Central Delta Basin (Faridpur Beels). The findings of Morgan and McIntire (1959) indicate a techtonic origin of these beels. They theorised that part of the Delta (Sylhet- Mymensingh) with the multitude of large lake-like bodies has been sinking into its present saucer-shape and is thought to be intimately connected with the rise of the Madhupur Tract (Rashid, 1977). Morgan and McIntire (1959) contended that this basin has sunk at least 10-15 m within last few hundred years and is connected with the earthquake of 1762 that began the diastrophic sinking which is still going on.

Estuary and coastal channels:

Morphologically the eastern coast line of Bangladesh along Chittagong to the south can be classified as a Pacific Type coast running parallel to the young fold mountain ranges, while the rest of the coast line of the country can be termed as Atlantic Type in which the coast line in general is transverse to the continental margin (Kibria, 1975). The nine eastern estuaries in Bangladesh viz. the Hatia Channel, the Sandwip Channel, Matabari Channel, Moheshkhali Channel, the Naaf as well as the estuaries on the much broken Arakan coast of Burma are under the direct Hydrological influence of the Burma Trench, while the fourteen western estuaries, viz. the Shahbazpur Channel, the Tetulia-Tabanabad Channel, the Niloganj, the Buriswar, the Haringhata, the Bhangra, the Pussur, the Kunga or Marjatta, the Malancha, the raimangal and the Hariabhanga (on the Indian border) are physically and hydrologically linked with the Swatch of No Ground. Shallow submerged inner and outer bars resultant from upland sediment supply, tidal current, and waves characterize these estuaries at the present coastline.

Flooding:

Bangladesh is a low lying region characterized by three discernible physiographics. The alluvial plains constituting 80% of the land have a general lay sloping north to south. 65% of the alluvial plains is below 7.5 m with respect to mean sea level. The average fall in elevation is 23 cm in every 1 km. These plains are vulnerable in flooding, in both localized and extensive senses. The remaining 20% of the land consist of Pleistocene terraces (accounting for 8%) and the hill areas, (12%) formed during the time Miocene and Pleistocene are free from flooding. Although floods have been a perennial occurrence in Bangladesh, very little is known about the historic floods. The Imperial Gazette (Bengal, vol II) reports that the river Teesta changed its course following and devastating floods in the districts of Kushtia, faridpur, Jessore and Khulna during the early part of the 18th century are considered to have caused the development of the Gorai-Madhumati.

There are records of major floods in the districts of Rangpur, Bogra and Pabna during the period 1787 through 1830 when shifting of the Brahmaputra course through Jamuna took place. Some of these record floods are considered to be associated with earthquakes causing major physical changes in river courses. Records of the major floods in the western Bogra and Mymensingh are also available. In most cases of these records show that timing of a flood is more important than magnitude, because of the damage caused due to early-untimely arrival of the floods.

Bangladesh is subject to annual flooding. The monsoon or rainy season, June through October, brings on the normal floods each year, inundating 26,000 sq. km of land surface. During more severe floods the areal extent may reach over 52,000 sq. km (MPO, 1985). Statistics and estimates show that in the recent past the devastating floods of 1954, 1955, 1956, 1962, 1964, 1968, 1970, 1971, 1974, 1978, 1980, 1984, 1987, 1988, 1998 engulfed an area ranging from 35,000 to 1,10,000 sq. km. Recent estimates show that 50% of the total land of Bangladesh is vulnerable to floods of one kind or the other. The flood depth to which the land normally inundated varies from 30 cm to 2.5 m.

Depending on the intensity, duration, areal extent and genesis of floods in Bangladesh are categorized into three major groups:

- Monsoon floods: Characterized by relatively low rise, long duration engulfing large areas. Monsoon floods are related principally to the three major river systems in Bangladesh.
- Flash floods: Known by their rapid rises and short duration occurring in response to intensive-localized rainfall. The southeastern regions and the northern bordering areas are subject to flash flooding. Flash flooding in Bangladesh is much less predictable.
- Tidal Inundation: The coastal areas of Bangladesh are subjected to this kind of flooding. Inundation occurs due to storm-tidal surges associated with cyclonic storms and spring tides.

MPO (1985) has identified four main sources of flooding in Bangladesh such as:

- Spill from the Brahmaputra, Ganges and Meghna Rivers;
- Drainage congestion all over except hilly regions;
- Minor river overflows and
- Storm surge (tidal surge).

Flood is a climatically controlled event, and most of its attributes can be found within the drainage basin. For the last three decades several scholars and professionals have raised and explained the reasons of flooding in Bangladesh, and their views may be summarized as below:

Floods of Bangladesh are caused by simultaneous heavy run-off from the catchment areas (due to heavy rainfall and snowmelt), low general topography of the country, high water in estuaries and riverbed siltation. In addition, factors, which may cause increasing flood intensity, are:

- Synchronization of flood peaks of the Brahmaputra-Ganges-Meghna.
- Intensive rainfall directly on the flooded area, or small localized areas.
- High water level of principal rivers retarding the flow of their tributaries.
- Effects of flood control measures (excluding reservoirs) in the river basin.
- Deforestation, urbanization and other changes in the land uses in the catchment.
- Rise of mean sea level during monsoon.

- Earthquake and other tectonic disturbances causing changes in hydrological regime
- Storms (cyclones) associated with tidal bore.

Surface Water and Morphological Aspects

The annual cycle of water in association with three major rivers works have dominated the landscape of the Bengal Basin. Throughout the Quaternary sediments deposited by the Brahmaputra, Ganges and Meghna rivers and their numerous tributaries and distributaries have formed one of the largest deltas of the world. The three rivers combined may discharge over 1,40,000 m^3 during flood. The average annual discharge of the three rivers is about 43,000 m^3/sec to the Bay of Bengal over 16,50,000 sq. km (ECAFE, 1966). These rivers are heavily charged with sediments composed primarily of fine sands and silts with little clay matrix. During floods, the rivers transport a suspended-sediment load of the order of about 13 million tons per day to the Bay of Bengal. Little or no information is available about bed-load transport (Coleman, 1969). However, an estimated annual sediment load of about 2.4 billion tons is also known to be extremely unstable, constantly to adjust their bed configurations to differing flow regimes. The Brahmaputra and the Ganges not only deposit million of tons of sediment but are also highly susceptible to erosion when flow conditions change. They are migratory in nature.

