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TECHNICAL PAPERS

CONTAMINATION OF ARSENIC IN GROUND WATER AND SOME TECHNIQUES FOR REMOVAL OF ARSENIC

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Abstract

Contamination of arsenic in ground water is our national problem now. During the last decade a large number of tube well have been drilled throughout the country by Department of Public Health Engineering and other organizations with the financial aid of the UNICEF and many other donors for drinking water to save the people from the water born diseases. Now this drinking water has become poisonous by the pollution of arsenic. In this paper different sources of arsenic contamination in groundwater have been discussed. This paper also provides the information about the arsenic investigation situation of Bangladesh. Some techniques for removal of arsenic developed by different countries of the world have been presented in this paper which may be applied in the arsenic polluted areas of the country.

In this paper the technique developed by River Research Institute based on Iron Removal Plant has been described which will be helpful to the people of the country in constructing such type of plant for removal of iron as well arsenic to some extent in arsenic polluted area. Finally some recommendations have been kept in the paper which are very essential in order to tackle the arsenic problems.

Introduction

High concentrations of arsenic in ground water in Bangladesh as well as in neighboring West Bengal have become a serious problem in recent years. Implementation of major tube well – drilling-programs over the last decade have resulted in large populations relying on ground water. As a result the evidences of chronic health problems due to ingestion of arsenic polluted ground water are accumulating day by day. About 1 (one) million people of West Bengal are thought to be at high risk of arsenic. In Bangladesh all the districts adjoining the West Bengal border are affected by arsenic. Based on existing arsenic data in ground water and on the regional distribution of unconsolidated sedimentary aquifers, about 23 million people of 30 districts are at high risk due to consumption of arsenic contaminated water [Smedley, 1997]. The Daily Inqilab reported in editorial on the basis of latest information with reference to a workshop held in Dhaka, Specialists and Scientists opine that the arsenic has contaminated in the ground water of 59 districts out of 64 districts of Bangladesh (Editorial, The Daily Inqilab 25th March, '99). In that workshop the specialists and Scientists also mentioned that now arsenic is our national problem and under this situation water of each of the 40 lacs of tube wells of the country should be tested urgently for determination of the level of arsenic. The environmental Scientists and Specialists has given special emphasis for supply of arsenic free drinking water to the Government.

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In a five-day international conference on "arsenic contamination of ground water" arranged by Dhaka Community Hospital and School of environment studies, Jadavpur University Calcutta held in Dhaka on 12th February, 1998, the experts and scientists opine that the arsenic contamination in ground water in Bangladesh is possibly the largest mass poisoning case in the world now. In this conference many experts and specialists from home & abroad participated and many papers were presented on different aspects of arsenic contamination of ground water and its impact on human health. They mentioned that the areas vulnerable to arsenic contamination are the Ganges flood plain, the tidal region, the coastal plain and the Meghna flood plain. The experts suspect that people of 42 districts out of 64 districts in Bangladesh are at high risk of arsenic pollution, (Majumder, 1998).

In a inter-ministerial meeting arranged by the Ministry of Local Govt. the specialists comment that in Bangladesh arsenic has appeared as epidemic form. According to latest survey report of the Government about 20 million people have been suffering from the toxicity of the arsenic and 70 million people are at high risk of arsenic contamination. This meeting was called for the preparation of the coordinated action plan to over come the complex arsenic problem of the country. In a report prepared by the Ministry of Local Govt. it is suspected that arsenic may accumulate in crops from the water of deep tube well used in irrigation. It is quoted from the survey report of the Government arsenic affected area in the country is increasing every year. In the last year arsenic was found (detected) in tube well water of 17 P.S. of 44 districts out of 64 districts but in this year arsenic has been found in tube well water of 211 P.S. of 59 districts. In this inter-ministerial meeting held in the Ministry of Local Govt., the experts and specialists expressed that the deep tube well which were drilled for drinking water, arsenic has also been found in them. In this meeting emphasis was given not to drill any more deep tube well in country (Salam Jabayer, 99).

Sources of arsenic contamination

The scientists, experts and the specialists opine that real source of arsenic contamination in ground water is the geological formation. But still no comprehensive investigations or studies is underway in Bangladesh about the source of contamination of arsenic in ground water. From the assignment report (Dave 1997) on "Arsenic contamination of drinking water in Bangladesh" and the data from the geological investigations in area adjoining western border in West Bengal show that ground water of three aquifers are in use. These are the shallow one (about 30 m), the intermediate one (40-80 m) and the deep one (below 100 m).

Arsenic was observed to be deposited in first aquifers as adsorbed primary metal on sand grains of biotite and quartz with a few scattered grains of arsenopyrites. Arsenic (leached out of the first aquitard) appears to be confined to the intermediate aquifer (30 – 45 m depth). The unscientific and unintentional development of ground water from this aquifer through uncased perforation of the first and also the second aquitard has inter connected the aquifers with transfer of arsenoferrous water to both upper and lower ones. This is a probable explanation of the appearance of arsenic in ground water of West Bengal, India. This explanation may also be applied in case of contamination of arsenic in ground water of Western border of Bangladesh as the same alluvial formation also extends to the western border of Bangladesh. Some specialists opine that the arsenic problem in ground water particularly in southern region of Bangladesh is probably due to arsenic rich deltaic deposition and hydraulic connection with contaminated region of West Bengal province of India. But from the analysis of initial studies, it is clear that the main areas of contamination are the Ganges dependent areas of Bangladesh. The survey program by UNDP, DFID and the World Bank to identify the source has been under taken (Rasheed, 1998).

Arsenic and its compounds are mobile in the environment. Weathering of rocks convert arsenic sulphides to arsenic trioxide which enters the arsenic cycle as dust or by dissolution in rain, river or ground water. Human exposure of arsenic occurs primarily from air, food and water. Arsenic in air is high near smelters or power plants that burn out fuel with high arsenic content. Marine foods like crabs, lobsters shrimps contain arsenic about 10 – 40 mg/kg, but the fresh water fish contains less arsenic. Ground water contains highest arsenic, especially where geo-chemical condition favors arsenic dissolution. From the report published by the center for study on Man and Environment, Calcutta based on studies carried out by various scientific organizations, it transpires that the arsenic contaminated ground water in West Bengal is limited in the intermediate aquifers of the rural areas of Upper Delta Plain (UDP) of the Ganges (Guha mazumder,1996). The source of arsenic is considered to be geological, as long duration pumping tests conducted in some of the tube wells, tapping the affected aquifers, showed almost similar level of arsenic even after 8 hours of pumping, bringing out huge quantity of the material. It is detected that this arsenoferrous belt lies within the Upper Delta Plain (UDP) of the Ganges and it is characterized by a series of meander belts formed by the rivers. The UDP has been built up of sediments deposited by a succession of meandering stream, one superimposed/ cutting across the other. It is generally being accepted that the source of arsenic in the ground water in the affected areas is the sediment themselves.

The impersistent silty clay separating the intermediate from shallow aquifers is characterized by relatively high arsenic content with sand grain coated arsenopyrites. The change of geo-chemical environment due to high ground water withdrawal has been thought by some to cause decomposition of arsenopyrites and solubilisation of arsenic in the subsurface water. However it is difficult to say whether the hydrological imbalance either due to heavy ground water withdrawal for irrigation or seasonal water level fluctuation has any relation with process of arsenic contamination in ground water as is being expressed by some geologists(Guha Mazumder, 1997). The erratic distribution of arsenic in ground water in the affected areas points to complex geological situation. It is essential to delineate as accurately as possible the area-specific regional stratrigraphy, indicating aquifer disposition and geometry from available information and thus build up the platform for more detail scientific studies on arsenic pollution process in the nature. But for detection of arsenic no comprehensive study has yet been undertaken with multidisciplinary approach to understand this geology, geo-hydrology and geochemistry of the affected area of the country.

Arsenic investigation situation in Bangladesh

Recent surveys of ground water from deep and shallow tube wells across Bangladesh have revealed the levels of arsenic contamination above accepted WHO norms. The magnitude of the risk to public health has created great concern and new surveys by

UNDP, DFID and the World Bank are being under taken to identify the source, extent and degree of contamination. Estimate vary but from 37 to 76 million people may be affected to some degree by arsenic contamination of shallow and deep tube wells supplies (Rasheed, 1998). The Govt. of Bangladesh is trying to find out the real cause of arsenic contamination and means to mitigate the problem of arsenic. For this, a national steering committee has been formed to investigate the arsenic situation in Bangladesh. The committee has identified the following areas of actions (Khan, 1997).

1. Detection of arsenic affected areas.

2. Investigations of sources of arsenic in affected areas.

3. Water analysis for detection of contamination and provision of alternate sources of water in the arsenic polluted area.

4. Development of appropriate technique for removal of arsenic from drinking water.

5. Treatment of affected people and measures for prevention of arsenic toxicity.

6. Development of public awareness about the adverse effect of arsenic.

The Departments/organizations which are responsible for implementation of the above action plans have been mentioned below.

- NIPSOM, DCH are carrying out the responsibility of detection of arsenic cases, laboratory investigations of the patients, measures for prevention of toxicity and health awareness in the affected area. The Disaster Forum, an NGO, The Bangladesh Center for Advanced Studies (BCAS) and a Dhaka based environmental "Think – Tank" on the environmental policy are also carrying out the program of public health awareness about the adverse effect of arsenic.

- DGHS, BAEC, GSB, DU, JU, RRI, DPHE and LGED have been implementing action plan of water analysis for detection of contamination.

- The program of detection of arsenic affected areas by collecting and testing water samples and provision for alternate source of drinking water in the Polluted area is being implemented by DPHE with the financial aid of World Bank. Water analyses for detection of arsenic are also being carried out by NIPSOM, BAEC, GSB, DU & RU (Geology department), RRI and BCSIR.

- The health aspects of arsenic problem are being investigated by NIPSOM, DCH, DGHS.

- BAEC, Ground water circle of BWDB, and DOE are investigating the sources of arsenic contamination in ground water.

- JAICA and the Asian Arsenic Network in Japan have also shown keen interest in investigation and mitigation of arsenic problem is Bangladesh (Smedley, 1997).

In a inter-ministerial meeting recently held in the ministry of Local Govt. for preparing a coordinated action plan on arsenic investigation situation in Bangladesh, the experts and specialists pointed out that the contamination of arsenic in ground water has been found in 59 districts out of 64. One of the Govt. officials reported that water of the seventy thousand shallow tube wells are tested for determination of arsenic out of 45 lacs shallow tube wells in the country. The existence of arsenic is found in water of 63% of the seventy thousand tube wells.

Techniques for removal of arsenic

Removal of arsenic from ground water is very complicated problem. Various countries of the world have carried out research and study on removal of arsenic from ground water and developed some techniques or methods for removal of arsenic. Some of them applied in the affected areas are mentioned below

Method of co-precipitation (developed by AIIH & PH, India)

The common valency of arsenic in raw water source is +3 (arsenite) and +5 (arsenate). The effective removal of arsenic from water requires the complete oxidation of As +3. The free chlorine, hypochloride, ozone, permanganate etc. may be used as oxidants. For availability of bleaching power solution, it can be used as appropriate oxidant for oxidizing As^{+3} to As^{+5} in process of removal of arsenic. Since there is some residual effect of chlorine in water, so the dosage of chlorine should be within 0.5 mg/l. Chlorine dose also take care bacteriological contamination, if any during the water treatment process.

The next step of co-precipitation process is coagulation – flocculation.

Aluminium and ferric salts are used in drinking water treatment for coagulation-flocculation. Both the metal salts under go hydrolysis to various products, but can be reduced to a very low residual if the poorly soluble hydroxides are formed at the proper P^H and can be filtered off completely.

Dissolved substances such as heavy metals, phosphates and humic substances may also be precipitated directly. AIIH & PH indicates that alum salt and ferric salt should be used 30 – 40 mg/l and 20 – 40 mg/l respectively for coagulation. The addition of coagulants should be carried out by rapid mixing for 60 seconds followed by very slow mixing for a couple of minutes for development of flocs. The flocs are then allowed to settle at the bottom. The supernatant water is then filtered through selected filtering media. The filtering media used by AIIH & PH was domestic candle, gravel and sand-gravel. The comparative study using alum and ferric salt indicates that ferric salt has slightly more edge over aluminum salt for removal of arsenic. While the arsenic removal efficiency with application of Aluminium salt is 95%, the same with ferric salt may be as high as 99%.

The performance of alum in removal of arsenic in co-precipitation method monitored by AIIH & PH for a ground water sample containing 0.335 mg/l of arsenic has been shown in **Table 1.**

Table 1: Performance of alum in removal of arsenic

Initial concentration of Arsenic in raw
Ground water = 0.335 mg/l. Initial P^H = 7.1

Dosage of Alum (mg/l)	Control of P^H by adding lime	Conc. of Arsenic after treatment	% of removal of Arsenic
10	7.5	0.11	67.16
20	7.3	0.07	79.10
30	7.2	0.05	85.07
40	7.2	0.03	91.04
50	7.3	0.02	94.02
60	7.2	0.01	97.00

The performance of alum in arsenic removal by co-precipitation method (using chlorine for oxidizing As^{+3} to As^{+5} and alum solution) monitored by AIIH & PH has been shown graphically in **Fig.1**.

The performance of ferrous sulphate for removal of arsenic was also monitored by AIIH & PH. Initially bleaching powder was added to convert arsenite to arsenate in the raw water. Lime (CaO) was added to maintain P^H within optimum condition. The performance of ferrous sulphate as coagulant dose in co-precipitation of arsenic has been shown in **Table- 2**

Table 2 : Performance of Ferrous sulphate as coagulant for removal of arsenic.

Raw water arsenic content (mg/l)	Bleaching powder as chlorine dose (mg/l)	Ferrous sulphate (mg/l)	Lime CaO (mg/l)	Arsenic Content after treatment (mg/l)	% Removal
0.21	0.50	10	2	0.14	33
0.21	0.50	20	4	0.06	71
0.21	0.50	40	8	0.03	86
0.21	0.50	80	16	0.01	95
0.21	0.50	120	24	Nil	100

Rapid and slow mixing after addition of coagulant are important for development of flocs. A minimum detention time of 30 min. is essential prior to filtration.

The royal Duch Government – aided water supply program is also being implemented in the arsenic affected district. This program covers the drilling of deep-tube wells for arsenic free water, where possible. An arsenic removal treatment plant for Meherpur town is also being constructed. The method used for iron removal is based on the precipitation of arsenic with P^H change, using lime and alum as coagulants. The plant is expected to have a 95% removal rate and the resulting sludge is to be stored in an impervious concrete tank with storage capacity lasting 50 years. Though it may prove prohibitively expensive on a large scale, yet the plant is under implementation.

Technique developed by RRI for removal of arsenic

River Research Institute is situated in Faridpur. It is a flood plain of the Padma river and composed of alluvial soil. The ground water of the Compound contains higher degree of iron and arsenic and it is not suitable for drinking or domestic use.

To develop a technology for removing iron from ground water, Ground Water Utilization Division of RRI, takes up a research program. The iron rich deep tube well of RRI is selected to conduct the experiments, where average iron and arsenic concentrations were 8.5 mg/l and (.20-.15) mg/l respectively. For removal of iron from ground water the method of aeration followed by filtration and sedimentation is utilized. A brick-masonry three stage horizontal flow roughing filter is constructed in this research project. A four-stage cascade type aerator was also constructed in this iron removal plant as shown in **Fig. 4**. The brick-chips are used as filter bed material in all filter horizontal beds. There are buffer zones after each roughing filter to facilitate sedimentation and to avoid carrying of sludge into the next filter bed. A series of experiments are conducted by changing discharges, number of filter bed and the grain size of the bed materials to reduce the iron concentration within the allowable limit for drinking purpose. The average iron concentration reduces from 8.5 mg/l to 3.6 mg/l at the end of the horizontal roughing filter and it further reduces to 1.4 mg/l after filtration through the up flow filter (sand). The results of the experiment show that the average concentration of arsenic reduces from the range (.20-.15) to(0.06- 0.05) mg/l. It is possible from this iron removal plant to obtain arsenic free water by applying co-precipitation method with p^H change using lime and alum as coagulant. But in case of supply in large scale it is very costly.

Discussion

From the **Table 1** and **Table 2** it is seen that it has been possible to remove arsenic up to 97% and 100% from water samples containing arsenic 0.335 mg/l and 0.210 mg/l using Aluminium salt and ferric salt respectively as coagulants. It is also seen that dosage of coagulant depends on the initial concentration of arsenic in the water. The P^H control is necessary during the process of co-precipitation. Generally the P^H range between 7-8 is considered to be ideal.

The performance of arsenic removal by co-precipitation method adding chlorine for oxidising As^{+3} to As^{+5} and alum as coagulant has been shown graphically in **Fig. 1**. From the graph it is clear that the amount of arsenic content decreases with the increase of the dosage of alum.

The method of co-precipitation developed by All India Institute of Hygiene and Public Health (AIIH & PH) was applied in case of G.I. Sand-Gravel and Candle domestic filter developed by the same Institute. The Sand-Gravel G.I. domestic filter as shown in **Fig. 2** can produce 300 liters/day arsenic free water. Similarly domestic filter fitted with two candle filters as shown in **Fig. 3** can produce 30 liters/day arsenic free water. The iron removal plant for the water supply scheme can be utilized for arsenic removal and in this case alum salt can be used as coagulant. AIIH& PH carried out experiment for removal of arsenic utilizing iron removal plant for water supply scheme. In the experiment alum was used as coagulant. The experiment showed encouraging result. It is possible to supply 36000 litres/ day arsenic free water from this plant. It is reported that the plant is still running satisfactorily (Nath K.J.& others, 1997).

The lay out plan of the iron and arsenic removal plant developed and adopted by RRI has been shown in **Fig. 4**. The water of RRI deep tube well contains iron 8.5 mg/l and arsenic ranging from (0.20-0.15) mg/l which is beyond the permissible limit of WHO guide line value. It has been made possible through this plant to reduce iron from 8.5 to 1.4 mg/l and arsenic from the range (0.20-0.15) mg/l to (0.062-0.05) mg/l. It is possible to reduce more this level of arsenic by applying co-precipitation method developed by AIIH and PH, India. But for supply in large scale this is very expensive.

Conclusion and recommendation

Arsenic is now our national problem. About half of the population of the country is at the risk due to the presence of excess arsenic in ground water. The implications of public health are critical. As more information becomes known it is likely that villagers will revert to untreated pond and river water, exposing themselves and their children to the renewed risk of diarrhoeal disease. So immediate necessary step should be taken to find out the exact solution of the problem. Now it is very urgent to test water of the 45 lacs shallow and deep tube wells of the country for detection of arsenic. The following recommendations can be kept in order to tackle the arsenic problem.

- Removal of arsenic from groundwater is very complicated and long term process but in the interim appropriate treatment methods may be implemented in affected areas.
- Alternative source of water should be found out for prevention of arsenic pollution problem.
- Necessary steps should taken for treatment of surface water to make it suitable for drinking.
- In the affected area treatment plant should be constructed for removal of arsenic in order to supply arsenic free water.
- Use of domestic filter developed by AIIH & PH, India should be encouraged in the arsenic affected area.
- As far as possible, tube wells for drawing water should be installed in arsenic free aquifer in the affected area.
- In the affected area substitute supply either from deeper aquifers or from surface water must be accorded on top priority basis.
- During construction of tube well proper sealing should be done so that arsenic from higher aquifer does not enter into the lower aquifer.
- In arsenic affected areas if possible piped water supply scheme may be taken up to supply treated water from surface water source.

- In the area where arsenic level is beyond the permissible arsenic removal plant based on Iron removal plant using horizontal flow roughing filter may be constructed.

- Manufacturing of domestic filter for removal of arsenic by co-precipitation method should be encouraged.

- In order to identify the sources of arsenic in affected areas drilling is necessary to investigate the geochemistry and hydrology of the affected aquifer.

- From the studies of the various research workers belonging to different Scientific Institutions it may be concluded that the sand grain of the arsenic infested aquifer are generally coated with iron and arsenic rich material.

- The pattern of ground water arsenic content in the identified area needs to be studied intensively in order to determine seasonal variations and their trend over a period.

- A system of periodic monitoring of arsenic content in the tube well in and around the known arsenic polluted areas should be developed in order to check if arsenic incidence is spreading new areas.