Changing courses of the river

Channel may exhibit different response to changes in surface conditions of the basin depending on the relative magnitude of the change in flow and the change in sediment yield. The large discharge and heavy sediment load cause rivers to be extremely unstable, and channels are constantly migrating. Within recent time, both the Brahmaputra and the Ganges have occupied and abandoned river courses. The Brahmaputra followed a route some 90 km to the east of its present course (Jamuna) only 200 years ago. Many historic documents to the time of this change in the river course. La Touche (1919) suggested that the change was rapid, occurring in 1787 as a result of rapid increases of water flow in the Brahmaputra. Hirst (1916) indicated that the change took place gradually between the years 1720 and 1830. Mahalanobish (1927) agreed with La Touche and claimed that a catastrophic flood occurred in 1787, which caused the Brahmaputra River to abandon its old course. Coleman (1969) demonstrated that the increased flood discharge, faulting, or combination of both caused the major flow to shift from east of the Madhupur Tract to its west.

The Teesta was the most important river of the northwestern region until 1787. It used to be the principal source of supply for the Karatoa, Atrai, and other rivers. Considerable structural changes in the Barind affected Karatoa, which rapidly dwindled. This led to the Teestahaving a considerable amount of water left over, which it could not pass down to Atrai without causing floods. The excessive rains of 1787 brought down a vast flood of sand, which choked the Atrai channel. Finding no sufficient outlet, it overflowed and swept nearly whole of Rangpur district. The floods or rather deluge, happened in a single day (August 27) and killed one sixth of the population of Rangpur in 1787. The Teesta found a new course following the flood of 1787 and it has kept more or less the same course as of today. Similarly, then extensive floods in the districts of Kushtia, Faridpur, Jessore and Khulna during the early part of the eighteenth century are known to have caused the development of the Gorai-Madhumati course to their present locations (Rashid, 1977).

Channel migration

Bank stability depends on the behavior of the river during flood stage and the subsequent fall of the river. In a meandering river, changes and migration patterns are predictable because the river cuts on one bank and deposits on the opposite. Such scouring and bank erosion and deposition are related to the quantum of flow discharge, sediment transport, channel gradient and character of bank materials. In a braided stream, this type of cut and fill doesn't occur. That is both banks may experience erosion or deposition simultaneously. Several factors may be responsible; but Colemn (1969) identified the following:

- Rate of rise and fall of river level.
- Number and location of major channels during flood stage.
- Angles at which the thalweg approach the banks.
- Amount of scour and deposition.
- Formation and movement of large bed forms.
- Cohesion and variability of bank materials.
- Intensity of bank siumping.

Study of maps, airphotos, and recent landsat imagery show that the bank lines of the Brahmaputra-Jamuna are constantly moving towards west. Measurements of the 1963 banklines with respect to 1830 indicate that the right and left banks have moved towards the west by average distances of about 6,000 and 1,200 m respectively. The width of the Jamuna, at the junction with the old Brahmaputra, respectively has increased from 2.25 km to 7.2 km. In 1965 the maximum and minimum widths of the same river were about 16 km and 7.2 km respectively. Recent studies shows that the rate of bank erosion is as high as 12 km in a 4 year period (Tarafder, 1975).

Wandering of the Thalweg

In a river as highly charged with sediment as the Brahmaputra, wandering of thalweg is very common. Deposition of sediment in a locality causes concomitant deepening scour in another place. Such process of deposition and scouring including pool and riffles and other bed forms formation result into continuous wandering of the thalweg in the river. In case of a straight reach of a channel such wandering is quite drastic.

Sand bars and bedforms

The Brahmaputra River is laterally choked with sandbars, mega and micro-bedforms (dunes). These depositional features are directly related to the high rate of sediment transport and sudden change in relative steepness of the channel gradient. Bars and bedforms are highly mobile and related to the changing flow discharge and sediment transport conditions. Coleman (1969) found some of the bars in the Jamuna River near Sirajganj migrated downstream by distances ranging from 800 m to 1,600 m within a year.

Topographic depressions

Topographic depressions in the larger Mymensingh and Sylhet districts are locally known as Haors. The depressions are associated with subsidence in the Bengal Basin usually thought to be controlled by the techtonic activities of the region. Beels are also the topographic depressions in the districts of Faridpur, Pabna and Bogra which are believed to be the areas where delta development was left incomplete when a siltladen river to a new location. All these depressions are the areas of active sedimentation and subsidence.

Estuary and coastal belt

Bay of Bengal in the shallower belt with offshore islands is geologically, morphologically, and hydrologically very active. Sedimentation and at places erosion have been occurring since a long time. A huge amount of sediment enter into the Bay through the rivers, and it mixed up with sea water and some portion gradually gets deposited in the bed of the Bay. Experts recently identified huge sedimentation accretion in the Bay through Landsat imageries and ground trough, and recognized a submerged finger like delta formation moving towards the Swatch of No Ground.

Conclusions and recommendations

Rivers, haors-beels, estuaries, and floods, constituting the surface water system of Bangladesh, carry enormous quantity of water, sediments, chemicals, and biota to the Bay of Bengal. In accomplishing this task the water forms and maintains highly organized and interlocked systems of physical and bio-chemical entities. Details of interactions and interrelations in these systems are quite complex, and it is difficult to visualize many of them simultaneously. Assuming these conditions are prerequisite in our discussion, we have come up with a theoretical framework under the caption *process-response* system analysis. In order to understand the environmental aspects of the surface water system, especially the physical aspects, we need to know certain facts and figures about the morphological and hydrological variables related to the surface water system of Bangladesh. We, therefore, suggest the following future research on:

- The inflow and outflow of sediments (bed-load, suspended-load and dissolved load) including their quantity, properties and seasonal variability.
- Rate of erosion and sedimentation in the alluvial plains, haors-beels, estuaries and coastal belts, etc.
- The cause and effects of channel migration both of major and minor rivers.
- Process of siltation in the channel, at the confluence of tributaries, distributaries, etc.