- Arsenic along with other parameters and sampling or aquifer provenance of ground water should be determined in order to investigate source and mobility of arsenic.

- Immediate step should be taken to set up more laboratories for determination or detection of arsenic.

- News media should take necessary steps to aware the people about the adverse effect of arsenic through advertisement so that people understand the problem and come forward to combat it.

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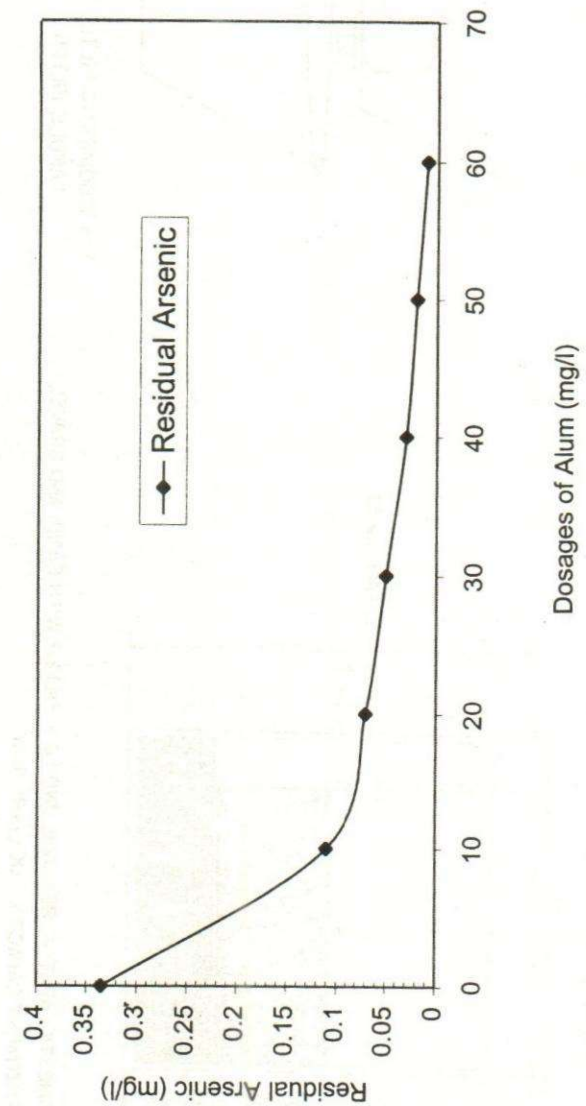

Fig 1 : Arsenic Removal by Co-Precipitation

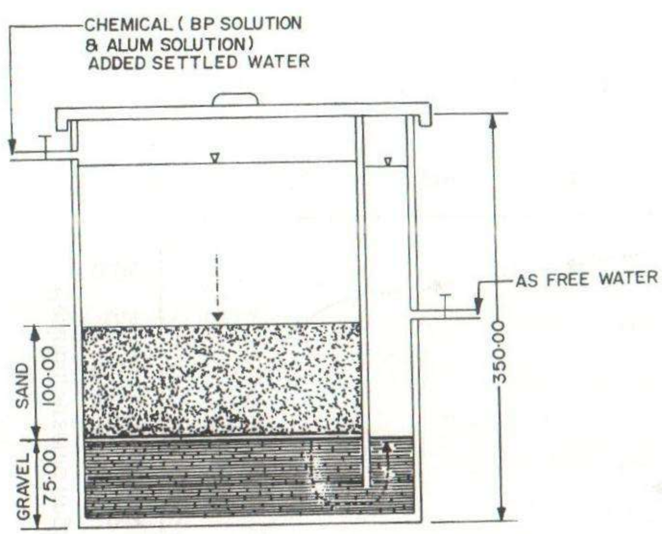

FIG.2: DOMESTIC ARSENIC REMOVAL UNIT (G.I. FILTER WITH SAND AND GRAVEL).
TREATMENT CAPACITY 300 LITRE/DAY

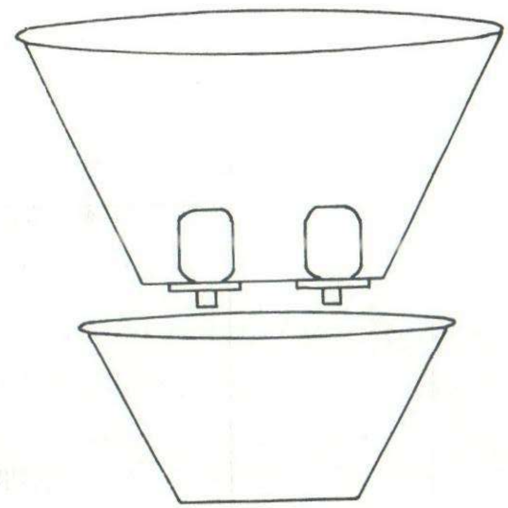

FIG.3: DOMESTIC FILTER FITTED WITH TWO CANDLE FILTER

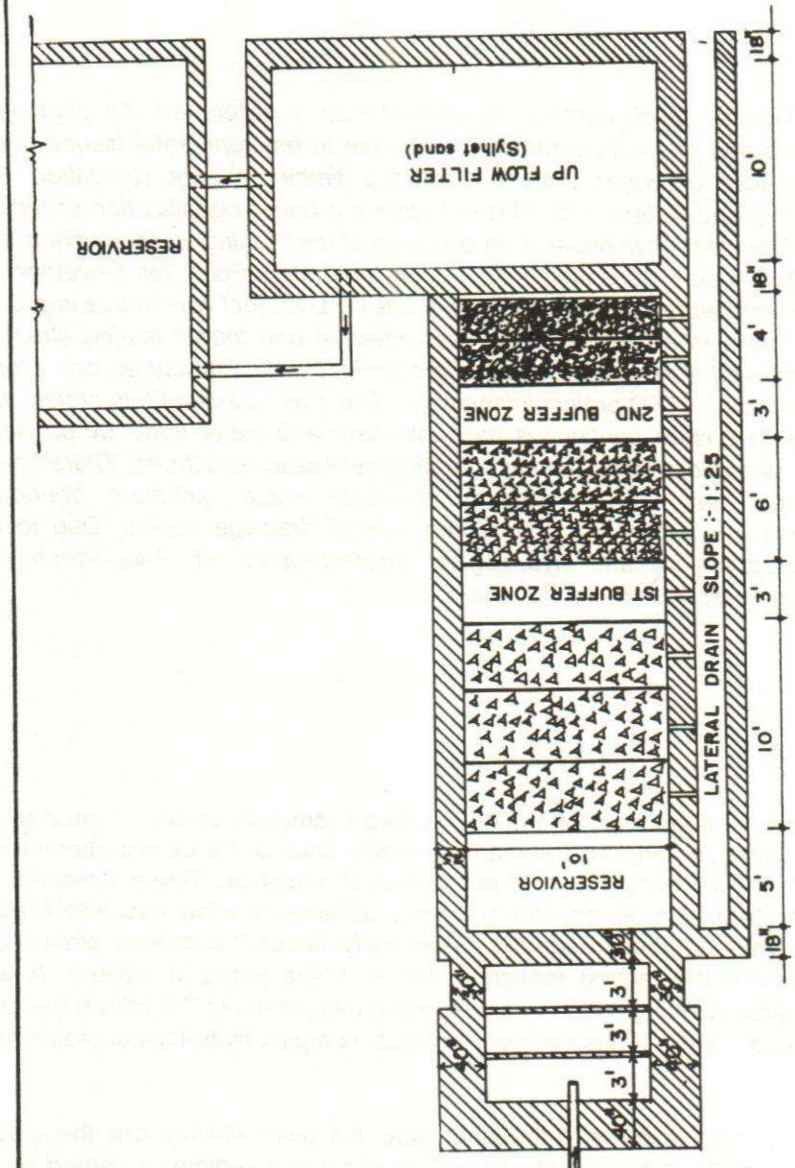

FIG. 4 : LAYOUT PLAN OF IRON & ARSENIC REMOVAL PLANT

AN APPROACH TO DETERMINE SIZE OF DRAINAGE STRUCTURE FOR MICRO-CATCHMENT

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Abstract

There are many factors, which combine to characterize a watershed. As population continues to expand and interests continue to increase in environmental issues, more emphasis will be placed on water, mineral resources, timber, energy, recreation, and residential uses on upland watersheds. This will require a better classification schemes. The background of this technical paper is an outcome of the training to engineering field staff of CARE International in Bangladesh under Integrated Food for Development (IFFD) Project. The concepts in implementation of the IFFD project was to use improved design to provide more free flow of water, cost effective and longer lasting structure which could be achieved through the policy of proper location of structures with proper design and construction in IFFD project intervention. The major thrust of this project was to use more pipes to ensure continuous passable road and water flows by providing necessary earthwork and drainage and incorporating necessary structures. Therefore, in this paper attempts have been made to delineate micro-catchment boundary, determining its area, drainage volume including size of drainage outlets. Due to the unique deltaic geographical and hydrological characteristics of Bangladesh, the problems of drainage vary from region to region.

Introduction

Problem statement

Bangladesh is a deltaic flood plain of 144,000 square kilometers of land located at the confluence of the Ganges and the Brahmaputra rivers. Due to the deltaic character of the country, most part of Bangladesh is comprised of wetlands. These wetlands i.e. lower slopes water areas are represented by rivers, streams, shallow freshwater lakes, ponds, marshes, water storage reservoirs, seasonally flooded cultivated plains, and estuarine areas where the largest mangrove forest in the world is located. A total seasonal wetland area of Bangladesh has been estimated at about 7.5 million hectares according to (Akonda, 1989). It has perhaps the most complex hydrological problems in the world.

Rivers form the dominant features in the landscape; the delta which forms the greater part of the country has been built by the deposition of silt and sediments carried by the three large rivers - the Ganges, the Brahmaputra - Jamuna and the Meghna and their tributaries. These rivers annually carry over two billion tons of sediments, along with their huge discharge, and lay them in the flood plains, rivers, low - lying areas, estuaries wherever the environment of deposition is favorable in the continually shifting channels. All of these rivers ultimately discharge into the Bay of Bengal (MPO 1987).

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The total drainage area of the three major rivers is about 1,554,000 sq. km but less than 10 percent of this is within Bangladesh. Bangladesh receives nearly 1.07 million cubic meters of average annual surface water inflow from India (Rashid 1977). Ninety percent of this originates from outside Bangladesh in India, Nepal, Burma and Bhutan.

The **Table 1** given below illustrates an indication about the origin and drainage area of the major river systems of Bangladesh.

Table 1: Drainage area of three major river systems

River	Total Drainage Area (Km^2)	Area Drained	Originated From
Brahmaputra	573,440	Drains large parts of Himalaya in Tibet and India before entering Bangladesh.	Himalaya Mountains in Tibet, passes through Assam, India and enters Bangladesh.
Ganges	1,116,800	China, Nepal, India and Bangladesh.	Devaprag, the meeting point of Bhagirathi (Uttar Pradesh) and Alakananda (Garthal - Tibet border).
Meghna	80,000	Parts of Assam and Tripura hills and major parts of Sylhet, Mymensingh, Dhaka and Comilla district.	Assam and Tripura hills in India.
Rivers in Southeast Bangladesh	-	The Noakhali, Chittagong and Chittagong Hill Tracts Districts	In the hill ranges of Tripura and Assam in India.

Source: Rashid 1977

Concepts in implementing the IFFD project

The major emphasis of improved design is to provide more free flow of water, cost effective and longer lasting structure, which could be achieved through the following policy of IFFD project intervention:

- "continuous passable road and water flows" by :
  - providing necessary earthwork and drainage;
  - incorporating necessary structurés;
- Improve the utility of the road and sustainability by :
  - reducing erosion by water;
  - blocking drains as silted up;
  - reducing hydraulic pressure which washes out roads;
- Remove negative effects by :
  - removing the blockage of natural water flow which created most previous problems;
- Improve free flow of water by :
  - structure best location;
  - ensuring natural balance for the ecosystem/food chain flows;
  - improving fish migration routes from one breeding area to another;
  - reducing residual effect of agro-chemicals on a localized area;

Objectives

The objectives are narrated on the basis of the IFFD Project requirements which are as follow:

- To assess the need of training or upgrading for the participants' in the field of hydrology;
- To improve the design efficiencies of structures and understanding of overland flows;
- To provide awareness about affects of impeded drainage;
- To apply the concept of "continuous waterways" in improving the road network with better understanding of the environmental and hydrological parameters and calculations;
- Improve the utility of the road and sustainability by :
  - reducing erosion by water;
  - reducing blocking drains as silted up;
  - reducing hydraulic pressure which washes out roads;
- Remove negative effects by :
  - removing the blockage of natural water flow which created most previous problems;
- Improved free flow of water by :
  - structure best location;
  - ensuring natural balance for the ecosystem/food chain flows;
  - improving fish migration routes from one breeding area to another;

Literature review

In ODA (1992), it is mentioned that 'Watershed management' can mean different things to different people. Watershed in American sense is a synonym for what has been more commonly known in British as a 'catchment' – a usage that requires the traditional British watershed to be referred to as a 'watershed boundary'. Watershed management is a product of different permutations of several factors, of which the most important are:

- The physical unit to be managed, which can vary greatly in terms of:
  - scale, from very large (whole drainage or river basin) to very small (micro-watersheds of as little as 500 ha)
  - location (the whole basin, or only its upper or lower sections)
- The nature of the management task, which can range from water resources planning (usually at the level of the whole river basin or its lower, downstream section) to the implementation of development activities involving the combined management of water, land and other resources (often within upper watershed area only).

Methodology

The methodology of the study is detailed through the following sub sections:

Importance of catchment

- Catchments are important for calculation of water volumes and maximum flow rates;
- The delineation of the drainage basin can be done from the contour map of the area to be planned for development using the knowledge of watershed management.

In the present case, the drainage map of LGED has been used where the flow direction of water is mentioned for the defined channel. But for the undefined channel, alignment was visited and had a look around the topography further upstream of the road, talked with the local people and collected information about flow direction and AHFL.

- From the field observation and map information, the general flow direction of water could be determined on the basis of the topographical feature.

These compartments, or " micro catchments" discharge a certain amount of drainage water which is proportional to rainfall necessitating a bigger sized drainage structure at the final outlet.

There is a linear relationship between the size of catchment and runoff for a rainfall of certain duration and intensity when the other parameters remain unchanged.

The calculations and relevant concepts are mentioned hereafter. The useable maps for computation of watershed catchment and runoff are:

- Drainage and embankment map of LGED in 1:50,000 scale;
- Irrigation map of LGED in 1:50,000 scale;
- Land use map of LGED in 1:50,000 scale;

Watershed catchment area determination

The area of watershed catchment has been calculated using an **Acreage Computer**.

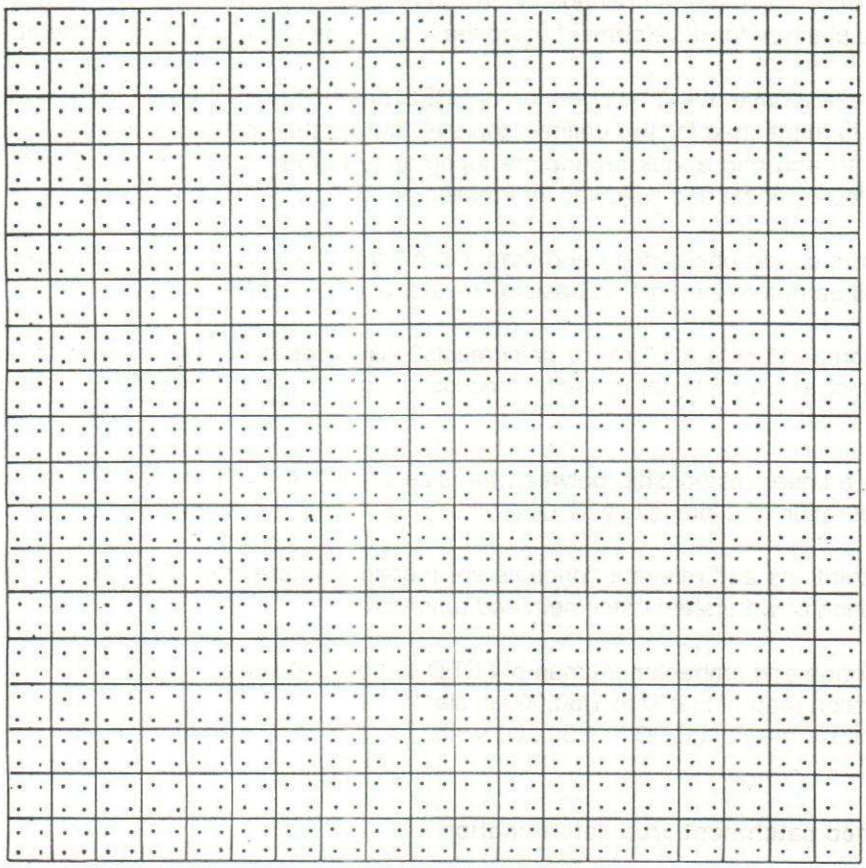
Acreage Computer (Plate-1) is a transparent sheet divided into a number of small grids of square size. Each square is marked with four dots. The method of calculating area of a nonuniform sized land surface is to count the number of dots by superimposing the transparent A.C. over the map. From the counted number of dots, area in acre will be calculated by a simple equation.

Measuring area

The computation exercise is very simple one. In **Figure 1**, four watershed catchments are identified. The sample calculation is done for **compartment 2**. The following steps should be followed in computation:

US DEPARTMENT OF AGRICULTURE, SOIL CONSEVATION SERVICE **Plate - 1**

ACREAGE COMPUTER



FRUCTIONAL SCALE	INCHES PER MILE	ACRES PER SQUARE	ACRES PER DOT
1 : 3,960	16	1/4	1/16
1 : 5,280	12	0.444	0.111
1 : 7,920	8	1	1/4
1 : 15,840	4	4	1
1 : 20,000	3.168	6.379	1.594
1 : 31,680	2	16	4
1 : 63,360	1	64	16

Formula to determine area at scales other than those given above:

$$\text{Area} = \frac{16 \times \text{No. of Dots}}{(\text{Scale in inches per mile})^2}$$

Example: Scale 7" = 1 mile, 400 dots: $A = \frac{16 \times 400}{7^2} = 130.6$ Acres

To convert fractional scale to inches per mile -- divide 63,360 by denominator of representative fraction

Example: Convert scale of 1 : 10,000 to inches per mile: $\frac{63{,}360}{10{,}000} = 6.34$" per mile

7 - L - 5569

- lay the map from where area will be measured;
- delineate the estimated boundary of the catchment;
- check the arrow of drainage flow on the map and also compare with the field condition;
- locate the structure site;
- overlay the A.C. on the map;
- count the number of dots in the upstream of the structure falling within the desired area which is 140 ;
- convert the fractional scale into inches per mile (1:50,000 scale is equal 1.27" = 1 mile);

Therefore area of the compartment is

$$A = \frac{16 \times \text{No. of dots}}{(\text{Scale in inches per mile})^2} = \frac{16 \times 140}{(1.27)^2} = 1388 \text{ Acres}$$

Table 2: Area of four micro-catchment

Micro-Catch.No	No of Dots	Area in Acre	Area in Ha	Area in Km^2
1	2	3	$4 = \frac{3}{2.47}$	$5 = \frac{4}{10^2}$
1	66	654.72	265.06	2.65
2	140	1388	561.9	5.62
3	57	565.44	228.92	2.29
4	29	287.69	116.47	1.17

Catchment design discharge measurement

Design Storm Consideration:

Catchment design discharge or runoff volume generated from a design storm can be computed by the steps mentioned below:

- The computation is based on a micro-catchment (compartment 2 in **Figure 1**) which falls in the Rangpur district;
- Locate the micro-catchment on the 4 month rainfall index map of **Figure 2**. (an isohyetal map of 5 day duration rainfall with 1 in 10 year return period developed by IECO Master Plan for Bangladesh). It is evident that the 4 month rainfall index value is 1650 mm;
- Rainfall stations nearest to the catchment area will be the most representative data. If two or more stations are equidistant, the station with largest amount of value should be selected;
- Multiply the 4 month rainfall index by IECO values to arrive at accumulative point rainfall. The daily increment of point rainfall will be calculated by subtracting the previous days accumulative value;

Table 3: Calculation of point rainfall

Day	Fraction of 5 day accumulative rainfall (From IECO Master Plan	Accumulative point rainfall (mm) (Fraction of 4 month rainfall index)	Point rainfall daily increment (mm)
Col. 1	Col. 2	Col. 3 = 1650 x Col. 2	Col. 4
1	0.128	211.2	211.2 (day 1)
2	0.192	316.8	105.6 (day 2 – day 1)
3	0.230	379.5	62.7 (day 3 – day 2)
4	0.257	424.05	44.55 (day 4 – day 3)
5	0.276	455.40	31.35 (day 5 – day 4)

Source: Hydrologic and Hydraulic Design Procedures, BWDB

- Determine areal reduction factor for the point rainfall, which decreases as the size of the catchment increases. The storm is then modeled by drawing concentric circles representing isohyets with a radius of 800 m (0.50 mile), 2400 m (1.5 mile) and then each successive 1600 m until the whole catchment is covered.