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STRENGTH CHARACTERISTICS OF SOILS OF SOME REGIONS OF BANGLADESH

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Abstract

The shear strength of soils is an important aspect in foundation engineering such as the bearing capacity of shallow foundations and piles, the stability of the slopes of dams and embankments and lateral earth pressure on retaining walls. In this paper the undrained shear strength characteristics of soils of the four geological zones of Bangladesh namely south-western zone, south-eastern zone, north-western zone and north-eastern zone to effective overburden pressure have been described graphically. The gist data, cohesion and angle of internal friction have been taken from triaxial shear test. In this paper, an attempt has been made to establish the correlation between plasticity index of soils and the ratio of undrained shear strength to effective overburden pressure, which may help the design engineer for designing for structures of construction in the respective zone.

Introduction

Bangladesh is a riverine country having most complex river system of the world. Various problems are caused every year flooding by three major rivers, the Brahmaputra-Jamuna, the Ganges and the Meghna and their tributaries and distributaries. So as for preventive measures against disastrous floods, Government of Bangladesh undertakes so many programs every year for construction of embankments, sluices, pumping station, improvement of water drainage and different large & small water control structures.

The soil properties is an utmost important factor for construction of any structure of which shear strength is the principal engineering property that controls the stability of a soil mass under loads. It governs the bearing capacity of soils, the stability of slopes in soils, the earth pressure against retaining structures and many other problems (Arora, 1992).

Strength properties cohesion (c) and angle of internal friction (ϕ) of soil have been taken from the triaxial shear test results of different development projects in the study areas. The cohesion c and the angle of internal friction ϕ are important are the important soil properties which affect the performance of soil. The tests on soil samples of the above mentioned four zones has been conducted in Soil Testing Laboratory of River Research Institute. From triaxial shear test undrained shear strength and effective overburden

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pressure have been calculated and the plasticity index of the same samples have been taken from Atterberg's limit test. Then an attempt has been made to establish the correlation between the ratio of undrained shear strength to effective overburden pressure (S_u/σ') and plasticity index (I_p). The graphical representation of the results of typical rupture line of undrained shear strength and effective overburden pressure & I_p vs. (S_u/σ') of the soils of different zones and the range of data presented in the paper may help the designer in constructing of the structure in the respective zones.

Literature review

Geological background of soils of Bangladesh

Bangladesh forms a major portion of Bengal basin. Being located close to one of the world's major subjection fault in the north and a major transform fault in the east, the Bengal basin and its adjacent area from one of the most active tectonic regions of the world. Structural activity, primary faulting has significantly influenced the quaternary geology (Morgan and McIntire,1954).

The landscape of Bangladesh is mainly of a monotonous flat plain. From geological point of view, Bangladesh is located at the eastern part of Bengal basin which is an extremely flat delta consists primarily of a large alluvial basin floored with sediments deposited by three major rivers, the Ganges-Padma, the Brahmaputra-Jamuna and the Meghna and their tributaries and distributaries. Most of the part has been subsiding slowly due to tectonic forces responsible for building up the Himalayas and other hilly areas. Due to the iso-static balance of the earth surface some regions have been uplifted e.g. Barind tract and some regions have a distinct trend to subside on the other hand e.g. Sylhet haor areas, Chalan beel, Faridpur beel etc.

The silty clay materials are found extensively in Khulna district.

Distribution of soils in Bangladesh is complex and are usually heterogeneous both in vertical and horizontal direction. Soils consist of wide varieties of material ranging from gravel, poorly graded sand to silt and clay.

From engineering point of view,the classification of ingredients or constituents of soil is of utmost importance. Because different types of soil contain different ingredients in varying amount for which its properties varies (Jahan, 1996).

On the basis of geological formation of soil of Bangladesh, Safiullah, Professor BUET described the soils of Bangladesh with their general characteristics which are given below.

Characteristics of Bangladeshi soil (Safiullah, 1994)

Soil types	General Characteristics
Hill soils	Variable soil types which are function of underlying geology. Frequently sandy clays and clays grade into disintegrated rock at shallow depths.
Raised alluvial terrace deposits	Comprises a relatively homogeneous clay known as Modhupur clay and Barind clay. (LL = 30-80%, I_p = 12-50%). Variable depth underlain by fine to medium uniformly graded sand
Himalayan Piedmont deposits	Mainly sandy silt in higher areas and silty clays in basin areas but often underlying fine sands at shallow depth.
Alluvial flood plain deposits	Locally variable but in general silts and silty clays. Silt size predominant. (LL = 20-50%, I_p = 4-30%). Fine sands abound at depths and close to rivers. Contain mica.
Depression deposits	In the south alternating organic/clay deposits overlying clay at depth. Elsewhere, predominantly silly clays and clays.(LL = 30-40%, I_p= 10-16%).
Estuarine and flood plain deposits	Generally silt and silly clays. Acid sulfate soils found near coast. Organic soils close to surface in some places. Widely varies in consistency and water content.

Where, LL = Liquid Limit, I_p = Plasticity Index

Equation for shear strength established by some soil engineers

Many soil engineers worked on shear strength of soil and developed many equations.

In 1776, Coulomb developed the following equation for the shear strength of soil,

$$S = c + \sigma \tan\phi$$

In terms of effective stress this equation can be expressed as

$$S = c + \sigma' \tan\phi$$

This is called Mohr- Coulomb revised equation.

Where, S = shear strength of soil, c = cohesion
ϕ = angle of internal friction
σ'= effective overburden pressure

In 1957, Skempton developed a relationship between undrained shear strength and plasticity index and it is given by

$$C_u/\sigma' = 0.11 + 0.0037\, I_p$$ --- --- --- --- ---(Ref: Terzaghi, 1967)

Where, C_u = Undrained cohesion intercept or undrained shear strength
σ' = Effective overburden pressure
I_p = Plasticity Index

Methodology

There are various methodologies for determination of undrained shear strength and effective overburden pressure. In this study the data has been taken from triaxial shear test results and the triaxial test has been conducted in undrained condition. Mohr diagram is drawn with the results of triaxial shear test from which the cohesion c and the angle of internal friction ϕ are found out. The undrained shear strength S_u is calculated using Mohr-Coulomb revised equation

$$S_u = c + \sigma' \tan\phi$$

Where, S_u = undrained shear strength of soil
c = cohesion
ϕ = angle of internal friction
σ' = effective overburden pressure

Effective overburden pressure is the difference of normal stress and principal stress and the undrained shear strength is the shear strength of soil in undrained condition. The plasticity index (I_p) is calculated using the following equation by determining liquid limit and plastic limit from Atterberg's limit tests.

$$I_p = LL - PL$$

Where, I_p = plasticity index
LL = liquid limit
PL = plastic limit

Laboratory study and presentation of results

A large numbers of undisturbed soil samples have been tested in soil mechanics laboratory of River Research Institute of different zone namely south-western zone, south-eastern zone, north-western zone and north-eastern zone of Bangladesh .The study area of different zone from where the samples were collected has been shown in **Fig. 1.** The results of undrained shear strength vs. effective overburden pressure has been plotted graphically and presented in **Fig. 2.** The ratio of undrained shear strength to effective overburden pressure vs. plasticity index has been plotted graphically and shown in **Fig. 3.** The range of undrained shear strength, effective overburden pressure and plasticity index of the soils of four zones has been shown separately in **Table 1**. The correlation equations obtained for the soils of different zones have been presented in tabular form in **Table 2.**

Discussion of results

It is seen from the results of the undrained shear strength and effecctive overburden pressure presented graphically in **Fig. 2** that generally the undrained shear strength S_u increases with the increases of effective overburden pressure σ' upto a certain extent. Then the overburden pressure increases but the undrained shear strength remains approximately constant. It is also seen from the graphical representation of S_u versus σ' of soils of different zones that rupture line started and ended at different S_u and σ' values. This is due to the fact the characteristics of the soil of different zones are not homogeneous. From the results of the ratio of undrained shear strength to effective overburden pressure (S_u/σ') and plasticity index (I_p) presented graphically in **Fig. 3**, it is seen that S_u/σ' increases linearly with the increase of I_p. From the graphical representation of ratio of S_u/σ' vs I_p of soil it is also observed that the ratio (S_u/σ') varies even for the same plasticity index I_p. This is due to difference in stiffness and ingredients of the soil.

Table 1 Range of undrained shear strength (S_u), effective overburden pressure (σ') and plasticity index (I_p) of the soils of four zones

Zones	S_u (kN/m^2)	σ' (kN/m^2)	I_p (%)
South-Western Zone	8.46 – 42.26	27.83 – 94.41	11 – 34
South-Eastern Zone	8.44 – 96.74	22.33 – 202.8	7 – 34
North-Western Zone	20.79 – 86.07	45.49 – 173.98	9 – 33
North-Eastern Zone	18.58 – 75.11	41.20 – 159.08	15 - 31

Table 2: Established equations and co-relation coefficient zonewise

Zones	Equations	Correlation coefficient (R)
South-Western Zone	$S_u/\sigma' = 0.206 + 0.0098\ Ip$	0.822
South-Eastern Zone	$S_u/\sigma' = 0.4092 + 0.0031Ip$	0.960
North-Western Zone	$S_u/\sigma' = 0.2484 + 0.0095Ip$	0.726
North-Eastern Zone	$S_u/\sigma' = 0.2866 + 0.007Ip$	0.569

Conclusion

Construction of any structures like building, irrigation canal, drainage channel, dams, embankment etc. are very expensive. Such important and expensive structures do not permit an error in design. A design engineer must have sound knowledge on soil types and its strength characteristics and other soil parameters for the proper design of structure. Although soil types and its distribution is very complex and are usually heterogeneous both in vertical and horizontal direction. However, the results & findings of this paper can play a significant role for planning and design of structure in study area of the respective zones. The graphical representation of some soil parameters and the relationships established for soils of different zones may help the designer in finding out one unknown parameter when other is known in the study area of the different zones. For detail information regarding soils of the respective zones further investigation is necessary.

Acknowledgment

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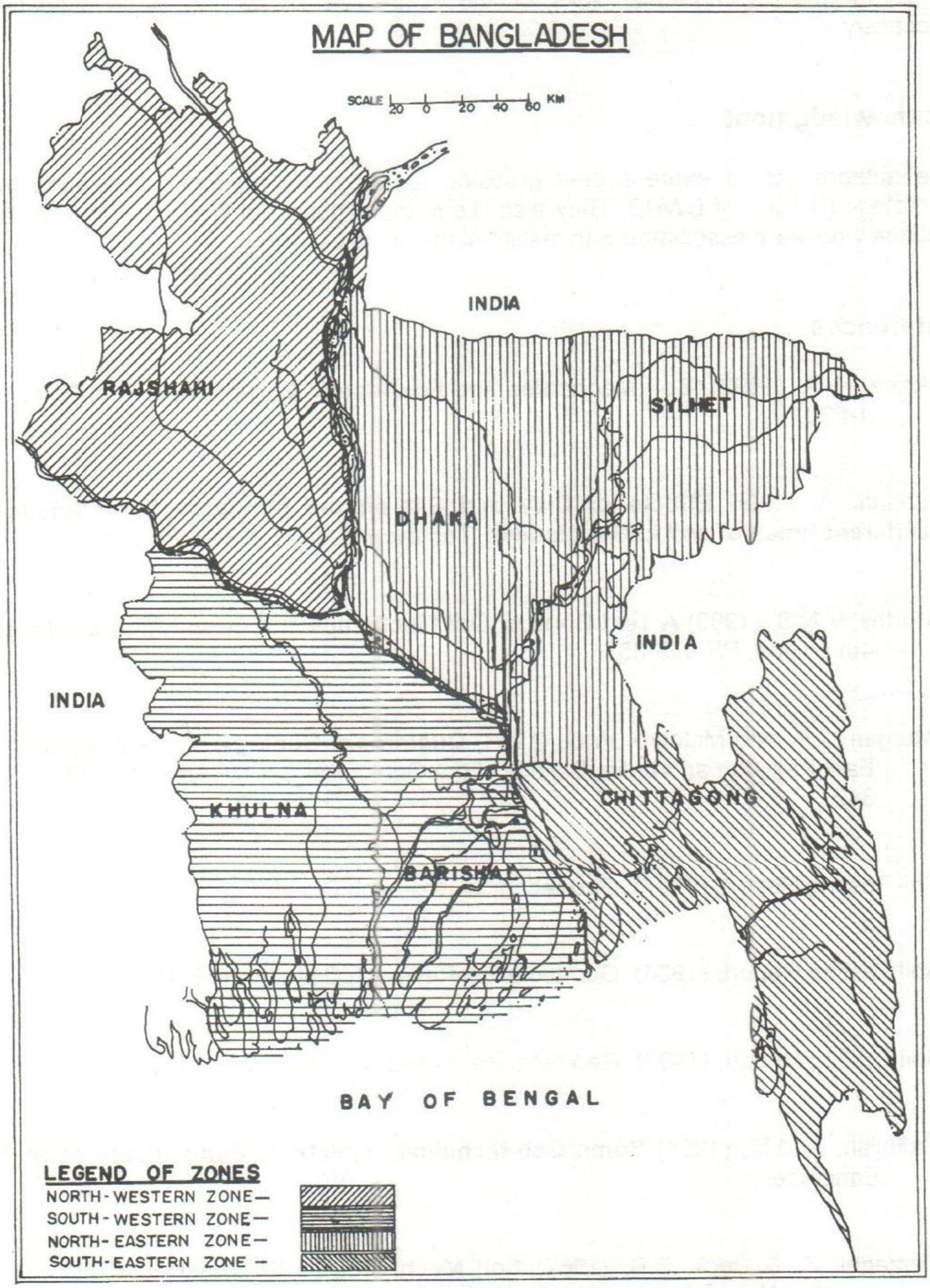