Area between the isohyetals/circles are measured. Table 4 shows the percentage of point rainfall to be used for areas between the circle limits. For the compartment 2, the entire catchment falls within 2400 m radius circle.

Table 4: Areal reduction factor computation

Distance from Center of Catchment	Percentage of Point Rainfall	Area Between Circle (Km^2)	Point Rainfall (%) x Area
Less than 800 m	100	2.17	2.17
Between 800 and 2400 m	88	3.17	2.79
2400 and 4000m	81.7		
4000 and 5600 m	77.5		
5600 and 7200 m	74.2		
7200 and 8800	72.0		
8800 and 10400 m	69.8		
10400 and 12000 m	67.8		
12000 and 13600 m	66.0		
13600 and 15200 m	64.5		
15200 and 16800 m	63.1		
Total		5.34	4.96

$$\text{Thus the areal reduction factor} = \frac{4.96}{5.34} = 0.93$$

The point rainfall shown in **Table 3** should be multiplied by the areal reduction factor in order to convert them to daily uniform depth equivalents which is carried out in **Table 5.**

FIGURE : 1 : TYPICAL DIAGRAM OF MICRO-CATCHMENT

FROM A MAP OF 1 : 50,000 SCALE (DRAINAGE AND EMBANKMENT MAP)

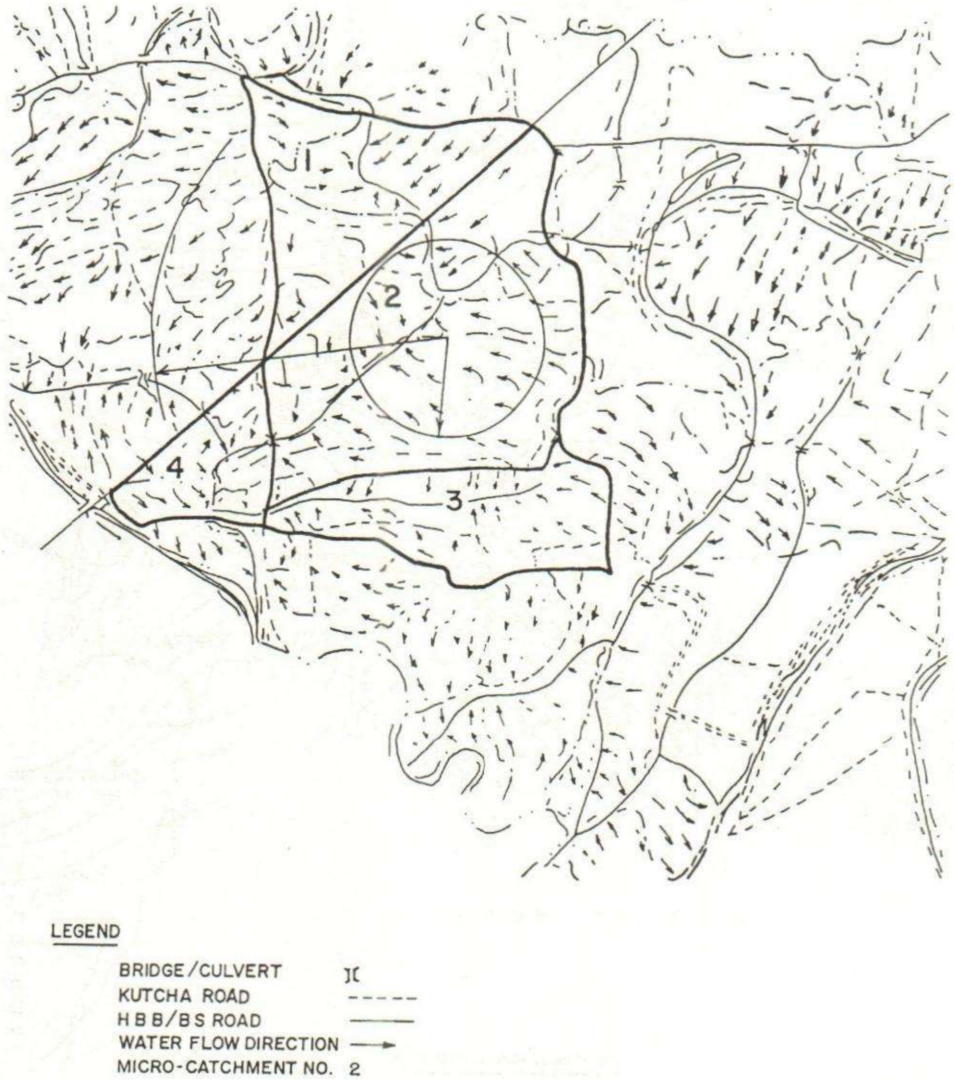
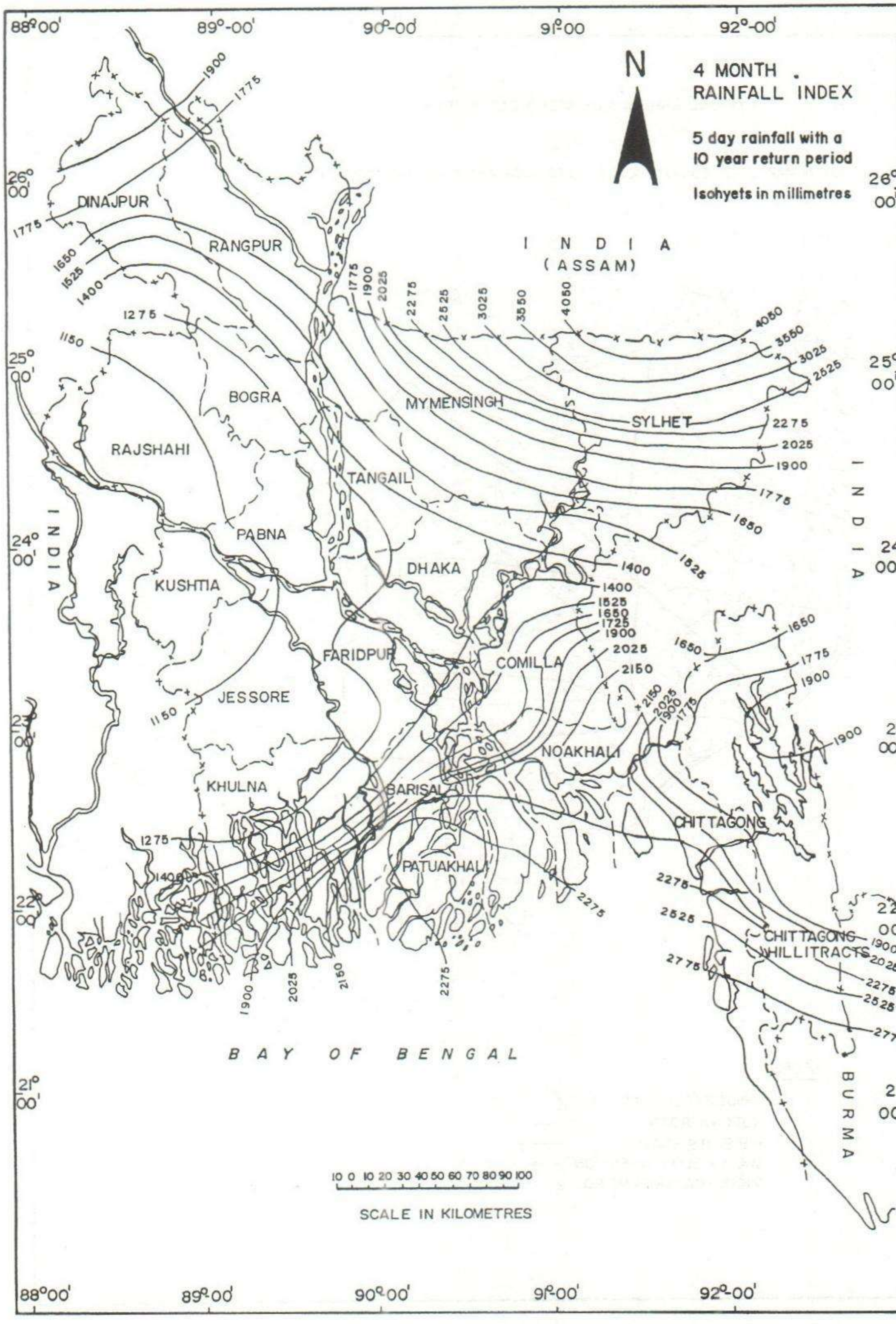


FIGURE- 2



4 MONTH RAINFALL INDEX
5 day rainfall with a 10 year return period
Isohyets in millimetres
N
INDIA (ASSAM)
INDIA
BURMA
BAY OF BENGAL
DINAJPUR
RANGPUR
BOGRA
RAJSHAHI
PABNA
KUSHTIA
JESSORE
KHULNA
MYMENSINGH
SYLHET
TANGAIL
DHAKA
FARIDPUR
COMILLA
NOAKHALI
BARISAL
PATUAKHALI
CHITTAGONG
CHITTAGONG HILLITRACTS
88°00'
89°00'
90°00'
91°00'
92°00'
26° 00'
25° 00'
24° 00'
23° 00'
22° 00'
21° 00'
10 0 10 20 30 40 50 60 70 80 90 100
SCALE IN KILOMETRES

Table– 5: Conversion from point rainfall to uniform depth equivalent

Duration	Point Rainfall (mm)	Uniform Depth Equivalent (mm)
Day 1	211.2	197
Day 2	105.6	99
Day 3	62.7	59
Day 4	44.55	42
Day 5	31.35	30

- The distribution of theoretical 5 day storm would be based on the daily increments, which can occur in order. In order to satisfy losses, which will occur early in the storm, the sequence of the days can be arranged into 3, 2, 1, 4 and 5.

The design storm for the micro-catchment – 2 will be

59 > 99 > 197 > 42 > 30

day 1 > day 2 > day 3 > day 4 > day 5

- Volume of water caused by design storm will be calculated through a balancing process in order to arrive at the volume of water stored in the catchment as a result of storm. This is possible only when elevation--storage curve is available prepared from estimated for filling beels, paddy land etc as depression storage.

- The catchment is assumed to be relatively dry prior to the 5 day storm. As such, 100 mm is estimated for filling beels, paddy land etc. as depression storage.

- Infiltration of water into the soil occurs which depends on the soil type. A value of 25 mm per day has been assumed for the computation.

Table 6: Available water balance from the design 5 day storm

Day	Rainfall (mm)	Depression Storage (mm)	Infiltration Loss (mm)	Available Storage (mm)	Net Storage Depth (mm)
1	59	100	25	- 66	0
2	99	66	25	8	8
3	197		25	172	172
4	42		25	17	17
5	30		25	5	5
				Total	202

Volume of water stored in the micro-catchment (compartment 2)

= Total Area in m^2 x Net Storage Depth in m

= $(5.62 \times 10^6) \times (0.202)\ m^3$

= $1.1 \times 10^6\ m^3$

- Determination of runoff is based on the critical level to which water should be evacuated to minimize major crop damage. But due to lack of elevation – storage curve prepared from contour map, it is not possible to determine the critical level.

Again, if we need to keep water inside the catchment for a certain depth, the volume of water to be evacuated in excess of the volume above that depth i. e.

$V_e = V - V_a$ Where,

V_e = Volume of water to be evacuated
V = Total volume of water
V_a = Allowable volume of water inside the catchment

The maximum amount of time that rice may remain under water before being damaged is approximated 3 days.

$$\text{Therefore discharge } Q = \frac{1135240}{3 \times 24 \times 60 \times 60} = 4.4\ m^3/s$$

From Chezy's equation, we get

$$D = C \times (Q^2/S)^{\frac{1}{5}} \quad \text{(Ray 1974)}$$

Where D = Pipe diameter in feet
Q = Discharge in ft^3/sec
C = Constant for pipe material (0.222)
S = Slope of pipe (1 in 30)

So, the total diameter required (if we provide pipe culvert)

$$D = 0.222 \times (4.4 \times 35.28)^{2/5}/(1/30)^{1/5} = 3.30 \text{ feet} = 1.00 \text{ m}$$

Area of pipe having diameter 1.00 m = 0.8 m^2

Area of one pipe of diameter 0.300 m = 0.10 m^2

$$\text{Therefore, total requirement of pipe} = \frac{0.80}{0.10} = 8 \text{ nos.}$$

Discharge is calculated as an ideal case with no inflow into the compartment or outflow from other outlets. Only the localised rainfall within the compartment is considered. The influence of discharge from other compartments will be determined in the same way for compartment 2 and a balancing equation will be set to determine the net discharge for a structure.

Results and discussion

In the wet land areas as well as other parts of Bangladesh, CARE funded a good number of kutcha roads projects where no drainage structures were provided which caused to cut the roads at the natural water flow path. Due to such cut the ultimate goal of improving rural transport system was not achieved. Therefore, CARE decided to train their engineering field staff in hydrology to upgrade knowledge for determining the size of such structures from the hydrological viewpoint. The methodology discussed herein was used by the end users of that project.

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EFFECTS OF CONSTRUCTION OF BANGABANDHU BRIDGE ON FLOW CHARACTERISTICS OF THE JAMUNA RIVER

Abu Musa Md. Motaher Ahmed[1] and Md. Abul Ala Moududi[2]

Abstract

This paper presents the effects of river training structures of Bangabandhu Bridge on flow pattern of the river Jamuna at the project area. An overall fixed bed physical model was constructed to investigate the changes in flow pattern and flow field and also to forecast the possible morphological changes around the bridge. In the modelling, a distance of 14 km at the upstream and 10 km at the downstream from the centerline of the bridge were considered. Requisite topographic, hydrographic, bed material data and design of training structures were collected and used for design of distorted model as per Froude's model law with horizontal scale ratio of 1:280 and vertical scale ratio of 1:80. The model was constructed over 450 mm thick sand bed so that bed geometry of the river could easily be reproduced in the model. A layer of brick was placed over the sand bed and then 15 mm thick sand-cement plastering was done. As per design requirements, brick chips were placed on the plastered surface to reproduce prototype roughness in the model. The upstream and downstream boundary conditions were taken from 2-D mathematical modelling results. After necessary calibration, test runs were conducted.

Due to the bridge construction, overall changes in flow pattern and their effects were investigated in this study. Also the impact of extreme flow on river training structures were examined. Effects of three dimensional aspects such as vortices, helical flow, scour etc. can not be observed in the model, but forecasts based on these effects can be encapsulated indicatively by the analysis of flow velocity and flow fields. Also morphological changes in terms of erosion, deposition, channel migration, channel formation, char formation etc. could be assessed based on flow concentration, which gives preliminary idea to the engineers. For quantitative results, movable bed sectional models are recommended.

Introduction

The river Jamuna is a barrier between the north-western and eastern parts of Bangladesh. To link these two parts of the country, Government of Bangladesh with the assistance of international donors is already implemented the construction of bridge The bridge having a curved length of 4.8 km and several river training works for guiding the flow to pass under the bridge corridor safely. The river training works comprised of two major guide bunds with length of more than 2 km each, and two hard points at the upstream of the bridge. At the bridge site the average width of the river is 9.3 km. The bridge length was reduced through constriction of the river by construction of a closure dam at the western side. Four minor channels of the river were closed due to closure. The river Jamuna is a multi-channeled river with typically 2 to 4 channels. The maximum and minimum discharge is about 90,000

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m^3/s and 2,900 m^3/s respectively. The river originates on the northern slope of the Himalayas and traverse through China, Bhutan, India and Bangladesh. It enters Bangladesh through northern border and joins the Ganges in the central part. The combined rivers take the name, the Padma, and flows further south where it meets with the Meghna and the combined flow in the name of lower Meghna is discharged into the Bay of Bengal. The river flows through fluvial deposits of fine sand and traces of silt. The river is braiding in nature and carries a huge quantity of sediment load, of the order of 750 million tons a year (Hossain, 1992a). The dominant and bankfull discharge for this river has been computed and found in the order of 39,000 m^3/s and 44,000 m^3/s respectively (Hossain, 1992b). Geomorphologically, the river is in a state of dynamic instability (Coleman,1969, Thorne et.al. 1995). This dynamic river changes its planform from year to year resulting in major shifts of main channel in braiding belt. The continuous morphological changes may affect the construction works and disrupting the maintenance of the bridge. The physical model study was undertaken to forecast the changes in flow pattern and simultaneous possible morphological changes due to construction of different river training structures and also to over see the impact of extreme flow on them.

In this study, the model was operated at extreme flow conditions (i.e. with design discharge of bridge, 91,000 m^3/s) to ascertain the needs/options for temporary protective measures during and after construction.

The study area

The study area is about 135 km downstream of Bangladesh border of the Jamuna river. The latitude and longitude of the study area lies between $24^0 21'$N to $24^0 31'$N and $89^0 41'$E to $89^0 51'$E. The main bridge along with the components are shown in Figure 1. The eastern part of the project area is located under Tangail district and western part under Sirajgonj district. The area comprised of flood plains, mega-chars (stable chars), unstable chars where the people lives. The study area is extended towards north for 14 km at the upstream and 10 km towards south at the downstream from the bridge centre line. The maximum and minimum width of the river within the study area are about 15.0 and 8.0 km. The width of the river at bridge crossing is 9.3 km.

Data acquisition

Requisite data were collected from different sources, which are described below:

Topographic Data :

The bathymetric survey was carried out by Surface Water Modelling Centre (SWMC) during August-September 1996. In this survey a total 115 numbers of cross-section were surveyed at an interval of 100 m in the vicinity of the bridge covering 3.0 km upstream and 3.0 km downstream. Then cross-sections were taken at an interval of 200 m for 3.0 km upstream and 3.0 km downstream. For remaining 8.0 km at the upstream and 4.0 km at the downstream, cross-sections were taken at an interval of 500 m. Depth were measured at an interval of 50 m along each cross section with the help of echo-sounder. At the same an interval of 50 m along each cross section with the help of echo-sounder. At the same period, elevations of islands and sand bars were taken.

Hydrometric Data:

Hydrometric data such as water level, discharge, average velocity were collected from Surface Water Modelling Centre (SWMC), Bangladesh Water Development Board (BWDB) and Construction Supervision Consultants (CSC i.e. RPT-NEDECO- BCL) of Jamuna Multipurpose Bridge Authority.

Bed material data:

Several numbers of bed material samples were collected from different locations of the surveyed area and analyzed. The grain size varies from 0.062 mm to about 4.15 mm with a median size of about 0.19 mm.

Design data :

Design of west channel closure, East and West guide bunds, bridge and hard points etc. were collected from Jamuna Multipurpose Bridge Authority (JMBA) and CSC. These data were used for model design.

Theoretical considerations

The velocity and discharge scale ratios can be obtained as follows:

$$V_r=h_r^{1/2} \tag{1}$$

$$Q_r=L_r h_r^{1/2} \tag{2}$$

Where, V_r, Q_r, L_r and h_r are the velocity scale ratio, discharge scale ratio, horizontal scale ratio and vertical scale ratio respectively.

The Chezy's roughness co-efficient (C) for the Jamuna river in the vicinity of the bridge site can be obtained from the following equation (RPT-NEDECO-BCL, August, 1989):

$$C= (0.126Q^{0.126})/S^{0.5} \tag{3}$$

Manning's roughness co-efficient (n) can be obtained by,

$$n= R^{1/6}/C \tag{4}$$

Where, R and S are hydraulic radius and energy gradient respectively.