FIG- I : Map of Bangladesh. Different Zones denoted in the map by legend.

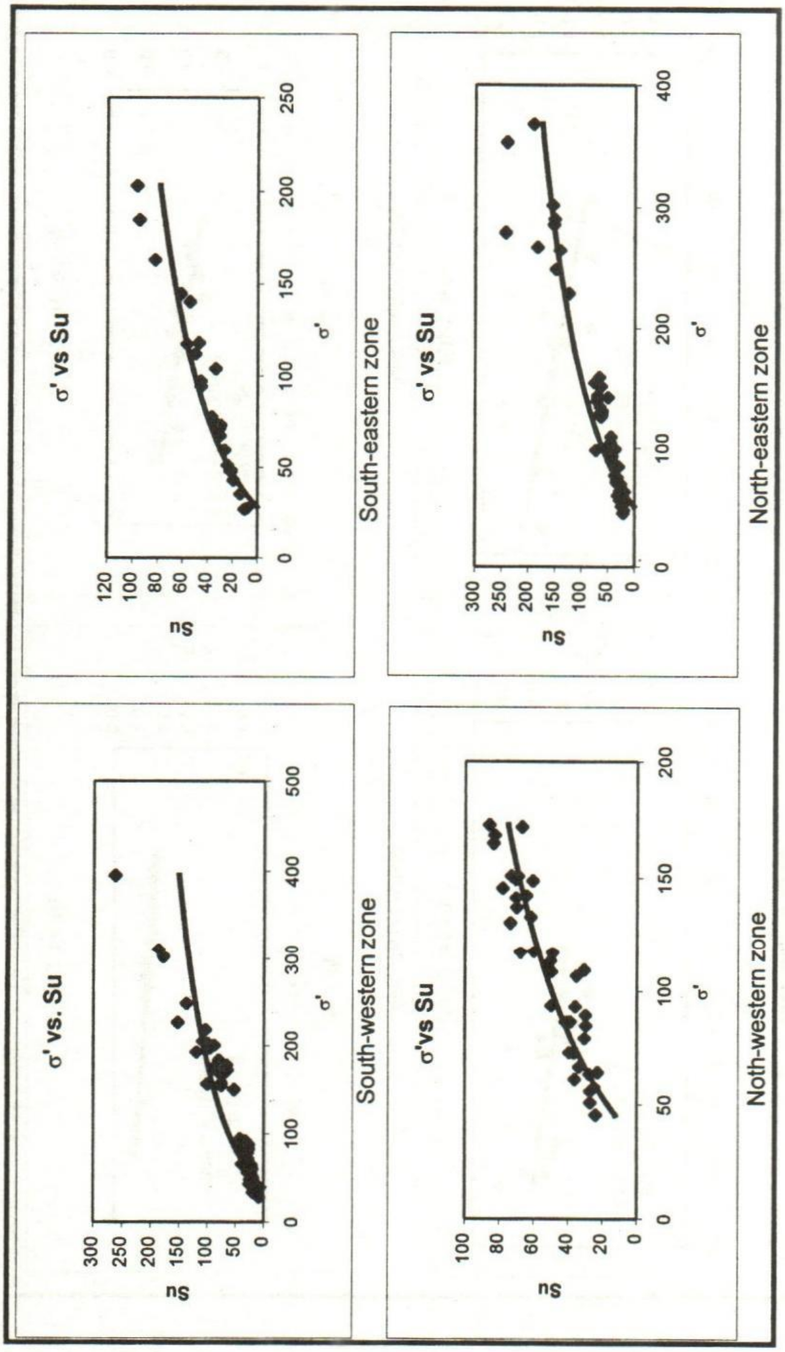

Fig-2 : Typical rupture line of undrained shear strength and effective overburden pressure of soils of different zones