For long stretches of either a river or a canal the scaling requires a distortion in slope. This is done to offset the disproportionately high resistance and to obtain a sufficiently high value of Reynold's number, which ensures turbulent flow. The required distortion of slope can be computed by Manning's formula as follows:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{5}$$

In order that the velocity ratio will vary with the square root of the depth scale as required by the Froude's law

$$V_r = \sqrt{Y_r} = \frac{R_r^{\ 2/3} S_r^{\ 1/2}}{n_r} \tag{6}$$

Substituting y/L for S results in the following expression

$$\sqrt{Y_r} = \frac{R_r^{\ 2/3} Y_r^{\ 1/2}}{n_r L_r^{\ 1/2}} \tag{7}$$

By rearranging the above equation, we obtain

$$\frac{Y_r}{L_r} = \frac{n_r^{\ 2} Y_r}{R_r^{\ 4/3}} \tag{8}$$

If n is known for model and prototype, n_r is also known and the exaggeration y_r/L_r can be computed for a given depth y and hydraulic radius R. In models for which the slope distortion is dictated by other considerations, an adjustment of model roughness is required to simulate prototype conditions. If the distortion and value of n for the prototype are known, the required value of n for model can be computed from the above equation.

The model with roughness adjusted for a particular depth will yield dependable results for flow at or near that depth. If a problem involves several depths, the model roughness should be adjusted to give an average friction that is approximately valid for each depth and the roughness may be varied with depth for a closer approximation at all depths (Warnock, 1949). The fact is that the Manning's n is strictly a roughness characteristic, and that the Manning's formula applies only at sufficiently high values of R for viscous effects to be negligible.

The relative roughness of model and prototype are kept same to get geometric similitude. For dynamic similitude, the Froude's number and Reynold's number should be the same in model and prototype. But it is usually impossible and unnecessary to satisfy both criteria in model. However, the viscous effect (expressed by Reynold's number) is minimized by ensuring rough turbulent flow in the model.

In many cases, a commonly used alternative is to exaggerate the model depths and to use a distorted model. This has been found to be practically effective but there are fundamental reasons, which suggest the possibility of distortion. Distortion increases the model roughness and the model must be rougher than the prototype. To minimise the scale effect, the distortion was kept minimum with the availability of space and pumping capacity for the present study.

Model set up and scale ratios

In the frame work of the model study, an overall fixed bed model covering 14 km at upstream and 10 km at downstream of the bridge axis was reproduced with a view to forecast the changes in flow pattern and simultaneous possible morphological changes due to construction of different river training structures and also to see the impact of extreme flow on them. The extent of the model area including the water supply system is shown in Figure 2 and the geometric scale factors are given in Table 1

Table 1 : Geometric scales

Parameters	Unit	Scale factor
Length	m	280
Width	m	280
Depth	m	80

The other scale ratios were determined based on Froude's model law to achieve geometric and dynamic similarity in the model and are presented in Table 2.

Table 2: Model scale ratios

Parameters	Unit	Scale factor
Discharge	m^3/s	200350
Flow velocity	m/s	8.94
Water surface slope	-	1/3.5
Relative roughness(k/h)	-	0.0233
Chezy's co-efficient (C)	$m^{0.5}/s$	1.87
Mannings co-efficient (n)	$s/m^{0.33}$	1.11

Model construction

The model bed was constructed as per bathymetric data collected by SWMC during August-

September 1996. A total of 115 numbers of cross-sectional elevation were given by leveling instruments to reproduce prototype river bed geometry. A layer of brick was placed over the sand bed and then plastered by 15 mm thick sand-cement mortar. Finally, brick chips were placed to reproduce prototype roughness in the model as per model design. A stilling basin was constructed at the upstream boundary of the model to dissipate excessive energy of falling water from the weir so that steady uniform flow was ensured at the upstream of the inflow section. Eight numbers of tail gates were constructed at the immediate downstream of the last section of the model to maintain a constant water level.

Boundary conditions

For a fixed bed overall model, two types of boundary conditions were selected, such as, an upstream boundary condition at the inflow of the model and the downstream boundary condition at the outflow of the model. The upstream boundary condition is the discharge distribution along the upstream section of the model. The downstream boundary condition is the water level along the downstream of the model boundary. The upstream discharge distribution and downstream water level were collected from 2-D mathematical model which was carried out by Danish Hydraulic Institute (DHI) in association with Surface Water Modelling Centre (SWMC).This discharge distribution was ensured at the upstream of the model boundary by several trails and the downstream water level was controlled by operating tilting gates.

Model calibration and verification

Discharge data was not available for monsoon 1996 within the modelled area. However, peak water level was known at the bridge corridor, which was measured at 13.06 m PWD. For estimation of peak discharge at the bridge corridor, available stage-discharge relationship at Bahadurabad for 1995-96 and stage-discharge relationships at Bahadurabad and Sirajgonj for 1994-95 were analyzed. After analysis, the discharge and the corresponding water level were found to be 68,457 m^3/s and 13.26 m PWD respectively. This estimated water level was found to be close to the measured data. From the bathymetric survey, the cross-sectional area of the calibration section was known. The average velocity at that section was computed by dividing this discharge by cross-sectional area which accounts for 1.63 m/s. This discharge was used for model calibration. The model calibration was done at a section immediate downstream of the bridge line until achieving optimum prototype similarity.

The Chezy's roughness value of the Jamuna river at Sirajgonj varies between 40 $m^{0.5}/s$ for low flow and 100 $m^{0.5}/s$ for flood condition (Klaassen et.al., 1988). In the Jamuna bridge project study, it was found that the Chezy number varies between 40 $m^{0.5}/s$ for low flow and 100 $m^{0.5}/s$ for flood based on Bangladesh Water Development Board (BWDB) discharge measurement (RPT-NEDECO-BCL, August 1989). In that study, it was mentioned that the Chezy's roughness value varies with water depth. On the basis of these studies, the Chezy's roughness value of 70 $m^{0.5}/s$ for 68,457 m^3/s was considered. For fulfilment of geometric and dynamic similitude in the model, the model bed was made with unfinished concrete to reproduce prototype behaviour. The bed roughness in the mcdel was adjusted by several trail runs. After calibration, the results found in the model are presented in Table 3. The model was then verified based on average water depth, flow velocity and relative roughness.

Table 3: 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	9.17	0.115	80	9.20
Design discharge	m^3/s	68,457	0.343	200350	68,720
Flow velocity	m/s	1.63	0.182	8.94	1.62
Relative roughness height (k/h)	-	0.0026	0.123	0.023	0.0028
Chezy's Co-efficient(C)	$m^{0.5}/s$	70	36.83	1.87	69
Manning's co-efficient(n)	$s/m^{0.33}$	0.021	0.0189	1.11	0.021
Water surface slope	-	0.0000744	0.000211	1/3.5	0.00006

It may be seen from table 3 that the estimated values in prototype were found to comply closely to the measured values in the model.

Model testing

Several tests were carried out for different morphological conditions within the modelled area for design discharge of 91,000 m^3/s to investigate the overall effect of construction of various river training works on flow pattern. The impact of extreme flow on river training structures were investigated. Also possible morphological changes were investigated and necessary measurements were taken. The predictions were encapsulated based on analysis of flow velocity and flow field.

Analysis and interpretation

Measured flow velocity and flow field data were anlysed from which the effects of construction of bridge on flow characteristics at upstream and downstream of the bridge. The possible locations of sedimentation and erosion were identified based on critical velocity for initiation of sediment movement (van Rijn, 1984). Critical velocity for bed material movement lies between 0.2-0.4 m/s and the value is 0.36 m/s for bank erosion of Jamuna river (FAP-21/22, 1993, Klaassen et.al. 1988). Float tracking were performed from which flow concentration as well as trend of flow towards the banks were envisaged. A typical float tracking results is reproduced in Figure 3. Further details on the results are available in various reports (RRI 1996 a, 1996 b, 1997 a, 1997 b).

Findings and discussions

Due to the construction of all components of the bridge, the following results were found during different test runs:

From the velocity measurements, it was found that the average velocity in the main channel (between two guide bunds) increased by 10-19% with respect to no training structures in place and the maximum flow concentrated at the midstream of the main channel. On the other hand, velocity increased by about 25% in the vicinity of west guide bund and 5% in the vicinity of east guide bund. The rise in water level at the bridge corridor was found in the order of 12-20 cm.

The flow lines revealed that attacking tendency of flow along left bank started at 5.0 km downstream of east end pier and extended towards downstream direction. On the other hand, attacking trend of flow along right bank was found moderate at three different locations viz. 1.6 km to 3.1 km, 4.9 to 5.3 km and 7.3 to 8.3 km. These attacking nature of flow will lead to bank erosion. This erosion was mainly caused by the dredging of a short cut channel (as shown in Figure 4) at immediate downstream of the west channel closure. But due to erosion at the upstream char and reduction in velocity during the monsoon, the short cut channel might be gradually silted up. As a result the channel will lose its conveyance capacity which ultimately will not be the reason for bank erosion.
The tests revealed that backwater flow persisted in the vicinity of Dhaleswary new spill channel, which may cause to develop a large island. Char just downstream of the bridge along east guide bund will partly be eroded and char in the middle and downstream of the bridge may be washed away due to high velocity. The char just downstream of west guide bund will be washed out because of significant vortices observed there. The char in front of west guide bund will gradually be eroded and the trench around the west guide bund will be silted up in future. Possible erosion, sedimentation and simultaneous scour are shown in Figure 4.

Conclusions

Due to the construction of all components of the bridge, total flow passed through the 4.8 km bridge span and major flow concentrated towards left which exerts a great deal of impact on flow characteristics at both upstream and downstream of the bridge. For constriction of flow at the bridge corridor, the water level was increased which results in reduction of velocity at the upstream. So, the upstream land was found to be submerged during the occurrence of high floods. On the other hand, sudden increase in velocity was observed at the immediate downstream. For this high velocity, chars and banks were threatened. Changes in flow pattern were found to be significant at the downstream rather than upstream. But the overall changes were more pronounced along the left bank which is responsible for bank erosion. For this reason, lot of agricultural land and settlements were found to be engulfed resulting in severe socio-economic problem to the inhabitants of the bank. Also the char lands were found to be eroded which results loss of agricultural lands. Due to decrease in flow velocity along right bank, char may be emerged and can be used for encroachment in due course of time. Although the model bed was fixed, the predictions came through this study were found close to the field observations. Fixed bed model is proved to be very useful to forecast hydrodynamic parameters such as change in water level, velocity, flow pattern. Effects of three dimensional aspects such as vortices, helical flow, scour etc. can not be observed in the model, but forecasts based on these effects can be encapsulated indicatively by the analysis of flow velocity and flow fields. Also morphological changes in terms of erosion, deposition, channel migration, channel formation, char formation etc. could be assessed based on flow concentration, which gives preliminary idea to the engineers. For quantitative results, movable bed sectional models are recommended.

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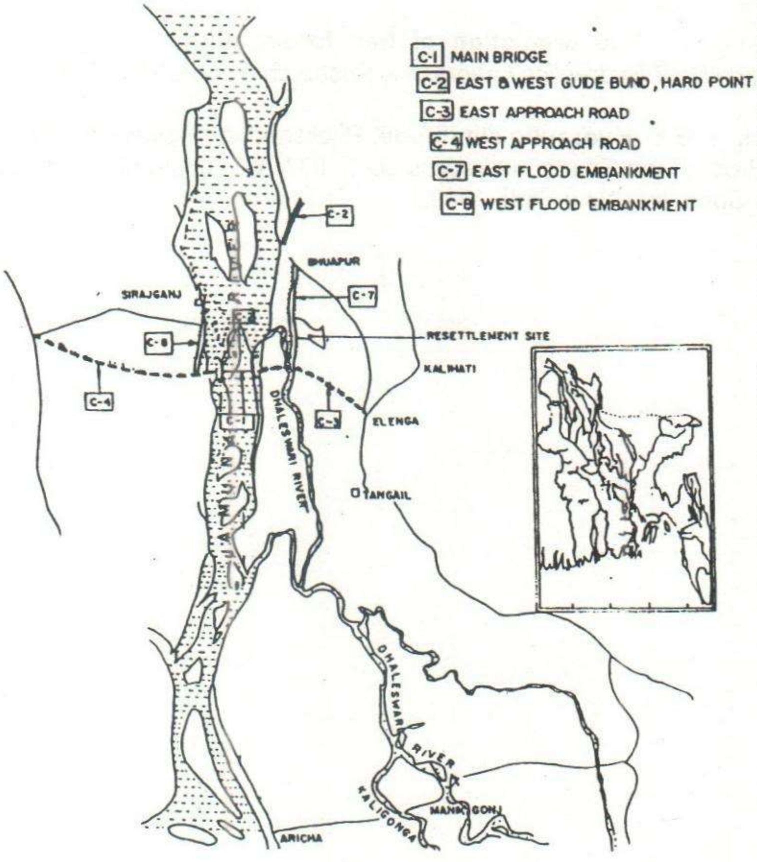


FIGURE 1 : LOCATION OF BANGABANDHU BRIDGE AND ITS COMPONENTS

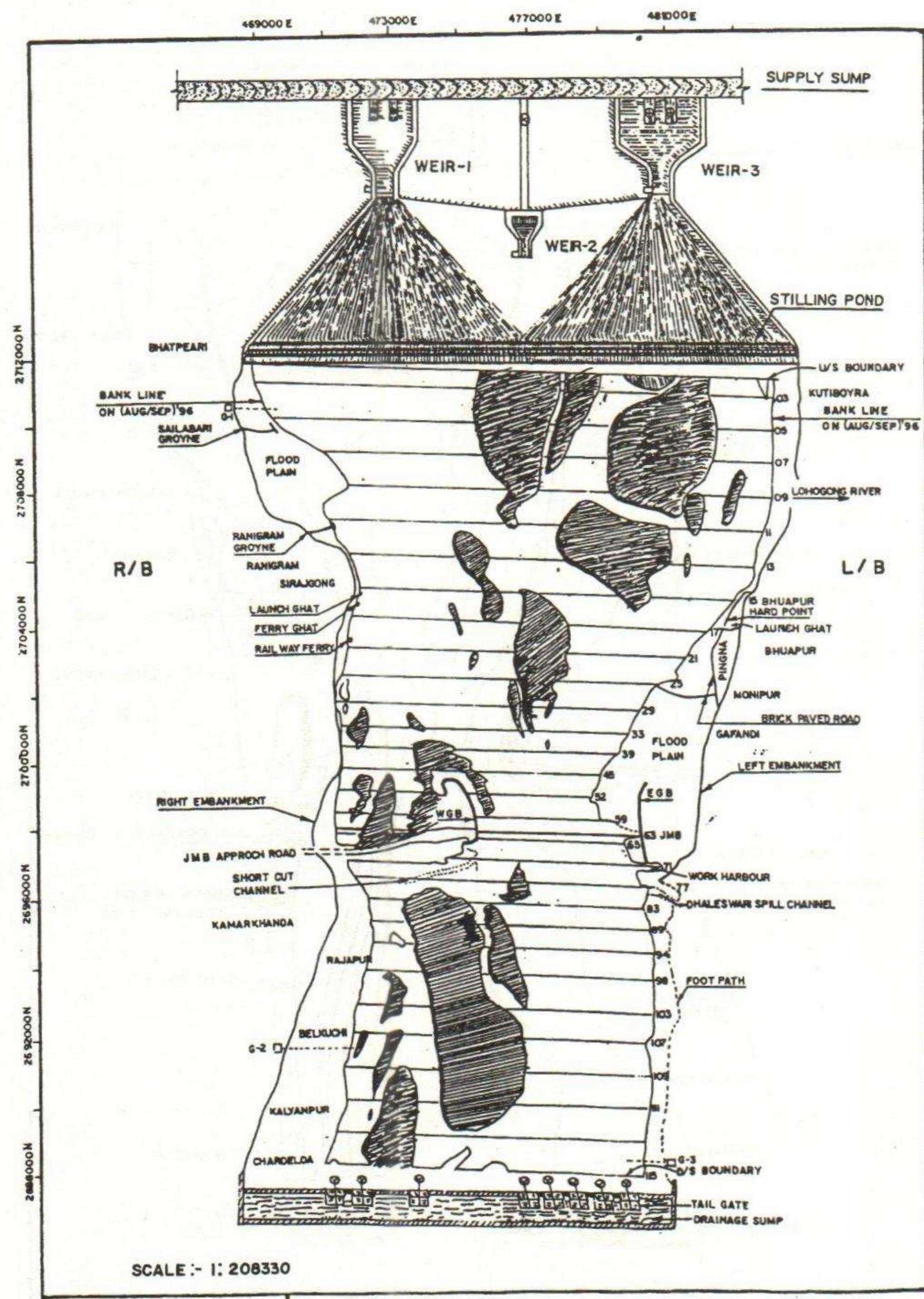
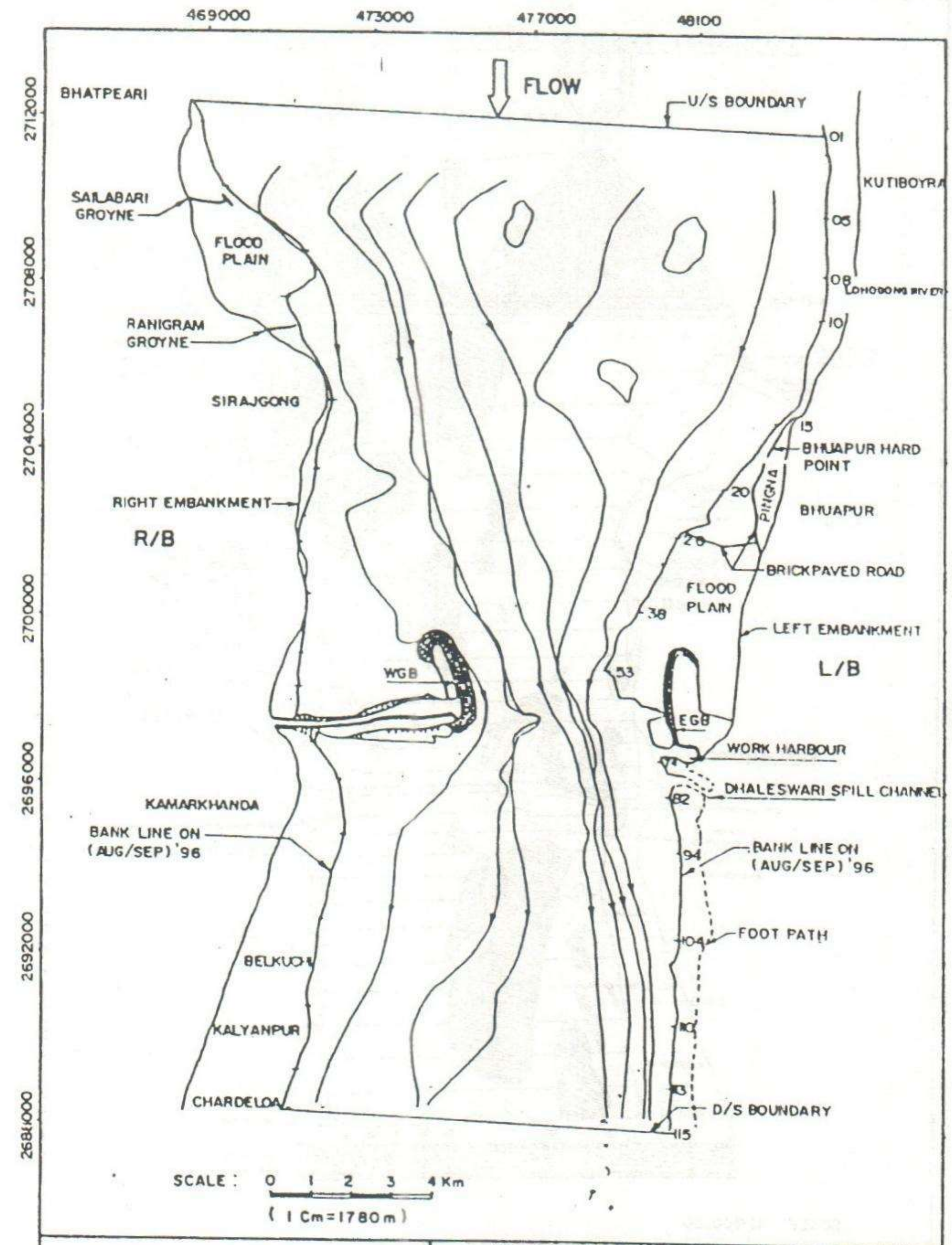


FIGURE NO. 2	EXTENT OF THE MODEL AREA INCLUDING THE WATER SUPPLY



DISCHARGE = 91,000 Cumec	MODEL STUDY OF JAMUNA RIVER AT BANGABANDHU BRIDGE PROJECT AREA

FIG. 3 : FLOW LINES SHOWING ATTAKING TENDENCY IN EXISTING FIELD CONDITION

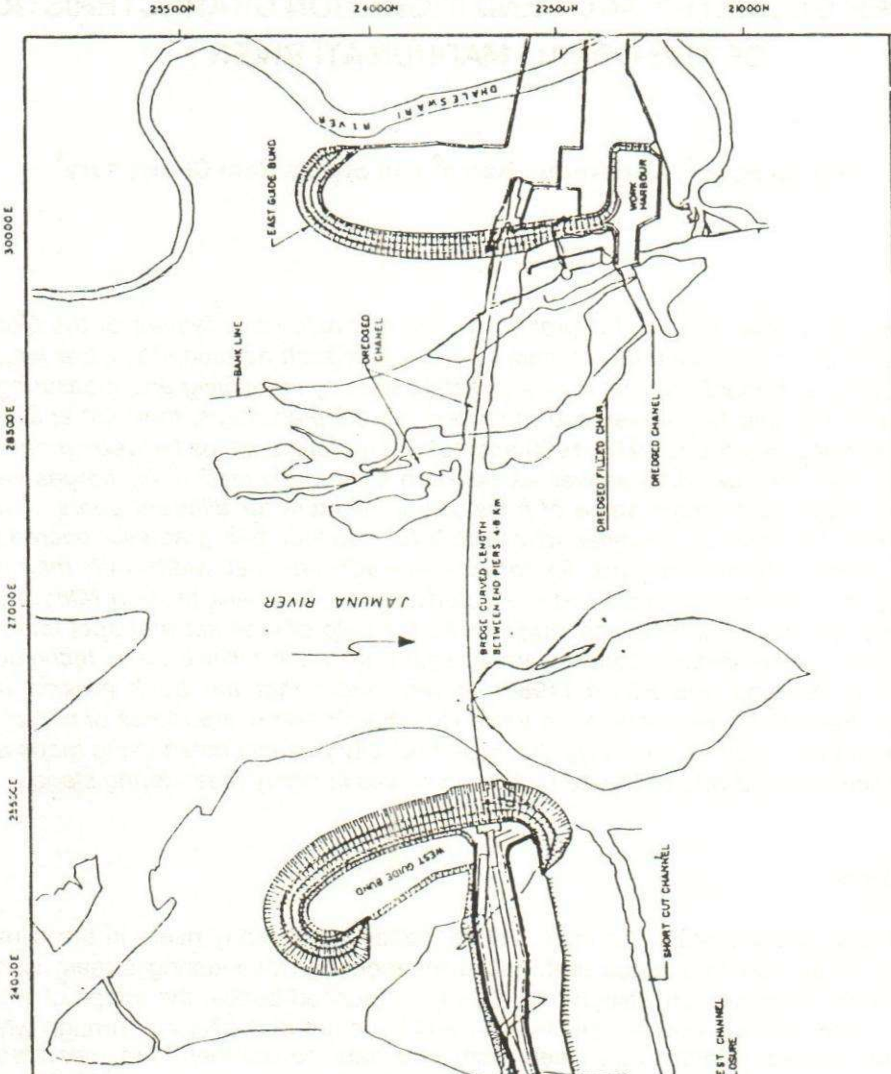


FIGURE NO. 4
(EXISTING FEATURES)

BLP CROSS LINES 1-9, FEBRUARY'96 SURVEY, RPT-NEDCO-BCL
MODEL STUDY OF JAMUNA RIVER AT BANGABANDHU BRIDGE PROJECT AREA

MEANDER GEOMETRY AND BEND MIGRATION CHARACTERISTICS OF THE GORAI - MADHUMATI RIVER

Md. Monirul Islam[1], Kazi Rezaul Karim[2] and Sheela Rani Chowdhury[3]

Abstract

Meander development and bend migration are the characteristics feature of the Gorai-Madhumati river. In this study the meander parameters such as meander wave length, meander belt and sinuosity for the river was established by identifying and measuring in detail the partially and fully developed meanders on the plan maps, land sat and spot imageries of the years 1953, 1973, 1980 and 1990. The relationships between meander parameters and channel width as well as between the representative discharges were developed. From the average value of meander parameters for different years, it was observed that the ratio of meander wavelength to meander belt gradually decreased with time. Strong correlation was found with average channel width with meander parameters in some cases. The bend characteristics i.e., the bank erosion rates along the curved sections of channel was studied with the help of land sat and spot images. Twenty-seven clearly distinct bends were analyzed in detail following the techniques suggested by Nanson and Hickin (1984). It was found that the bank erosion rate depends on the relative bend curvature value r/w where r and w are radius of curvature & average channel width respectively. Average sinuosity was estimated using maps and from sinuosity values it was observed that the river was in highly meandering stage.

Introduction

Alluvial rivers are migratory. An important characteristic of many rivers in the alluvial flood plains is its flow in sinuous path called meandering. Meandering stream is one whose channel alignment consists principally of pronounced bends, the shape of which have been determined predominantly by the varying nature of the terrain through which the channel passes. Water flows freely from one bank to another. The meandering causes the river to leave their original courses, force them to flow along new courses and thus devastating vast areas of land and affecting important and valuable nearby structures such as bridges, railway lines, roads etc. Bend and its migration through erosion-deposition processes in an alluvial river is a characteristic feature and one of the most conspicuous changes affecting fluvial landscapes. With time individual bend become distorted and change their relative positions. The rate of change in position quantitatively may vary from 5 my^{-1} to 150 my^{-1} in both laterally and longitudinally. The proper understanding of meander development and channel pattern changes of alluvial rivers is very important for all engineering projects, which have linkage with rivers.

Meandering is one of the means of a river to achieve dynamic or quasi-equilibrium state. In most of the alluvial rivers, overall meander pattern is irregular and while recognizing this irregularity and variability geo-morphologists have come to look on channel geometry in general and meander patterns in particular as manifestations of a dynamic

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equilibrium between hydrologic regime and local geologic environment. River training and construction of training works are also designed to achieve a proper equilibrium. The more the planned channel pattern, channel geometry, slope conditions etc. correspond to the natural conditions of the river in question the better will be the acceptance of the river for the new semi-artificial state.

Meander geometry

In describing the features of a meandering stream the meander wavelength (M_L), the meander belt (M_B), channel width (W), radius of curvature, (r_c) and the sinuosity ratio (S_r) are important. But traditional meander scale and shape indices are the meander wavelength and radius of curvature (Ferguson, 1975). Engineers and research workers have attempted to relate these strictly average characteristics of a natural stream to the flow characteristics and with each other. Axial wavelength and radius of curvature is difficult to measure in irregular bends, so that a careful operational definition is a must. Nevertheless the important relationships are based on these indices (Richards, 1982).
Inglis (1949) correlated the meander wave length and meander belt analyzing the huge amount of field data, with the maximum discharge as,

$$M_L = 29.60\ Q^{0.5} = 6.06\ W \qquad (1)$$

$$M_B = 84.70\ Q^{0.5} = 17.38\ W \qquad (2)$$

Where, Q = maximum discharge in cfs, M_L, M_B and W in feet.
Dury (1955) relates the meander wavelength to bankfull discharge as

$$M_L = 54.3\ Q_b^{\ 0.5} \qquad (3)$$

Although Ackers and Charlton (1970) showed the role of bankfull discharge on meander wave length, a stronger correlation exists with the channel width (Schumm, 1969).
Zeller (1967) and Leopold and Wolman (1960) found that

$$M_L = 7.32\ W^{1.11} \qquad (4)$$

and $M_L = 12.13\ W^{1.09}$ (5)

Neglecting the exponents, the linear regression

$M_L = 12.34$ W is quite satisfactory. (6)

In the above equation, the constant (co-efficient) is very close to 4*π (12.57), which suggests a close link with the riffle-pool wavelength (2*π*W), because in a regular meander bend there are two riffle-pool cycles to one bend with pools at the apices and riffles in the inflections (Richards, 1982). The strong correlation with width reflects the role of secondary circulation controlled by channel size, channel width is caused by an interaction of discharge and bank material properties. Discharge and sediment jointly control width, which then controls the wavelength of secondary flow (Yalin, 1971) and hence that of the riffle-pool sequence and meaner bends. Hence the correlation with the discharge are poorer than that with the channel width.

Hossain (1987) analyzed the data of the river Ganges for the period of 1965 to 1976 and found the following relationships :

$$M_L = 9.76\,W \quad \text{and} \quad M_B = 3.6\,W \quad \text{for } 1956 \tag{7}$$

$$M_L = 7.46\,W \quad \text{and} \quad M_B = 3.0\,W \quad \text{for } 1967 \tag{8}$$

$$M_L = 7.60\,W \quad \text{and} \quad M_B = 2.8\,W \quad \text{for } 1976 \tag{9}$$

Sinuosity of thalweg line, S_r = 1.27 and 1.26 for the year 1967 and 1976 respectively.

Hossain (1989) upon analyzing the field data for the period of 1973 to 1987 of the river Ganges found the following correlation:

$$M_L = 24.27 + 1.23\,M_B \text{ when } S_r < 1.2 \tag{10}$$

$$M_L = 15.7 + 0.96\,M_B \text{ when } S_r > 1.2 \tag{11}$$

The M_L and M_B value are governed by the several factors such as sediment size, nature of geologic strata, sediment load, form of hydrographs, etc., and hence the co-efficient in the above equations are expected to depend on them (Blench, 1957).

The study also indicates that experimental results obtained under laboratory conditions are not always transferable to seemingly analogous field conditions (Chang, 1988).

River bend characteristics

A feature typical of meandering rivers is the bend of varying size and shape. In each bend a helical current is produced and causes a variation in bed level. Most bends are the part of a meander or deformed meander system. Bends are normally formed as a result of the natural tendency for sinuous flow in alluvial channel when the slope of the river is less than $S=0.0017/Q^{1/4}$ The shape of bends varies from beautifully symmetrical patterns to the deformed bends encountered most frequently in nature. Bends can be designated by the curvature (r_c / W). In a sample of 50 rivers differing in size as well as in physiographic province, Leopold et al, (1964) found that 2/3 of the ratios for r_c / W value were in the range 1.5 to 4.3 with a median value of 2.7. In view of the this striking geometric regularity of winding rivers, they suggested that meanders are no accident and they appear to be in the form in which a rivers does the least work in turning.

Chang (1984a) analyzed the meander curvature and other geometric features using an approach together with relations for flow continuity, sediment load, resistance to flow, bank stability, and transverse circulation in channel bends. The analysis established the maximum curvature (r_c / W) for which a river does the least work in turning and the average value of that curvature was 3. Sinuosity varies among the meandering river, but r_c / W value remains similar in the different river bends of different sizes and physical settings (Leopold and Langbein, 1966; Leopold and Wolman, 1957).

Radius of bend curvature

Since bends are not, in general, circular arcs (Ferguson, 1975), difficulties arises in defining and measuring radius of curvature. A sine-generated curve (Leopold and Langbein, 1966) incerases in curvature towards the apex, so it requires description of minimum (apex) radius and mean radius. The latter is the best defined by replacing the continuous curve of the bend by equally spaced points (about one channel width apart), connecting these by straight segments, and finding the centroid of the polygon defined by their perpendicular bisector. This represents the mean center of curvature, which can subsequently be used to initiate radii in order to find the mean radius (Hickin, 1974 ; Goudie et al., 1990). But in 1983, Nanson and Hickin redefined 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 the other passes through x_1y_1, x_3y_3 and x_5y_5. Where x_3y_3 are the bend axis (point of maximum curvature). The digitizing interval, Δx for the five sets of coordinates is equal to the mean channel width measured at straight reaches (usually at planform inflection points). This measure of bend radius reflects both the strong curvature at the bend axis and the broader sweep at the limbs of the bends.

The coordinates for this calculation are located on the convex bank because it represents a bend curvature intermediate to those defined by the concave bank line and the scroll bar marking the beginning of the migration period in question.

River bend migration

A river is the author of its own geometry. It is adjusted in the long term. The adjustments include channel geometry, slope, meandering pattern etc. (Chang, 1984b). In this process, the migration at meander bends is a characteristic feature of alluvial rivers and one of the most conspicuous changes affecting fluvial landscapes. River bends shift laterally by erosion of the concave bank and concomitant deposition at the convex bank. This phenomenon is of considerable scientific and engineering significance. An attempt is of great importance to systematically examine the range of migration rates on a variety of river types. At present there is no means of explaining or predicting channel migration rates for selected reaches of variety of rivers where rates have never been actually measured. In general, the rate of channel migration (M), is likely to depend primarily on the variables as stream power (Ω), opposing force of boundary area (y_b) bank height (h), bend radius (r_c) & channel width (W).

But detailed measurements on the Beatton river, Canada, clearly show that channel bend migration rates are strongly controlled by bend curvature (Nanson & Hickin, 1983). The basic relation is, the migration rate,

$$M = f (r_c / W) \qquad (12)$$

Sediment transport also influence the channel migration rates. However, maximum curvature in terms of r_c / W is found to be a function of the channel slope, discharge, sediment size, and the average width-depth ratio (Chang, 1984b). Hickin and Nanson (1975) found in a survey of ten point bar complexes on the Beatton river in British Columbia, Canada, that the rate of channel bend migration reached a maximum value when the value of r_c/w approximated 3. The rate of channel migration rapidly declined for bends with values of r_c / W greater or less than 3. The radius of curvature used in this study was mean radius defined in previous Art.

In the typical meander bends maximum curvature for the value of r_c / W ranges from 2.2 to 4.0 ; it varies in direct portion to the channel slope or the width-depth ratio (Chang, 1984 b). Chang (1984 b) found analytically that the maximum channel bend curvature occurred when r_c / W = 3.2 and the median value is about 3.0. On the other hand Leopold and Langbein (1966) found it as 2.7 from field measurements. When the width-depth ratio is small, possibility of development of very sinuous channels. Width-depth ratio is found to be an important factor governing meander geometry (Hey, 1976; Lane, 1957 ; Schumm, 1973). While the r_c / W ratio is remarkably constant, it does indicate the possibility of more acute bends for streams of flatter slopes which are also associated with a smaller width-depth ratio.

In 1983, Nanson and Hickin redefined the radius of curvature in the form $r_m = (r' + r'')/2$ and analyzed additional field data relating to bend migration rates for the Beatton river which confirm the form of the relationship the writers described in 1975. He also found that the channel migration is a discontinuous process within any single bend and the river is down cutting at a rate of about 2 my^{-1} not at 10 my^{-1} as reported earlier.

In 1984, Nanson and Hickin studied on two different river reaches in Canada and tried to establish the general relationship between bend migration rates (M) and bend curvature ratio (r_c / W) for all field sites. He found that maximum migration rates occurred at 2.0 $< r_c/W <$3.0 and when $r_c/W <$2.0 as the bend curvature tightens, migration rates per unit channel width (M/W) quickly tends to zero at r_c / W= 1.0. These results are also similar to the results shown in 1983 (Nanson & Hickin).

In 1984, Ferguson suggested an alternative conceptually superior and convenient (as his opinion) method for measuring the radius of curvature and an appropriate idea about the bend curvature or simply curvature. But on the conclusion Nanson and Hickin (1984b) opined that in measuring the radius of curvature their method is less complicated and comparatively accurate.

Hooke (1987) critically reviewed the Nanson and Hickin's (1984b) consideration that bend migration rate is dominantly governed by the bend curvature (r_c / W), and investigated further for the controlling variables of the river bend in southern Devon, England. Finally he concluded that they were unaware of any attempt to examine systematically a range of migration rates on a variety of types. He found that, there eight independent variables governing the bend migration rates of which drainage area and silt-clay percentage are the dominant variables, though the curvature of the bend is important in influencing the distribution and magnitude of erosion and, therefore migration of the bend. Klaassen and Vermeer (1988) studied the channel characteristics of the braided Jamuna river in Bangladesh. They found that, the bend characteristics are approximately similar to meandering rivers, which may indicate that individual channels of a braided river may to some extent be comparable to the single channel of meandering rivers. Klaassen and Masselink (1992) studied the bank erosion rates for Jamuna river and found the erosion rates much faster than that predicted by Nanson and Hickin (1984b). They also found a negative correlation between the relative bend curvature (r_c / W) and erosion rate (E/W). There was no definite tendency for very sharp bends (r_c/W < 2.5) to have smaller erosion rates as was observed by Nanson and Hickin (1984b) for meandering rivers.

The meander parameters (M_L, M_B, S_r) and their interrelationships

From the bank line maps (compiled from the spot imageries, landsat images and SOB maps) of the river Gorai-Madhumati for the year 1953, 1973, 1980 and 1990, computations of meander length (M_L), meander belt (M_B) and channel width (W) were made as shown in Tables 1,2,3 and 4. In this river the number of complete bends were 12 in 1953 and 13 in 1973, 1980 and 1990. Relationships were established between meander length (M_L), and channel width (W) for each meander. From the analysis average relationships were as

$$M_L = 13.60\ W \quad \text{in} \quad 1953 \qquad (13)$$
$$M_L = 12.87\ W \quad \text{in} \quad 1973 \qquad (14)$$
$$M_L = 14.32\ W \quad \text{in} \quad 1980 \qquad (15)$$

and
$$M_L = 13.26\ W \quad \text{in} \quad 1990 \qquad (16)$$

Power relationships between meander wave length and channel width were also established and found to be

$$M_L = 14.89\ W^{1.106} \quad \text{for} \quad 1953 \qquad (17)$$
$$M_L = 5.94\ W^{0.164} \quad \text{for} \quad 1973 \qquad (18)$$
$$M_L = 8.51\ W^{0.51} \quad \text{for} \quad 1980 \qquad (19)$$

and
$$M_L = 6.38\ W^{0.297} \quad \text{for} \quad 1990 \qquad (20)$$

Which compare quite favorably with the Leopold & Wolman (1960) and Zeller (1967) relationships. General relationships between meander wave length and channel width for the entire period were also established (fig.1) and found to be

$$M_L = 0.33+11.90W \text{ and} \qquad (21)$$
$$M_L = 6.613W^{1.10} \qquad (22)$$

Effort has also been made to find the relationships between meander wave length and discharge. The relationships (through regression analysis) with the average past annual maximum discharge were as

$$M_L = 191.31\ Q_{ma}^{-0.41} \quad \text{with Gorai Rly bridge data and} \qquad (23)$$

$$M_L = 7.86\ Q_{ma}0.47 \quad \text{with the Kamarkhali transit data} \qquad (24)$$

Relationships were also developed between meander belt (M_B) and channel width (W) for the river Gorai-Madhumati, using the meander bends considered. The relationships were

$$M_B = 8.62\ W \quad \text{for} \quad 1953 \qquad (25)$$
$$M_B = 10.09\ W \quad \text{for} \quad 1973 \qquad (26)$$
$$M_B = 11.75\ W \quad \text{for} \quad 1980 \qquad (27)$$
and
$$M_B = 11.68\ W \quad \text{for} \quad 1990 \qquad (28)$$

Meander belt vs. Q_{ma} (annual maximum discharge) relationships were also established and found to be

$M_B = 0.244\ Q_{ma}^{0.32}$ with Gorai Rly bridge data and (29)

$M_B = 6.66\ Q_{ma}^{-0.057}$ with Kamarkhali transit data (30)

General relationships between meander belt and channel width (fig.2) were also established and found to be

$M_B = 0.18+12.32W$ and (31)

$$M_B = 5.07\ W^{1.10} \qquad (32)$$

An interesting relationship between average meander wave length and meander belt was observed form the study. The relationships were as follows :

$$M_L = 1.576\ M_B \quad \text{in} \quad 1953 \qquad (33)$$
$$M_L = 1.275\ M_B \quad \text{in} \quad 1973 \qquad (34)$$
$$M_L = 1.218\ M_B \quad \text{in} \quad 1980 \qquad (35)$$
And
$$M_L = 1.135\ M_B \quad \text{in} \quad 1990 \qquad (36)$$

From the above relations it was observed that the ratio between meander wave length and meander belt gradually decreased with time. That is, as meander belt increases, the meander length decreases and tend towards stable form.