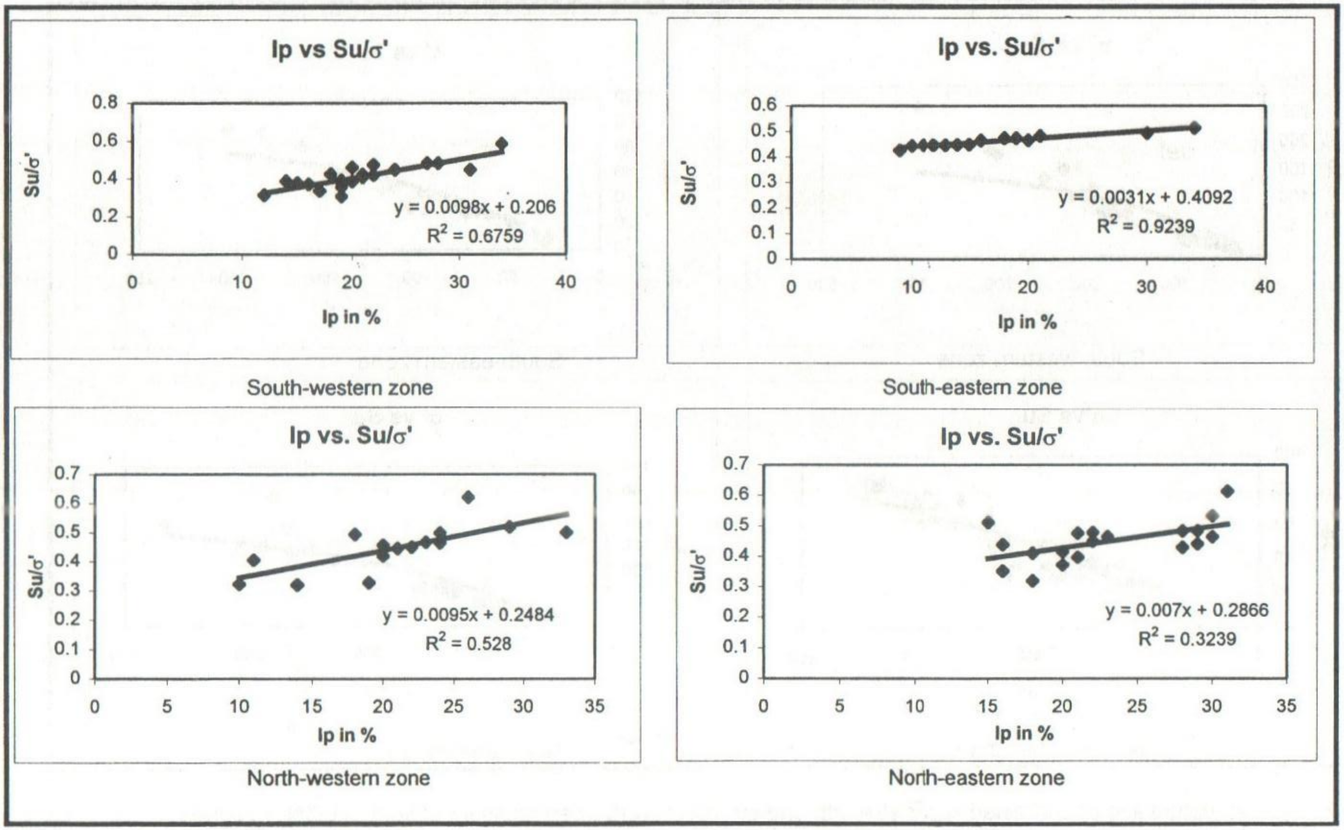

Fig-3 : Relationship between undrained shear strength to effective overburden pressure and plasticity index of soils of different zones

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SIMULATION OF PARAMETERS IN FIXED BED PHYSICAL MODELLING OF BANGABANDHU BRIDGE PROJECT

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Abstract

This paper describes the design features of fixed bed physical modelling. A distorted model was designed and constructed at River Research Institute(RRI) with a view to visualize the river processes in an integrated way and to assess possible impacts of human interventions. To forecast the changes of hydraulic characteristics for a large braided river, fixed bed distorted model could give the results qualitatively. If accurate quantitative data are to be obtained from a model study, there must be dynamic similitude between model and prototype. For this reason, design of model is the principal consideration to the modeller.

Introduction

The Jamuna river is a large braided sand bed river, the number of braids(during low flows) varies between 2 to 3 and the total width of the braided channel pattern varies between 5 km and 17 km. Maximum discharge of the river at Bahadurabad is 100,244 m^3/s whereas minimum and average are 2,427 m^3/s and 20,177 m^3/s respectively.
The valley slope of the river decreases in downstream direction. Near Bahadurabad it is about 8 cm per kilometer, while near the confluence with the Ganges river it is about 6 cm per kilometer. The bed material of the river is fine sand, with D_{50} varying between 0.22 mm at Chilmari and 0.165 mm near Aricha (FAP 24).

Considering the depth and slope of the Jamuna river in view of the particle size, the so-called Shield's parameter is quite high. This parameter characterizes the mobility of the sand. High Shield's numbers also cause the resistance to the flow to decrease. This is achieved through the bed forms that are present on the bed of the channels. Several types of bed form have been identified (Coleman, 1969 ; Klaassen et. al., 1988) but the sand waves are probably the most important one for the resistance to flow of the river bed. According to the investigations carried out in the flood season of 1987 (Klaassen and Vermeer, 1988a) the lee-side of the bed forms is fairly gentle, which explains the high Chezy co-efficients of 70 $m^{0.5}/s$ and higher that have been observed.

Physical models are reproductions of the real river at a smaller scale. In principle, all physical phenomena are active in these models. However, physical models are subject to scale effects because it is not possible to reproduce all physical phenomena at the same scale. For better representation of the model results, dynamic similitude between model and prototype is needed (Warnock, 1949).

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This similitude requires (a) that there should be exact geometric similitude and (b) that the ratio of dynamic pressures at corresponding points should be a constant which may also be expressed as a kinematic similitude.

Similitude requirements

The conditions required for complete similitude may be developed from Newton's second law of motion:

$$M_a = \text{vector sum of forces} = F_p \mapsto F_g \mapsto F_v \mapsto F_t \mapsto F_e \qquad (1)$$

Where, M_a = mass reaction to the acting forces or inertial force

F_p= Pressure force

F_g= Gravity force

F_v=Viscous force

F_t= Surface tension force

F_e= Elastic compression force

For overall similarity the ratio of the inertial or reactive forces, model to prototype, must equal the ratio of the vector sums of the active forces:

$$\frac{M_m a_m}{M_p a_p} = \frac{\left(F_p \mapsto F_g \mapsto F_v \mapsto F_t \mapsto F_e\right)_m}{\left(F_p \mapsto F_g \mapsto F_v \mapsto F_t \mapsto F_e\right)_p} \qquad (2)$$

Where the subscripts m and p represent model and prototype, respectively. Perfect similitude, on the other hand, requires in addition that

$$\frac{M_m a_m}{M_p a_p} = \frac{(F_p)_m}{(F_p)_p} = \frac{(F_g)_m}{(F_g)_p} = \frac{(F_v)_m}{(F_v)_p} = \frac{(F_t)_m}{(F_t)_p} = \frac{(F_e)_m}{(F_e)_p} = \frac{(F_p)_m}{(F_p)_p} \qquad (3)$$

All but one of these ratios must be regarded as independent, that one being fully determined once the others are established. The pressure ratio is usually regarded as the dependent quantity, and hence it does not play a controlling part in similitude techniques.

Although all fluid weights and masses are proportional under the same gravitational conditions, no model fluid is known which also has the requisite viscosity, surface tension and elastic modulus to satisfy the conditions of Eq. 3. Moreover, viscous force is difficult to adjust for complete similitude. In most of the cases, the forces connected with surface tension and elastic compression are relatively small and can be neglected safely.

Froude's number

When gravitational effects predominate, a pertinent basis for similitude can be established by equating the ratio of gravitational forces to that of inertial forces and neglecting the other forces in Eq. 2 becomes:

$\rho L^2 V^2 = \gamma L^3$ when, inertial force, $M_a = \rho L^2 V^2$ and gravitational force,

$F_g = \gamma L^3$ The relationship can be obtained as

$$\frac{V^2{}_m}{g_m L_m} = \frac{V^2{}_p}{g_p L_p} \qquad (4)$$

or, reduced to the first power of V and generalized in form,

$$\frac{V_r}{\sqrt{g_r L_r}}=1 \tag{5}$$

Where, the subscript r indicates model-to-prototype ratio.

The dimensionless quantity $\frac{V}{\sqrt{gL}}$ is called the Froude number F, and the required equality of the number in the model and the prototype is known as the Froude law. The Froude law is applicable in the case of turbulent flow with a free water surface, since the effect of gravity outweigh those of viscosity, surface tension and elasticity. For open channel flow, the Froude law of similitude is more widely applicable than all other types combined.

Reynold's number

When viscous forces predominate, a significant basis for similitude may be obtained by equating the ratio of viscous forces to that of inertial forces and neglecting the other terms in Eq. 2. Representing the viscous force by its equivalent $F_V=\mu LV$, the ratio becomes

$$\frac{L_r V_r}{\left(\frac{\mu}{\varrho}\right)_r} = 1 \tag{6}$$

Where, the $(\mu/\rho)_r$ is the ratio of kinematics viscosity of the model and prototype fluids, also designated by ν_r.The dimensionless quantity LV/ν is the Reynolds number R, and the required equality of the number in the model and the prototype is known as the Reynold's law.

Steady flow through a pressure conduit or flow around a deeply submerged body, generally occurs under conditions to which the Reynolds law is applicable.

Weber number

When surface tension effect predominate, the pertinent similitude law is obtained by equating the ratio of the surface tension forces to that of the inertial forces and neglecting the remaining terms in Eq.2. Upon substitution in Eq.2 of the expression F_t = L, where F_t is the surface tension per unit length, the following ratio between inertia and surface tension results:

$$\frac{V_r}{\sqrt{\frac{\sigma_r}{\rho_r L_r}}} = 1 \tag{7}$$

The dimensionless parameter $\frac{V}{\sqrt{\frac{\sigma}{\rho L}}}$ is known as the Weber number(W). The required equality of the number in the model and prototype is known as the Weber number.

Capillary waves in small channels and capillary movement in soils generally occur under conditions to which the Weber law is applicable.

Mach number

Neglecting all forces other than those resulting from elastic compression permits the derivation of another dimensionless parameter which is known as Mach number. When F_e in Eq.2 is replaced by its equivalent EL^2, where E is the bulk modulus of the fluid, the following expression is obtained:

$$\frac{V_r}{\sqrt{\frac{E_r}{\rho_r}}} = 1 \tag{8}$$

The dimensionless parameter $\frac{V}{\sqrt{\frac{E}{\rho}}}$ is known as the Mach number M, and its required equality in model and prototype is called the Mach law.

Except for cases of unsteady flow, especially water-hammer problems, similitude based on Mach number has little application in hydraulic modelling.

Roughness number

The roughness number offers a means of establishing the same type of roughness action in the model as exist in the prototype, and it is useful in the design of river models.

Selection of model scales

The model scale is based on the existing facilities in the laboratory such as: the capacity of water pumping, the existing water/discharge, the area of out/indoor in the laboratory, number of laboratory staff etc.

Design methodology

Based on available facilities of the laboratory, the geometric scales are selected and the other scale factors are obtained by satisfying the criteria of Froude model law. Since the model bed is fixed, so for dynamic similitude between model and prototype, the criteria for rough turbulent flow and the bed roughness in the model are ensured by changing the governing parameters (viz. Manning's n, Chezy's C and bed roughness, K) as desired. After finalization of design, the model bed is prepared as per bathymetric survey data and roughness is reproduced by placing of simulated bed materials to ensure prototype behaviour. Then the model is calibrated at a representative section and verified with prototype data(i.e. water level, average velocity along the section, roughness, water surface slope etc.). Several trial runs are performed to achieve desired values. The prototype data are collected from Draft Final Report, November, 1997.

Prototype data used for model design

Average water depth	:	12.25 m
Water level	:	14.52 m PWD
Discharge	:	62,500 m^3/s
Water surface slope	:	$7.50*10^{-5}$
Size of bed material(D_{50})	:	0.00019 m
Size of bed material(D_{90})	:	0.00030 m
Average velocity	:	1.95 m/s
Kinematic viscosity of water	:	$8.97*10^{-7}$ m^2/s

Model scale

Horizontal	L_r	:	280
Vertical	h_r	:	80
Velocity ($\sqrt{h_r}$)		:	8.94
Discharge($L_r h_r^{3/2}$)		:	200,350

Procedure of model design

General:

Since the river flows on free surface, so the gravitational effect predominates. For this reason, the model is designed on the basis of fulfillment of the Froude's law. Generally, rough turbulent flow prevails in the rivers and the viscous effects is negligible. But in model, the water depth becomes lower and in some areas the depth is so low that the viscous effects is dominating there. To overcome this viscous effect in the model, rough turbulent flow is ensured. The criteria for rough turbulent flow in model and prototype are described based on Reynold's number. In principle, the Reynold's number should be same in model and prototype. But this is not practicable except with the model scale ratio of 1: 1. However, the turbulent flow is ensured by satisfying the criteria of critical Reynold's number($R_{e(cr)}$= 2000).

Steps in calculation

Froude's number in prototype,	$F_{r(p)}$	:	0.18
Froude's number in model,	$F_{r(m)}$	:	0.18

- Checking of turbulent flow by Reynold's number:

Reynold's number in prototype, $R_{e(p)}$	:	26630434
Reynold's number in model, $R_{e(m)}$	:	37217

The Reynold's number in prototype and model are much higher than the critical Reynold's number for turbulent flow which confirms turbulent flow in both cases.

To verify the rough turbulent flow, the equivalent roughness height is required.