From the plan maps, the sinuosity of the river were found to be

$$S_r = 212/116.5 = 1.82 \quad \text{in} \quad 1953 \qquad (37)$$
$$S_r = 199/118.5 = 1.69 \quad \text{in} \quad 1973 \qquad (38)$$
$$S_r = 210/117 = 1.79 \quad \text{in} \quad 1980 \qquad (39)$$
and
$$S_r = 212/117 = 1.812 \quad \text{in} \quad 1990 \qquad (40)$$

Analysis of bend migration

The aim of analysis of bend migration rates was to develop a relationship between bank erosion and relevant channel characteristics that can be used for prediction purposes. This study has yielded a relationship between the relative bend erosion rate E/W and the relative radius of bend curvature (r/W). That is, E/W is expressed as the function of r/W

$$E/W = f(r/W) \qquad (41)$$

Bends were selected using the following criteria:

- Only migrating bends
- No bifurcation or confluence in the vicinity of the bend
- No widening or narrowing on sides of the channel
- Minimum width should be 0.5 mm on the satellite image prints, which was 25 m in reality. This smallest distance one can measure with reasonable accuracy.

For the 27 selected bends of Gorai-Madhumati river, the width, the bend migration and the radius of curvatures were obtained from the classified images [plan maps (SOB), spot and landsat imageries (SWMC and SPARRSO)] of the year 1953, 1973, and 1990. These data were plotted in several graphs (not shown) to investigate the possible relationships. The observed bend migration rates were analyzed within a frame work similar to the previous work by Nanson and Hickin (1984) for meandering river by establishing the relation between the bend migration rates and the relative bend curvature of the channels.

The migration rate i.e., migration per year (M/y) have been plotted against the relative radius of curvature (r/W) for the widths ranges from W=0-0.35 Km and W= 0.36-0.5 Km (not shown). The movement of the apexes i.e., the bend migration rates were ranges from 5-142 m/year. The maximum bend migration rate observed was 142.6 m/year and the average bend migration rate was about 40 m/year.

In fig. 3, the bend migration rate (M/y) has been plotted vs. relative bend curvature (r/W). This is a general relationship. In figs. 4 & 5 the relative bend migration (M/W) vs. relative bend curvature (r/W) have been plotted. Upon inspection of the figures the following observations can be made:

(a) There is a negative correlation exists between the relative bend curvature (r/W) and the bend migration rate (M/y). Low relative bend curvature lead to relatively fast migration rates up to the certain extent and vice versa. But beyond the certain values of r/W (>3), the migration rates again decreases.

There was a tendency for relatively very flat bends (r/W>3) to have smaller migration rates, as was observed for meandering rivers by Nanson and Hickin (1984). Klaassen and Masselink (1992) found slightly different tendency i.e., increase in relative erosion with the increase in relative bend curvature to the certain extent in the studies carried out for FAP-21/22 in the braided river Brahmaputra-Jamuna in Bangladesh.

An interesting relationship have been observed in all the figures is that, the bend migration rates gradually increases for r/W>1 and is maximum for r/W=2.5 (app.) and then gradually decreases for r/W>2.5. That is the bend migration rate was maximum for the relative bend curvature value (r/W) of approximately 2.5.

The same relations were also found by Nanson and Hickin(1984b) and Klaassen & Masselink (1992). The reason for the maximum bend migration due to erosion at r/W = 2.5 is that for this relative curvature the velocities along the bank (outer) are largest. A further decrease leads to the main flow going through the inner bank, thus reducing the outer bank erosion. Klaassen and Masselink (1992) analyzed the data from the Jamuna river and found a similar shape of the bank erosion rates.

b) Klaassen & Masselink (1992) found for the Jamuna river that the *relative erosion* rates by large channels was relatively small. But in the present study it was observed that there was no definite trend that the bend migration rates for the relatively large channels was small. In a few cases it was observed that the bend migration rates for large channels are relatively more than the narrow channels.

The trend found in the present study that maximum bend migration rates occurred at the relative bend curvature, r/W = approximately 2.5 which comply with the Bagnold's (1960) inference. He made a theoretical evaluation indicating that an optimum channel curvature should exist with a ratio of bend radius, r to channel width, W between two and three; i.e., r/W = 2 to 3.

The flow in eroding outer bends is affected by the channel characteristics like discharge, dimensions and slope. In bends, also the bend characteristics, in particular the radius of the bend is important, together with the overall planform of the river. In the bend a helical flow pattern is generated that is of great importance for the resulting bed topography. The bank properties include bank material weight and texture, shear strength and cohesive strength, physicochemical properties, bank heights and cross sectional shape, ground water levels and permeability, stratigraphy, tension cracks, vegetation and constructions. Many of these factors are only approximately known. The influence of water quality is assumed to be of minor importance for the rivers in Bangladesh.

Conclusion

Based on the present study, the following conclusions can be drawn :
The Gorai-Madhumati is a highly meandering river from its early stage. Its down stream part is more meandering than its upstream part. There are certain reaches in this river which is remaining unchanged for long time due to human intervention or geological constraints. Meander wavelength and meander belt was strongly correlated with the average channel width in a few cases. There also good correlation between annual maximum discharge and meander parameters. It was also observed that the ratio of average meander wavelength to average meander belt was gradually decreasing with time. The Gorai-Madhumati has a tendency to reduce her sinuosity in the short term. But in long term no appreciable change in sinuosity observed.The bends were migrating in an unpredictable manner. The migration rates were ranges from 5-142 m/year and the average migration rate was about 40 m/year. The bend migration rate depends on the relative bend curvature (r/W). The maximum migration rate occurred with the relative bend curvature value r/W = 2.5(approximately). The overall sinuosity of the river was found to be in the range of 1.69 to 1.82 which is greater than 1.5, which indicates clearly a meandering river. Sinuosity at the lower reach was found more than that at upper reach i.e., lower part is more meandering as observed also from maps.

Recommendations for the future study

In view of the limitations faced in the present study, the following recommendations were made for future study :

1. More good quality maps (preferably spot imageries) should be collected with the close time intervals to study the formation of cut-offs and direction and phases of meander development.
2. Frequent and detailed field investigations are required to study the bend characteristics accurately and exhaustively.

Acknowledgement

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Table 1: Meander Parameters, 1953

SL No	Meander Wave length, M_L, in km.	Meander belt, M_B, in km	Channel width, W in km.
1.	3.725	1.65	0.38
2.	6.00	3.60	0.49
3.	8.90	5.60	0.47
4.	10.50	8.15	0.495
5.	6.30	3.85	0.337
6.	5.00	4.15	0.445
7.	3.30	2.10	0.362
8.	5.30	1.60	0.321
9.	7.00	2.60	0.375
10.	3.70	3.05	0.362
11.	4.00	2.70	0.437
12.	1.60	2.35	0.337

Table 2; Meander Parameters, 1973

SL No	Meander Wave length, M_L, in km.	Meander belt, M_B, in km	Channel width, W in km.
1.	7.75	4.625	0.40
2.	6.25	3.45	0.40
3.	3.40	1.90	0.325
4.	6.075	3.829	0.390
5.	5.85	5.225	0.435
6.	5.25	3.575	0.48
7.	4.45	6.70	0.58
8.	4.025	4.125	0.45
9.	4.20	4.375	0.393
10.	7.125	7.65	0.415
11.	4.125	2.575	0.356
12.	6.40	2.925	0.341
13.	3.85	2.920	0.366

Table 3 Meander Parameters, 1980

SL No	Meander Wave length, M_L, in km.	Meander belt, M_B, in km	Channel width, W in km.
1.	7.85	4.50	0.41
2.	6.25	3.40	0.38
3.	3.20	1.75	0.340
4.	5.86	3.80	0.36
5.	5.62	5.20	0.44
6.	4.75	4.50	0.262
7.	4.25	5.90	0.412
8.	4.25	5.75	0.406
9.	4.00	4.91	0.380
10.	6.40	8.15	0.425
11.	5.20	2.10	0.337
12.	6.25	2.89	0.30
13.	3.64	2.95	0.295

Table 4 Meander Parameters, 1990

SL No	Meander Wave length, M_L, in km.	Meander belt, M_B, in km	Channel width, W in km.
1.	8.00	4.50	0.425
2.	6.25	3.40	0.370
3.	3.10	1.625	0.341
4.	5.775	3.725	0.340
5.	5.45	5.10	0.44
6.	3.85	4.57	0.475
7.	4.20	6.40	0.44
8.	3.25	5.00	0.431
9.	3.75	5.20	0.347
10.	6.975	8.875	0.560
11.	4.50	2.475	0.218
12.	6.175	2.875	0.245
13.	3.00	2.92	0.216

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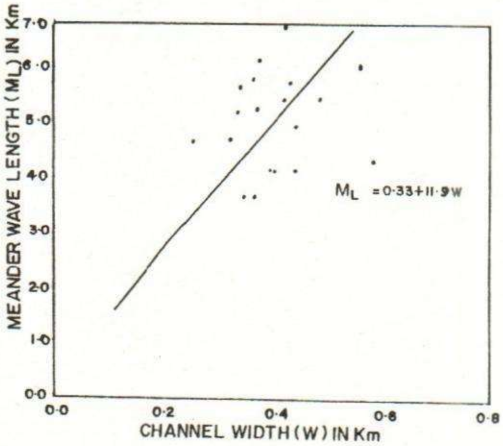

Fig.1: THE GENERAL RELATIONSHIP BETWEEN MEANDER WAVE LENGTH (M_L) AND THE CHANNEL WIDTH (W) (GORAI-MADHUMATI RIVER)

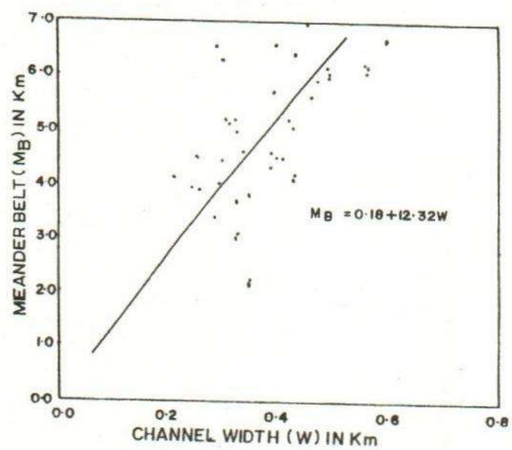

Fig.2: THE GENERAL RELATIONSHIP BETWEEN MEANDER BELT (M_B) AND THE CHANNEL WIDTH (W) (GORAI-MADHUMATI RIVER)

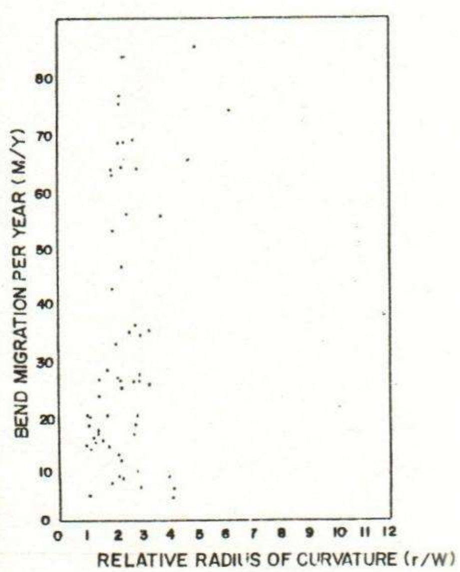

Fig.3: THE GENERAL RELATIONSHIP BETWEEN BEND MIGRATION RATE (M/Y) AND RELATIVE BEND CURVETURE (r/W) (GORAI-MADHUMATI RIVER)

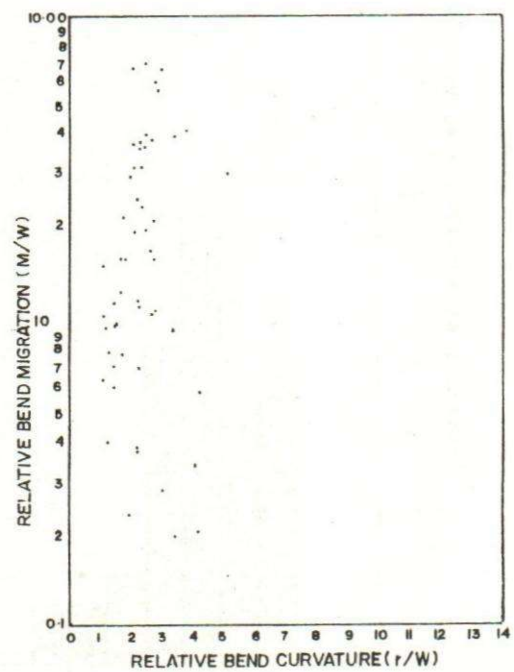

Fig.4: THE RELATIONSHIP BETWEEN BEND MIGRATION RATE (M/W) AND RELATIVE BEND CURVETURE (r/W) (GORAI-MADHUMATI RIVER)

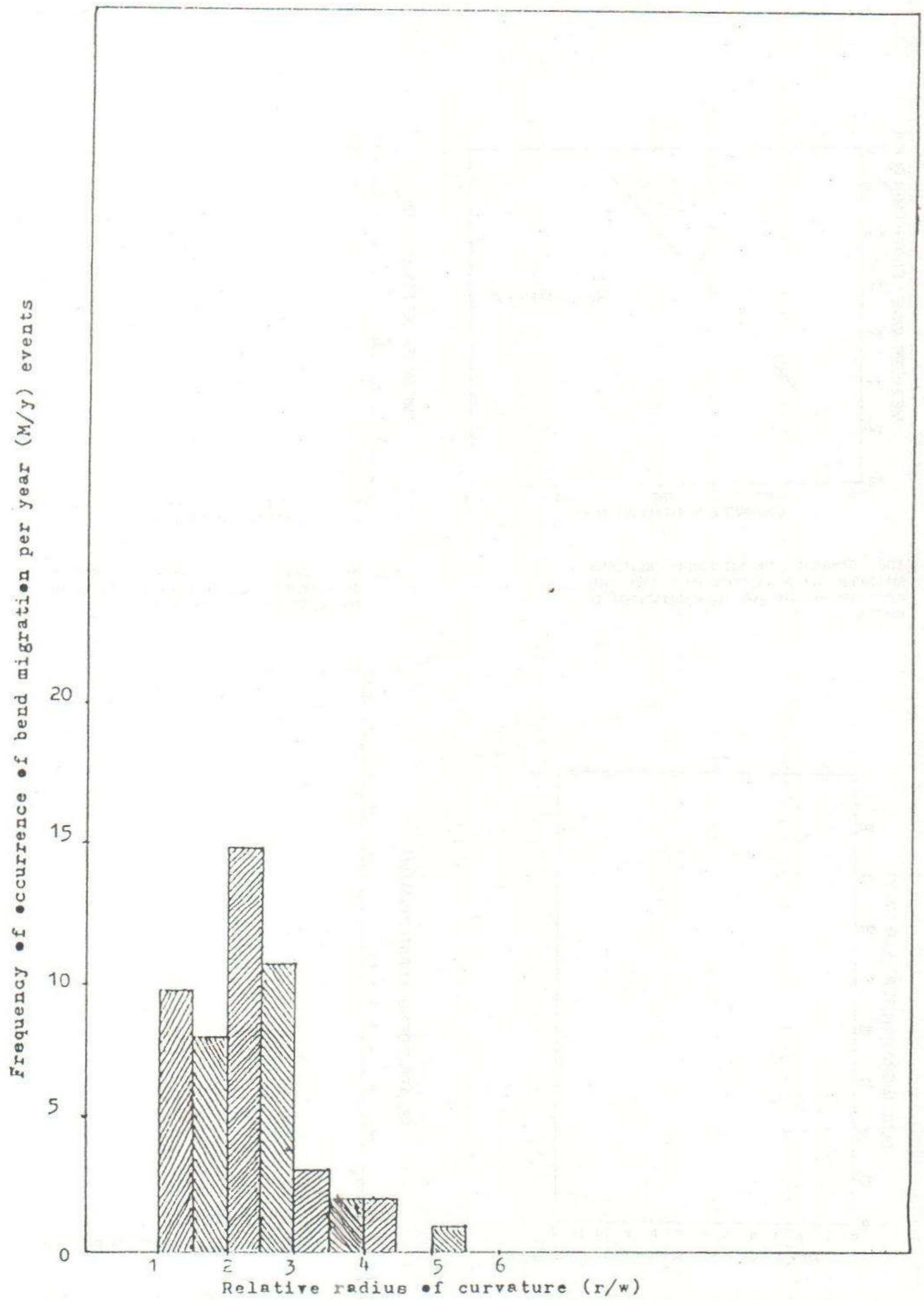

Fig.5: THE HISTOGRAM FOR FREQUENCY OF OCCURRANCE OF BEND MIGRATION (M/Y) AND RELATIVE BEND CURVERATURE (r/W) (GORAI-MADHUMATI RIVER)

LABORATORY INVESTIGATION TO ESTIMATE THE LOCAL SCOUR AROUND BRIDGE PIER

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Abstract

Many experimental investigations of local scour around bridge piers have been conducted so far, which vary widely. Scour around the pier is an important parameter, which affects the stability of bridge and plays an important role for the design and cost estimate of the whole project. A laboratory scale model investigation was conducted to estimate the extent of scour with two pier options ; option-A and option-B. In option-A, it was steel pier with racking pile group of each diameter 2.2 m and circular pile cap & option-B was concrete pier with vertical pile group of 3.0m diameter and hexagonal pile cap. Tests were carried out in two series with changing water depth, velocity and angle of approach flow. It was found to occur more scour around the pile group of larger diameter and it increases with the increase of angle of approach flow. This paper has been prepared from the results and findings obtained from the physical model study of proposed Paksey Roadway Bridge.

Introduction

Roads and Highways Department under the Ministry of Communication, Bangladesh proposed River Research Institute to conduct a physical model study of a roadway bridge having span length 1786m to be constructed on the Ganges River at Paksey. The main objective of this scale model study was to get an insight of the extent and nature of local scour in two pier options for the design finalization of the proposed bridge.

Accordingly a flume study was carried out at River Research institute. Two pier option; option-A and option-B were investigated in this study. In option-A, it was steel pier with four racking piles of each diameter 2.2-m and circular pile cap & option-B was investigated in this study concrete pier with four vertical piles of 3.0-m diameter and hexagonal pile cap. In first series the study was conducted with model velocity 0.53 m/s corresponding to prototype velocity 5m/s, water depth 0.33 m corresponding to prototype water depth 30m and angle of attack 0,5,10,20 and 30 degree. The model velocity 0.32 m/s, water depth 0.11m, angle of attack 0,5,10 and 20 degree was adopted in the second series. Pier option-A and Pier option-B both were tested at a time placing them adjacent to each other. The experiment carried out in a rectangular flume filled with fine sand and the geometric scale was taken 1:90.

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Previous investigation

Hossain (1981) defined local scour as the abruptness in bed near a hydraulic structure due to erosion of bed material by the local flow induced by the structure

According to **Posey (1949), Laursen and Toch (1956), Shen Schinieder and Karaki(1969),** the generation of scour hole is preceded by the appearance of large scale vortex around the structure in the flow. The sudden appearances of the vortex of the structure are the basic mechanism of local scour

Ahmed (1962) conducted experiments on models of several bridge sites of Pakistan. The experiments were carried out so that there was good general movement of the bed. The relationship of discharge intensity to total scoured depth, both around the piers and abutments and in bends, was investigated and he proposed a formula of the following type

$D_t = 1.486\ K.q^{2/3}$

Where the co-efficient K is a function of boundary geometry, shape of pier nose or abutments, characteristics of bed material and distribution of velocity in the channel section at the piers. Ahmed recommended that a value for K = 1.7 to 2.0 be adopted for use in predicting the total scoured depth around piers.