- Roughness in prototype:

A relationship between Q, C and I was established for Jamuna river(Feasibility report of the Jamuna Bridge project, Volume II, 1989) as follows:

$$C = \frac{0.126Q^{0.14}}{I^{0.5}}$$

Where, C is Chezy's co-efficient, Q is discharge and I is water surface slope.

Therefore, the Chezy's co-efficient in prototype, C_p= 68.27

Manning's roughness co-efficient, $n_p=(R_p)^{1/6}/C_p = 0.022$

Equivalent roughness height, $K_p= (26n_p)^6 = 0.035$ m

- Verification of rough turbulent flow in the model:

i) Using brick work with cement mortar
Standard Manning's roughness co-efficient for brick work with cement mortar, $n_b = 0.015$

Equivalent bed roughness height in the model, $K_m= (26\ n_b)^6 = 0.0035$ m

The value of $(K/h)_m$= 0.02285

Equating Darcy-Weisbach and Manning's equation,

Factor for rough turbulent flow in the model, $R_k=2.35*1000*(K/h)^{-7/6} = 193063$

So, the value of R_k is much higher than the model Reynold's number $R_{e(m)}$ which means that the flow in the model will not be rough turbulent. For this reason, the model bed will have to be made rougher with cemented rubble masonry.

ii) Using cemented rubble masonry

Standard Manning's co-efficient for the cemented rubble masonry, n= 0.025

Equivalent roughness height, $K_m= (26^*\ n)^6 = 0.0754$ m

The value of $(K/h)_m$= 0.4928

Factor for rough turbulent flow in the model, $R_k=2.35*1000*(0.4928)^{-7/6}$= 5365.58

The value of $R_{e(m)}$ is much higher than the value of R_k , which confirms rough turbulent flow in the model.

Checking of model roughness by $(K/h)_r= (h_r/L_r)^3$

The value of $(K/h)_r=(K/h)_p/(K/h)_m = 0.002857/0.4928= 0.0058$

The value of $(h_r/L_r)^3 =(80/280)^3 = 0.0233$

The value of $(K/h)_r$ and $(h_r/L_r)^3$ are not equal. Hence the model roughness does not match with that of prototype. Though the roughness height corresponding to the rubble cement masonry produces rough turbulent flow in the model, the simulation criteria of the model roughness by $(K/h)_r= (h_r/L_r)^3$ is not satisfied. Therefore, rubble masonry should be further smoothened. Now, the model roughness will have to be simulated satisfying this equation with checking of rough turbulent flow in the model.

iii) Simulation of model roughness

The value of $(h_r/L_r)^3 = 0.0233$

The value of $(K/h)_p = 0.002857$

We have, $(K/h)_r=(K/h)_p/(K/h)_m = 0.002857/(K/h)_m = 0.0233$

So, $(K/h)_m = 0.122$

Now, the roughness height, K_m=0.0187 m

Manning's roughness co-efficient, $n_m= (0.0187)^{1/6}/26= 0.0198$ which equals for brick khoa.

Chezy's co-efficient in the model, $C_m=18 \log(12R_m/K_m) = 18 \log (12*0.153/0.0187)=35.86$

The height of the model roughness should be 0.0187 m and the corresponding Manning's and Chezy's co-efficient will be 0.0198 and 35.86 respectively.

Checking of roughness height in the model by $K_r=(h_r)^4/L_r^3$

The value of $K_r=K_p/K_m= 0.035/0.0187 = 1.87$

The value of $(h_r)^4/L_r^3 = 1.87$

So, the value of K_r is equal to the value of $(h_r)^4/L_r^3$ which confirms that the model roughness is matched with that of the prototype.

Checking of rough turbulent flow in the model

The value of $(K/h)_m$ equal to 0.122

The value of factor $R_k = 2.35*1000*(0.122)^{-7/6} = 27351.51$

The value of $R_{e(m)}>R_k$, which confirms rough turbulent flow in the model.

Checking of distortion factor in the model

Upper limit of $(K/h)_m = 0.122$

The value of $(K/h)_p = 0.002857$

The distribution factor $[(K/h)_r^{1/3}]= 0.286= 1/3.5$

Hence, the distortion factor is found 3.5.

The key design parameters as obtained from simulation is shown in Table 1.

Table 1: Simulated parameters for fixed bed model

Parameters	Unit	Prototype	Model	Scale factor
Average Water Depth	m	12.25	0.153	80
Design discharge	m^3/s	62,500	0.312	200350
Average Velocity	m/s	1.95	0.218	8.94
Relative Roughness (K/h)	m	0.002857	0.122	0.0233
Chezy's Co-efficient, C	$m^{0.5}/s$	68	36	1.87
Manning's Co-efficient, n	$s/m^{0.33}$	0.022	0.0198	1.11
Water Surface Slope		0.000075	0.000262	1/3.5

Model Calibration and Verification

Based on the simulated design parameters, the model was constructed. For verification of the measured prototype data, the model was calibrated with the discharge of 62,500 m^3/s. After final calibration, the results obtained from the model is shown in Table 2.

Table 2 : Comparison of model and prototype data after calibration

Parameters	Unit	Prototype data	Observed value in model	Scale factor	Observed value in Prototype
Average water depth	m	12.25	0.151	80	12.08
Design discharge	m^3/s	62,500	0.302	20035 0	60,506
Flow velocity	m/s	1.95	0.225	8.94	2.01
Relative roughness height (k/h)	-	0.002857	0.117	0.023	0.0027
Chezy's Co-efficient(C)	$m^{0.5}/s$	68	37.01	1.87	69.2
Manning's co-efficient(n)	$s/m^{0.33}$	0.022	0.0181	1.11	0.020
Water surface slope	-	0.000075	0.00063	1/3.5	0.00018

Table 2 shows a good agreement between model and prototype parameters. However, average water depth and discharge were found to be lower by 2% and 4% respectively in the model. Average velocity increased by 3% as the water surface slope was found 2.4 times steeper in the model. The compliance of the other parameters are satisfactory.

Conclusions

Though the fixed bed physical model gives indicative and qualitative results, but nevertheless it gives quick solution of the problems for large river like Jamuna. For accurate forecasts, the model parameters should be simulated in such a way that the model gives optimum similitude of the prototype. Hydrodynamic parameters such as change in water level, velocity and discharge distribution through the individual channels could be predicted by fixed bed physical modelling within a very short period of time. These parameters will give an idea to the contractors during the construction of a large bridge.

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