Laursen (1962), Chabert and Engeldinger (1956) investigated the variation in the depth of local scour with the angle of attack of the approaching flow and concluded that the depth of scour increases as the angle of attack of the approach flow increases (after Kabir 1984).

Bruce W. Melville(1984) studied scour depth at cylindrical bridge piers. He established relationship between the normalized maximum equilibrium scour depth (maximum scour depth/pier diameter) and mean flow velocity for different pier size and constant flow depth. He found two scour peaks i,e, threshold peak and transition flat bed peak.

Inglis (1949) developed a relationship from the model study as follows

$D_t/D_b = 2.32(q^{2/3}/b)^{0.78}$

Where, D_t = Total scoured depth below high water level in meter, b = width of pier in m, q = discharge per unit width of the approach flow in m^3/sec per meter width. He also developed a general empirical relationship as follows

$D_t=2[(0.473\ q/f_e)^{1/3}] = 2d_e$. Since Lacey's regime depth, de = $0.473(Q/f_e)^{1/3}$. Where Q is *the maximum discharge in cusecs*, d_e *is the maximum depth of scour below highest* flood level, f_e is equal to $1.76\sqrt{(D_{50})}$

Laursen(1960) found experimentally that with continuous sediment motion, the depth of scour is a function of the depth of flow and geometry of the structure.

Experimental set-up

A Rectangular flume having effective dimension 18 m × 2.2 m filled with fine sand of d_{50} = 0.08 mm and separate water circulation system was used. Two types of pier and one type of sand were used throughout the study. The model was designed to attain average velocity 0.53 m/s and 0.32 m/s corresponding to the average water depth 0.33 m and 0.11 m respectively, ensuring the sediment movement i.e, the model was run with live bed condition. The model was run with steady flow condition, until the equilibrium scour was achieved. Sediment feeding was also done continuously and the rate of feeding was estimated by the Engelund and Hansen formula as follows.

$$q^*_s = 0.05\sqrt{[(s_s - 1) * g * D_{50}^{\ 3}]} * Fr_g^{\ 2} * \tau_*^{\ 1.5}$$

Where q_s^* = q_s/q= dimensionless sediment transport capacity per unite width of channel; q_s = sediment discharge per unit width; q = water discharge per unit width; $S_s = \rho_s/\rho$ = sediment specific gravity; ρ_s = sediment mass density; ρ = water mass density; g = gravitational acceleration; D_{50}= median grain diameter. Fr_g = densimetric Froude number; τ_* = Shield's parameter. In the first test series this feeding rate was 0.33 m^3/h and in the second Series it was 0.014 m^3/h. Water and sediment were independently introduced into the flume at the upstream. The total study was conducted in two series changing the water depth and velocity with different angle of flow.

Test scenarios

Total ten tests were carried out of which one test was calibration of the model and rest nine tests were application tests. Five tests were conducted with depth average velocity 0.53m/s and average water depth 0.33m in the model, the angle of approach flow were 0,5,10,20 and 30 degree. Where as four tests were conducted with depth average velocity 0.32m/s and average water depth 0.11m in the model, the angle of approach flow were 0,5,10, and 20 degree. For each application test the model was run untill the model reached in equilibrium condition. Different tests scenario are shown in **table 1.**

Table 1: Different tests Scenario

Test Type	Test No.	Angle	Discharge in model	Desired velocity in model	Desired velocity in proto	Initial bed level	Water level
			(lps)	(m / s)	(m / s)	(mPWD)	+ (mPWD) +
Calibration	T1	No struct.	384	0.53	5	-15	15
Application (Series-1)	T2	0^0	384	0.53	5	-15	15
	T3	5^0	384	0.53	5	-15	15
	T4	10^0	384	0.53	5	-15	15
	T5	20^0	384	0.53	5	-15	15
	T6	30^0	384	0.53	5	-15	15
Application (Series-2)	T7	0^0	77	0.32	3	-15	-5
	T8	5^0	77	0.32	3	-15	-5
	T9	10^0	77	0.32	3	-15	-5
	T10	20^0	77	0.32	3	-15	-5

Discussion of test results

During the investigation, scour measurements were done in the pile group as well as in the individual piles and the analysis were done from the maximum observed value. Relationship was made between approach flow angle and the relative scour depth. It appears that the scour depth increases with the increase of angle of attack and maximum scour observed at the larger diameter pile group. Maximum scour was found to occur at different location in different tests. Scour depth was higher at the downstream transect of the pile group than that of the upstream transect. A typical equilibrium situation is shown in figure-1. Rare pile experienced more scour than front pile. Between these two test series, higher scour was observed at higher depth and higher velocity condition. Summary of different tests are shown in table-2

Conclusion

It was observed from the model investigation that Pier option-B experienced more scour than that of the Pier option-A, i,e. higher scour value at larger diameter pile in both series of tests. Maximum net scour observed about 22 m. Maximum scour was found to increase with the increase of angle of attack. Scour was higher at the rare pile than that of the front pile. But the limitation of this study was that the angle of attack kept within 0-30 degree. It is recommended to carry out further study by increasing the angle of attack upto 90 degree.

Fig.1 Typical Equilibrium Scour at u/s Transect and d/s Transect of pier

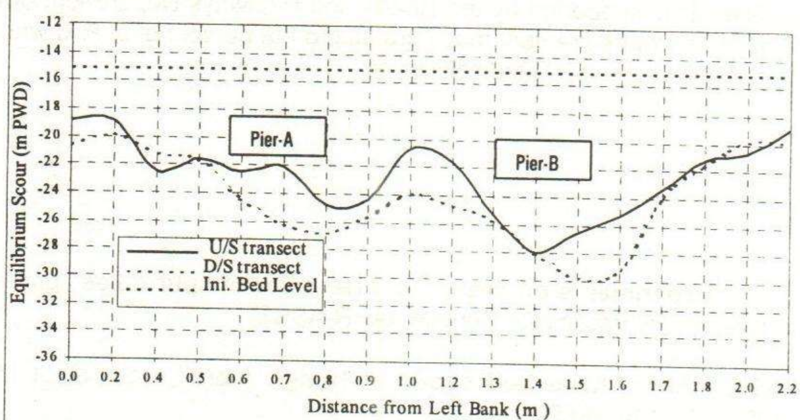

Table 2: Summary of test results

Test Series	No. of Tests	Pier orie. With flow	Water Level at Pier section	Ini. Bed Level	Max. scour (Trans-1)	Loction of Max. scour frm. LB (Trans-1)	Max. scour (Trans-2)	Loction of Max. scour frm. LB (Trans-2)	Indv. Pile Scour A1 (dry bed)	Indv. Pile Scour A2 (dry bed)	Indv. Pile Scour A3 (dry bed)	Indv. Pile Scour A4 (dry bed)	Indv. Pile Scour B1 (dry bed)	Indv. Pile Scour B2 (dry bed)	Indv. Pile Scour B3 (dry bed)	Indv. Pile Scour B4 (dry bed)
		(deg.)	m pwd	m pwd	mpwd	m	m pwd	m	m pwd	m pwd	mpwd	m pwd	m pwd	m pwd	m pwd	m pwd
	T1	(calibration)	15	-15												
1	T2	0	15	-15	-27.9	1.6	-30.2	1.5	-24.8	-27.0	-26.6	-25.4	-27.0	-28.8	-28.6	-28.1
	T3	5	15	-15	-27.9	1.4	-29.7	1.5	-24.8	-26.1	-25.6	-25.2	-28.1	-29.0	-28.0	-27.7
	T4	10	15	-15	-28.4	1.6	-30.6	1.5	-26.1	-27.3	-27.9	-26.1	-28.8	-31.1	-29.3	-28.8
	T5	20	15	-15	-31.5	1.6	-33.3	1.5	-28.4	-28.8	-30.7	-29.3	-32.6	-33.7	-34.4	-32.4
	T6	30	15	-15	-33.3	1.5	-35.6	1.6	-30.0	-30.6	-31.7	-29.5	-34.7	-36.9	-36.5	-36.0
2	T7	0	-5	-15	-23.4	1.6	-23.4	1.5	-20.7	-20.9	-20.5	-21.1	-24.1	-24.5	-24.3	-24.8
	T8	5	-5	-15	-24.8	1.6	-25.2	1.5	-21.6	-21.2	-23.2	-22.2	-25.8	-25.8	-26.0	-25.5
	T9	10	-5	-15	-23.4	1.6	-26.6	1.5	-22.1	-22.8	-24.3	-22.7	-26.1	-27.2	-26.1	-26.1
	T10	20	-5	-15	-23.9	1.5	-23.9	1.5	-21.1	-22.3	-22.9	-22.2	-23.9	-25.2	-25.0	-23.7

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INDIRECT ESTIMATION OF SOIL HYDRAULIC PROPERTIES FROM BASIC SOIL DATA USING PEDOTRANSFERFUNCTIONS

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Abstract

This paper describes an indirect estimation of soil hydraulic properties from laboratory measured basic soil data of the Khulna Jessore Drainage Rehabilitation Project (KJDRP) area. Soil hydraulic properties, namely, soil Moisture Retention Characteristics (MRC) and Hydraulic Conductivity (K) are very often represented by parametric functions, known as parametric models. The parameters of the van Genuchten's MRC model and Gardner's K-model were obtained by using Pedotransferfunctions that relate model parameters in a multiple regression analysis to the basic soil data, for example soil texture, bulk density and organic carbon content. Smooth sigmoidal MRC curves were successfully fitted to some of the retention data by nonlinear optimization techniques using computer model RETC. These curves appear to be typical representation of MRC of these soil and hence leading to a good estimate of K. The result can provide easier means to have a better insight into the physical processes of the soil, which is essential for properly estimating drainage criteria and for evaluating the sustainability of drainage system.

Introduction

Soil Hydraulic Properties, namely, Moisture Retention Characteristics (MRC) and Hydraulic Conductivity (K) play vital role in water and solute movement through soil and hence are of crucial importance for a drainage project. Direct estimation of MRC and K is difficult, time consuming, expensive and labour intensive, especially for a large area e.g. watershed, regional or national scale. Although in the past indirect methods, which predict the hydraulic properties from more easily measured soil-water retention data, did not receive much attention. But these "predictive estimating methods" can provide reasonable estimate of soil hydraulic properties with considerably less effort and expense. Many of the disadvantages of the direct techniques do not apply to the indirect techniques. The usefulness of predictive estimates depends on the reliability of the correlation or transfer function, and on the availability and accuracy of the easily measured soil data. The estimate functions are often called Pedotransferfunctions because they transfer measured soil data from one soil to another, using pedological characteristics. Pedotransferfunctions are usually based on statistical correlation between soil hydraulic properties, particle size distribution, and other soil data, such as bulk density, clay mineralogy, cation exchange capacity, and organic carbon content. The development of Pedotransferfunctions offers promising prospects for estimating soil hydraulic properties over large areas without extensive measuring programs. Such Pedotransferfunctions are only applicable to areas with roughly the same parent material and with comparable soil forming processes. Vereecken et.al. (1992) concluded that errors in estimated soil water flow were more affected by inaccuracies in the Pedotransferfunctions than by errors in the easily measured soil characteristics.

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Study area

The study area comprises polder-27 at Baradanga, Tiabunia and Kowratala regulator sites of Dumuria thana and Soilmari and B-5 link khal sites of Batiaghata thana of Khulna districts under KJDRP area.

Theoretical aspects of the study

Moisture Retention Characteristics (MRC)

MRC can be explained as a function of free energy in soil. The free energy can be determined as the attraction of the soil for water. If the water content is higher then the attraction of the soil for water is smaller. The attraction for water by soil is largely a matter of surface relationships of soil particles also. With analogous to acidity measurement index p^H, free energy can be defined by logarithmic function:

$$P^F = \log|h| \tag{1}$$

Where h is the pressure head in cm.

The force that keep the soil and water together can be described by the relationship between soil water content (θ) and matric potential. It can conveniently be described by the relationship between θ and P^F as well. It is one of the basic hydraulic properties of soil as such it has application in predicting infiltration and drainage and other soil physical properties like unsaturated hydraulic conductivity.

Model describing MRC

The model used for the present study is van Genuchten model (van Genuchten, 1985);

$$\theta(h) = \theta_r + \frac{\theta_s - \theta_r}{[1 + \alpha h^n]^m} \tag{2}$$

Where the parameters are as follows;

$\theta(h)$	=	volumetric soil water content at pressure head h [$cm^3 cm^{-3}$];
θ_S	=	the saturated volumetric soil water content [$cm^3 cm^{-3}$];
θ_r	=	the residual volumetric soil water content [$cm^3 cm^{-3}$];
α	=	the inverse of the air entry value [cm^{-1}] and
n, m	=	shape parameters of MRC curve.

Hydraulic conductivity (K)

Hydraulic Conductivity (K) is the single most important physical properties of soil, which is the determinant of moisture movement in soil. Moisture movement in homogenous and isotropic porous media during transient one dimensional flow can well be described by Richard's equation (Richard's 1931);

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z}\left[K(\theta)\left\{ \frac{\partial h}{\partial z} + 1 \right\} \right] \qquad (3)$$

Where,

z	=	vertical coordinate defined as positive upward [cm];
t	=	time [days] and
$K(\theta)$	=	hydraulic conductivity [cm day^{-1}]

By introducing differential water capacity $C(h) = \partial\theta/\partial h$, which represents the slope of the MRC curve, and by expressing the hydraulic conductivity as a function of the pressure head, the above equation can be rewritten as follows;

Equation (4) is applicable for both unsaturated and saturated flow conditions.

Model describing K

The following empirical model of Gardner (1958) was used for the study;

$$K(h) = \frac{K_{sat}}{1 + Bh^{n}} \qquad (5)$$

Where,

K_{sat}	=	the saturated hydraulic conductivity [cm day^{-1}] and
B	=	shape parameter of hydraulic conductivity function.

Materials and method

Data acquisition

The basic soil data, soil texture i.e. percentages of sand, silt & clay and bulk density were obtained from the test results of undisturbed soil samples of the study area. These tests were conducted at the Soil Mechanics Laboratory of Geotechnical Research Directorate of River Research Institute. Soil texture was estimated by sieve analysis and hydrometer method (for finer particles) and bulk density was estimated from Unconfined Compression Test (UCT).

Pedotransferfunctions

The following Pedotransferfunctions of Vereecken (1989) were used to estimate the parameters of Equation (2);

$\theta_s = 0.81 - 0.283*(\text{Bulk density}) + 0.001*(\%\text{Clay})$

$\theta_r = 0.015 + 0.005(\%\ \text{Clay}) + 0.014(\%\ \text{Organic Carbon})$

$\ln(\alpha) = -2.486 + 0.025*(\%\text{Sand}) - 0.351*(\%\ \text{Organic Carbon}) - 0.2617*(\text{Bulk density}) - 0.023*(\%\ \text{Clay})$

$\ln(n) = 0.053 - 0.009*(\%\ \text{Sand}) - 0.013*(\%\ \text{Clay}) + 0.00015*(\%\ \text{Sand})^2$

For hydraulic conductivity model parameters the following Pedotransferfunctions were used;

$\ln (K_{sat}) = 20.62-0.96*\ln(\% \text{ Clay})-0.66*\ln(\% \text{ Sand}) - 0.46*\ln(\%\text{Carbon})$
$- \quad 8.43*(\text{Bulk density})$

$\ln (B) = -0.73-0.01877*(\% \text{ Sand}) + 0.058*(\%\text{Clay})$

$\ln (n) = 1.195 - 0.196*\ln(\% \text{ Clay}) - 0.056*(\% \text{ Silt})$

The parameters obtained by using the above Pedotransferfunctions for equation (2) and (5) are shown in Table.1 and Table.2. Figures.1 through 5 are smooth MRC and K vs. P^F curves constructed by parameters fitting.

Table 1: Estimated parameters of van Genuchten model

Name of the site	Depth of soil profile	θ_s	θ_r	α	n
Dumuria Soil Site:Polder-27 Baradanga	5 – 7 feet	0.36	0.08	0.05	0.82
	10 – 12 feet	0.37	0.172	0.003	0.87
	15 – 17 feet	0.35	0.16	0.004	0.90
	20 – 22 feet	0.35	0.1	0.041	0.80
	25 – 27 feet	0.41	0.17	0.004	0.84
	30 – 32 feet	0.33	0.11	0.041	0.77
	35 –37 feet	0.36	0.07	0.05	0.88
Dumuria Tiabunia	5 – 7 feet	0.33	0.08	0.08	0.86
	10 – 12 feet	0.37	0.24	0.243	0.87
	15 – 17 feet	0.33	0.075	0.075	0.85
	20 – 22 feet	0.37	0.19	0.192	0.846
	25 – 27 feet.	0.36	0.06	0.06	0.91
	30 – 32 feet	0.33	0.07	0.07	0.85
	35 –37 feet	0.36	0.06	0.06	0.92
Dumuria Kowratala Regulator	5 – 7 feet	0.45	0.085	0.051	0.83
	10 – 11 feet	0.44	0.46	0.000002	0.91
	11 – 12 feet	0.45	0.34	0.00006	0.87
	15 – 17 feet	0.43	0.28	0.0002	0.85
	20 – 22 feet	0.29	0.09	0.038	0.85
	30 – 32 feet	0.32	0.075	0.04	0.89
Batiaghata B-5 Link Khal	5 – 7 feet	0.43	0.14	0.009	0.89
	10 – 12 feet	0.42	0.22	0.001	0.87
	15 – 17 feet	0.43	0.27	0.00045	0.82
	20 – 22 feet	0.38	0.216	0.00098	0.88
	25 – 27 feet	0.37	0.24	0.00092	0.82
	30 – 32 feet	0.32	0.07	0.045	0.88
Batiaghata Soilmari (Ramdia)	5 – 7 feet	0.36	0.17	0.03	0.68
	10 – 12 feet	0.36	0.17	0.03	0.68
	15 – 17 feet	0.34	0.15	0.006	0.85
	20 – 22 feet	0.33	0.12	0.01	0.88
	25 – 27 feet	0.33	0.11	0.017	0.87
	30 – 32 feet	0.31	0.05	0.08	0.84

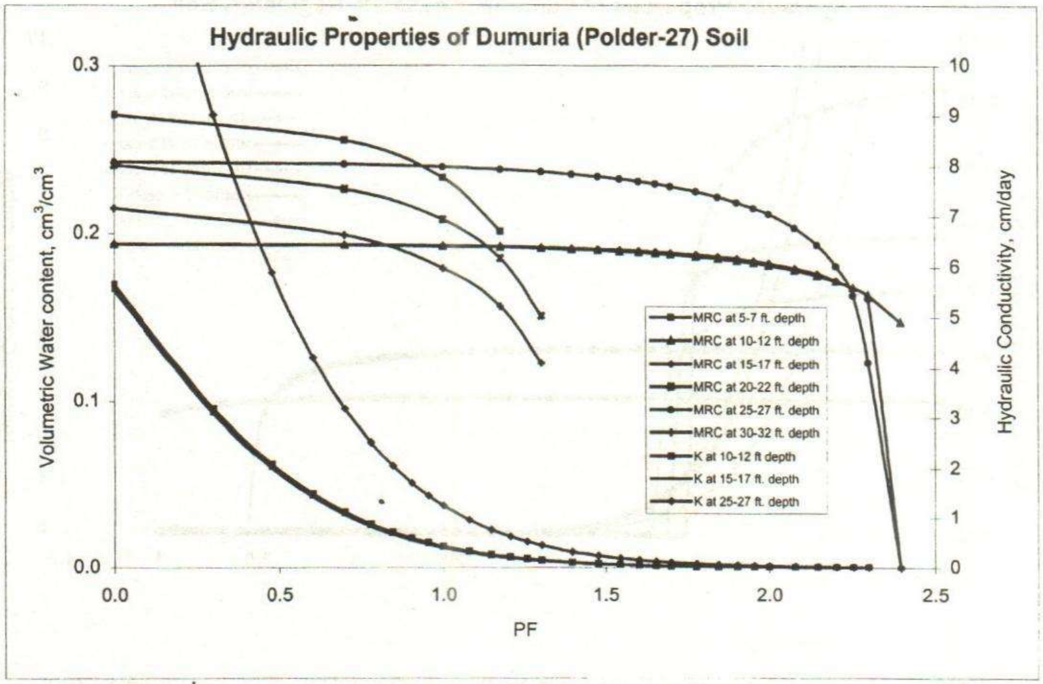

Fig.1 : MRC and K vs. PF curves of Dumuria (Polder-27) soils by parameters (from pedotransfer-functions) fitting.

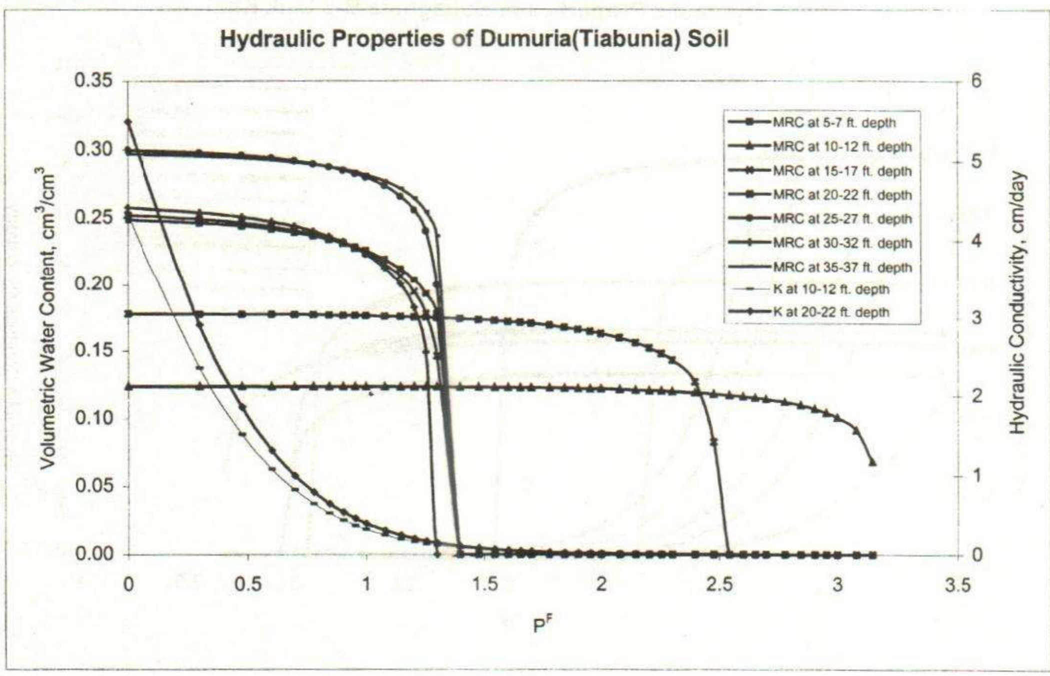

Fig.2 : MRC and K vs. PF curves of Dumuria (Tiabunia) soils by parameters (from pedotransfer-functions) fitting.

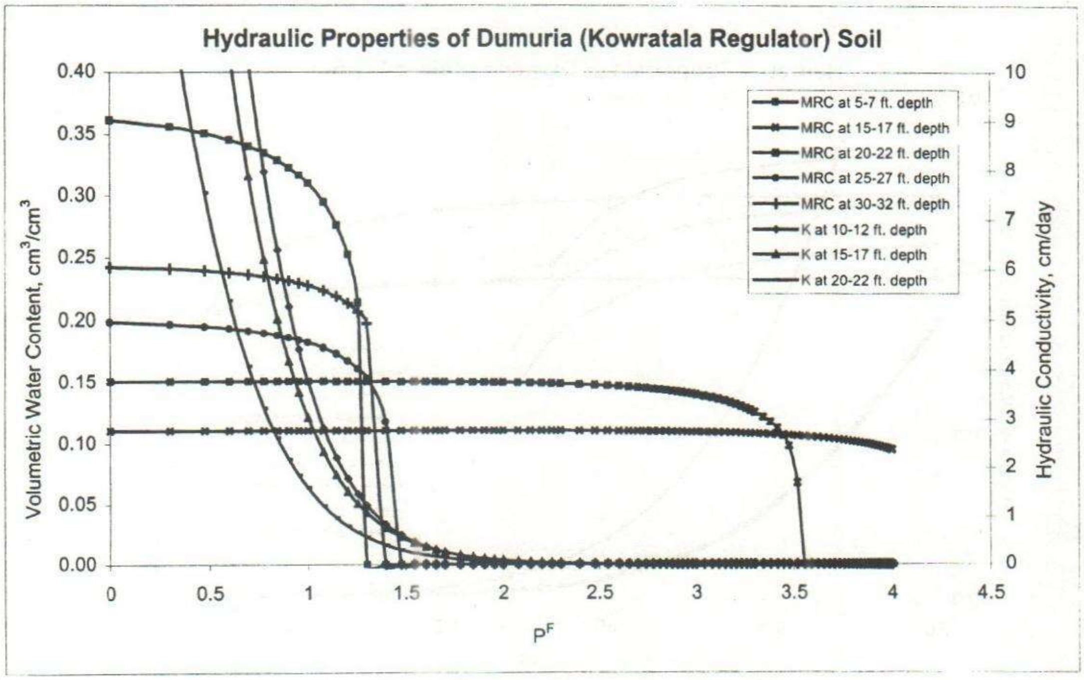

Fig.3 : MRC and K vs. PF curves of Dumuria (Kowràtala Regulator) soils by parameters (from pedotransferfunctions) fitting.

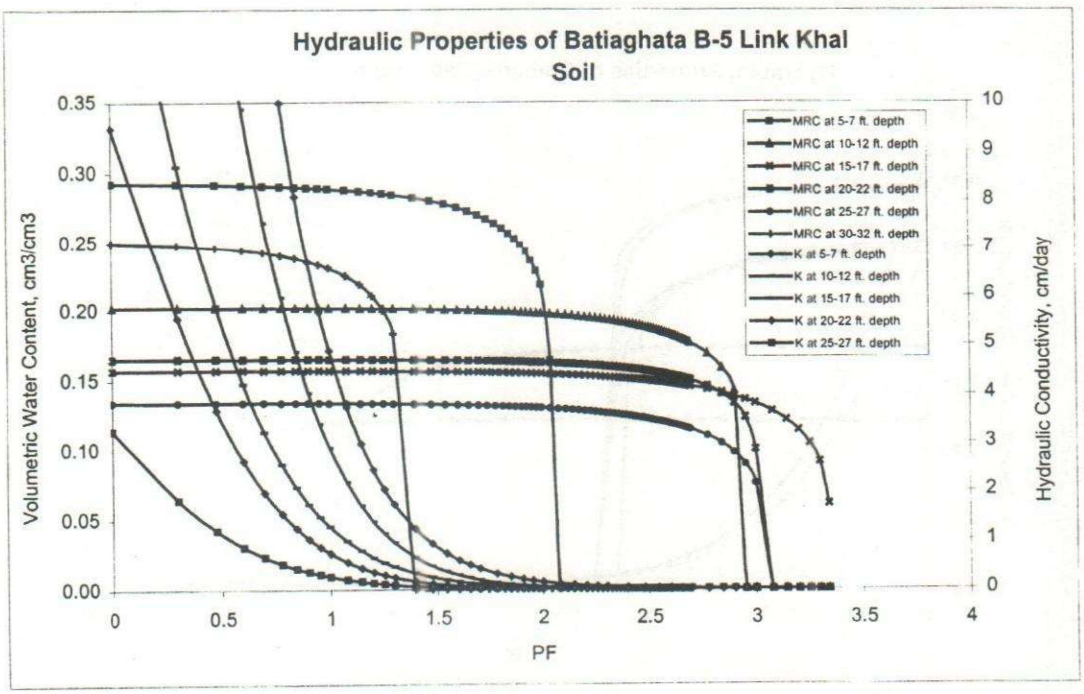

Fig.4 : MRC and K vs. PF curves of Batiaghata (B-5 Link Khal) soils by parameters (from pedotransferfunctions) fitting.

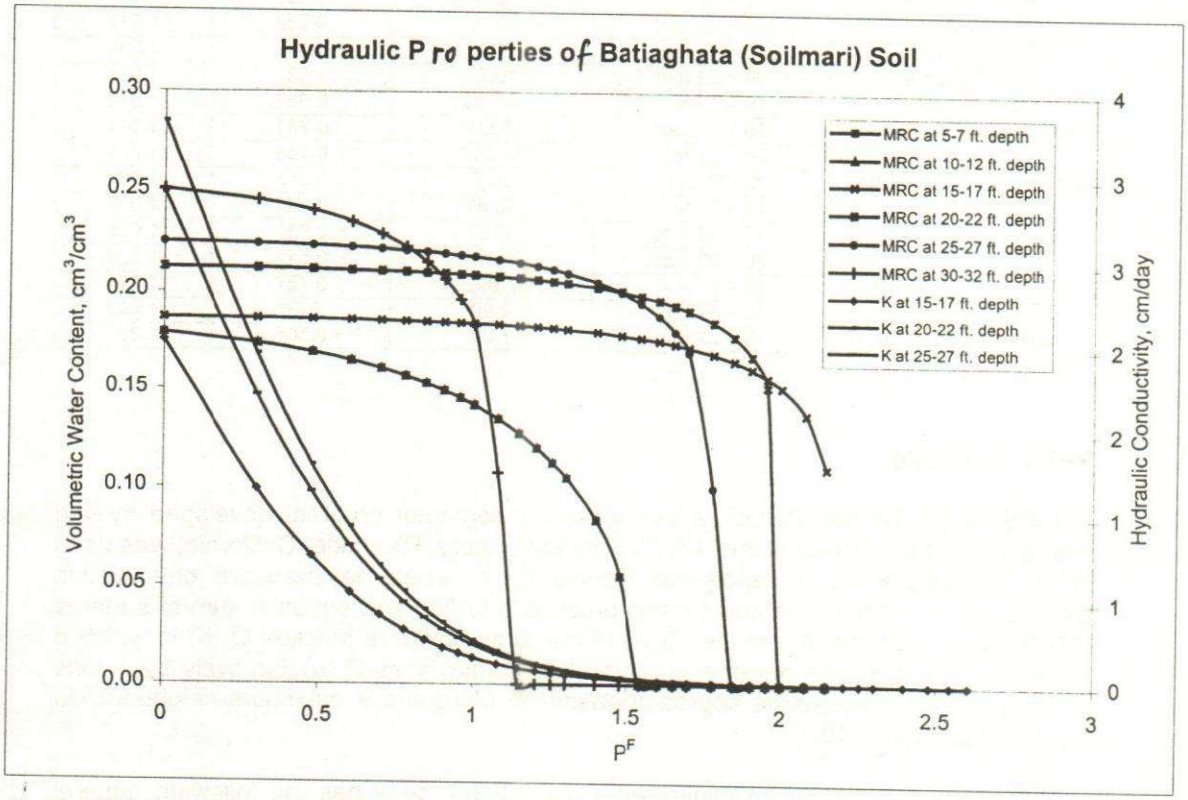

Fig.5 : MRC and K vs. PF curves of Batiaghata (Soilmari)) soils by parameters (from pedotransfer-functions) fitting.

Table 2: Estimated parameters of Gardner model

Name of the site	Depth of soil profile	K_{sat}	b	n
Dumuria Soil Site: Polder-27 Baradanga	10 – 12 feet	13.56	0.686	1.678
	15 – 17 feet	13.51	0.685	1.487
	25 – 27 feet	39.97	0.756	1.617
Dumuria Soil Site: Tiabunia	10 – 12 feet	11.77	0.88	1.585
	20 – 22 feet	16.77	1.087	1.518
Dumuria Soil Site:Kowratala Regulator	10 – 12 feet	117.12	0.566	1.814
	15 – 17 feet	100.54	0.741	1.644
	20 – 22 feet	51.217	0.756	1.617
Batiaghata B-5 Link Khal Soil	5 – 7 feet	186.71	0.862	1.609
	10 – 12 feet	56.31	0.374	1.906
	15 – 17 feet	33.21	0.639	1.628
	20 – 22 feet	20.187	0.515	1.821
	25 – 27 feet	7.992	0.757	1.594
Batiaghata Soilmari (Ramdia)	15 – 17 feet	4.89	0.70	1.647
	20 – 22 feet	7.00	0.505	1.822

RETC-GMI code

The RETC (RETention Curve), a user's friendly computer program developed by van Genuchten, et al., (1992) at the U.S. Salinity Laboratory, Riverside, California, was used to fit the parameters of analytical models by nonlinear least-square optimization techniques. The aim of the curve fitting process is to find and equation sum of squares (SSQ) associated with the model. SSQ refers to as objective function O (b) in which b represents the unknown parameter vector. RETC minimizes O (b) iteratively by means of a weighted least-squares approach based on Marquardt's maximum neighborhood method (Marquardt, 1963).

The objective function to be minimized in RETC-GMI code has the following general form when unknown parameter vector b is fitted simultaneously to observed retention and hydraulic conductivity data.

$$O(b) = \sum_{i=1}^{N}\left\{w_i\left[\theta o_i - \theta_{fi}(b)\right]\right\}^2 + \sum_{i=1+N}^{M}\left\{w_i W_1 W_2\left[Ko_i - Kf_i(b)\right]\right\}^2 \tag{6}$$

Where θo_i and θf_i are the observed and fitted water contents data, N is the number of retention data points, Ko_i and Kf_i are the observed and fitted conductivity data and M is the total number of observed retention and conductivity data points. W_1 and W_2 are the weighting factors as defined below.

$$W_2 = \frac{(M-N)\sum_{i=1}^{N} w_i \theta_i}{N \sum_{i=N+1}^{M} w_i \left|K_i\right|} \tag{7}$$

The input data were the moisture retention data obtained from the parameters of MRC model of van Genuchten (Equation.2), which were calculated from

Pedotransferfunctions. The parameters were then optimized) in the RETC-GMI code. The shape parameter m was restricted as m = 1-1/n

Results and discussion

The parameters for equation (2) and (4) have been tabulated in Table.1 and Table.2 respectively. In the graphs 1a. to 5a sigmoidal curves are describing MRC of soils at different depths of Dumuria and Batiaghata thanas of Khulna district under KJDRP Graphs 1b. to 5b. are showing relationships of hydraulic conductivity with P^F for the said soils at different depths.

Although not all the parameters obtained by Pedotransferfunctions could be optimized in the RETC-GMI code but still the optimization gave excellent fit to the retention data of some soils. The optimized values of the parameters are shown in Table.3.

Some of the soils gave excellent fitting to parameters of Equation 2 to smooth sigmoidal MRC curves, in particular MRC curves of Dumuria Polder-27 soil at depth 25-27 feet (Fig.6), Dumuria-Tiabunia soil at depth 20-22 feet depth (Fig. 7), Batiaghata B-5 Link Khal soil at depth 10-12 feet depth (Fig. 8) and Batiaghata-Soilmari soil at 15-17 feet depth (Fig. 9).

The optimized values of the parameters of Equation (2) of the soils that were successfully run in RETC-GMI code, their 95% confidence limit and standard error of data fitting are furnished in Table. 3. For optimization run θ_r was set to zero by the program itself.

The ranges of the R-squared (R^2) values were for this fitting is 0.996 to 0.9986 and the sum of the squared (SSQ) values were 0.001 to 0.01 that indicate more or less good fitting.

Table 3: Optimized values parameters of van Genuchten model of soil successfully run in RETC –GMI code.

Site	Depth		θ_s	α	n
Dumuria Polder-27 Baradanga	10′-12′	Value	0.197	0.058	1.08
		95% confidence limit	0.176- 0.217	0.07-0.08	1.02-1.14
		Standard error	0.0099	0.01	0.029
	15′-17′	Value	0.2	0.062	1.078
		95% confidence limit	0.187-0.212	0.05-0.08	1.03-1.12
		Standard error	0.0.00617	0.00704	0.02
	25′-27′	Value	0.248	0.019	1.26
		95% confidence limit	0.232-0.265	0.015-0.024	1.16-1.37
		Standard error	0.007	0.002	0.049
Dumuria Tiabunia	10′-12′	Value	0.125	0.021	1.045
		95% confidence limit	0.109-0.142	0.011-0.031	0.98-1.11
		Standard error	0.008	0.005	0.029
	20′-22′	Value	0.185	0.0255	1.14
		95% confidence limit	0.176-0.194	0.019-0.032	1.08-1.19
		Standard error	0.0.0046	0.0033	0.027
		95% confidence limit	0.11-0.135	0.014-0.031	1.03-1.08

Site	Depth		θ_s	α	n
Dumuria Kowratala Regulator	15′-17′	Value	0.1223	0.0225	1.053
		95% confidence limit	0.11-0.135	0.014-0.031	1.03-1.08
		Standard error	0.0061	0.0044	0.0113
Batiaghata B-5 Link Canal	5′-7′	Value	0.285	0.014	1.247
		95% confidence limit	0.267-0.303	0.008-0.02	1.0-1.492
		Standard error	0.0086	0.003	0.1185
	10′-12′	Value	02011	0.0183	1.097
		95% confidence limit	0.188-0.215	0.014-0.023	1.05-1.14
		Standard error	0.006	0.001	0.019
	20′-22′	Value	0.1726	0.0214	1.07
		95% confidence limit	0.159-0.186	0.006-0.037	1.03-1.11
		Standard error	0.006	0.007	0.019
Batiaghata Soilmari (Ramdia)	15′-17′	Value	0.194	0.063	1.15
		95% confidence limit	0.178-0.209	0.048-0.078	1.09-1.21
		Standard error	0.0074	0.007	0.028

The van Genuchten model parameters obtained from Pedotransferfunctions worked well for some soils for which it was made possible to fit the retention data to typical, smooth s-curves.

These curves seem to describe MRC of these soils leading to a reliable estimate of K. The established relationships can be used to predict hydraulic phenomena of soil in different layers consisting of soils with similar pedological characteristics. Thus this can be extended to large area for characterizing drainage criteria as an easier and cheaper kind of estimation technique of soil hydraulic properties.

Acknowledgement

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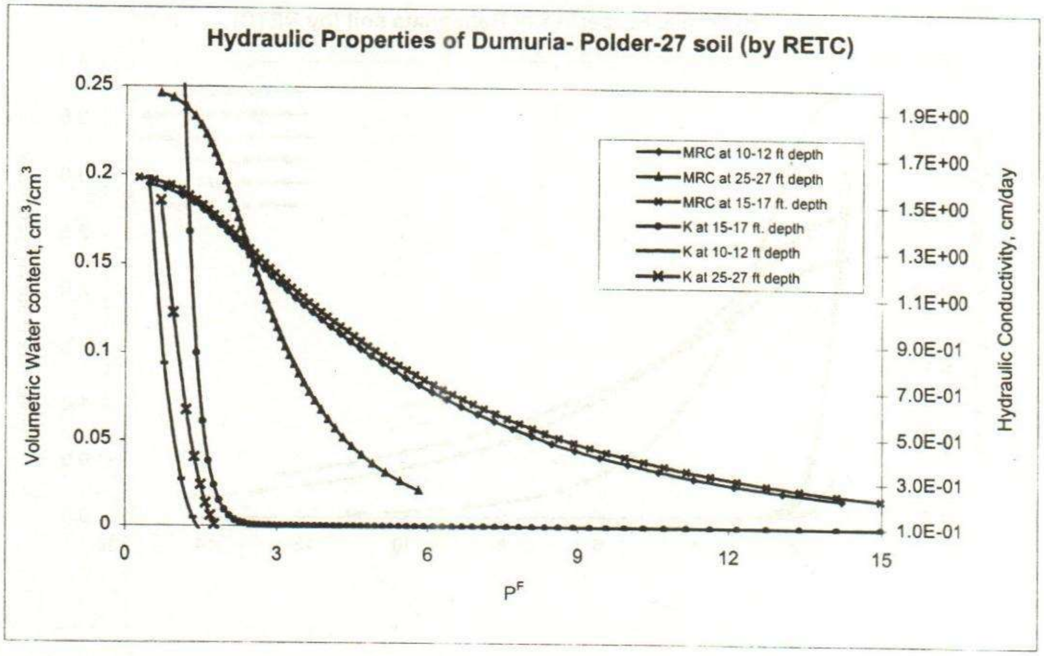

Fig.6: MRC curves and K vs. P^F plotting of Dumuria soil after parameter optimization by Computer software RETC

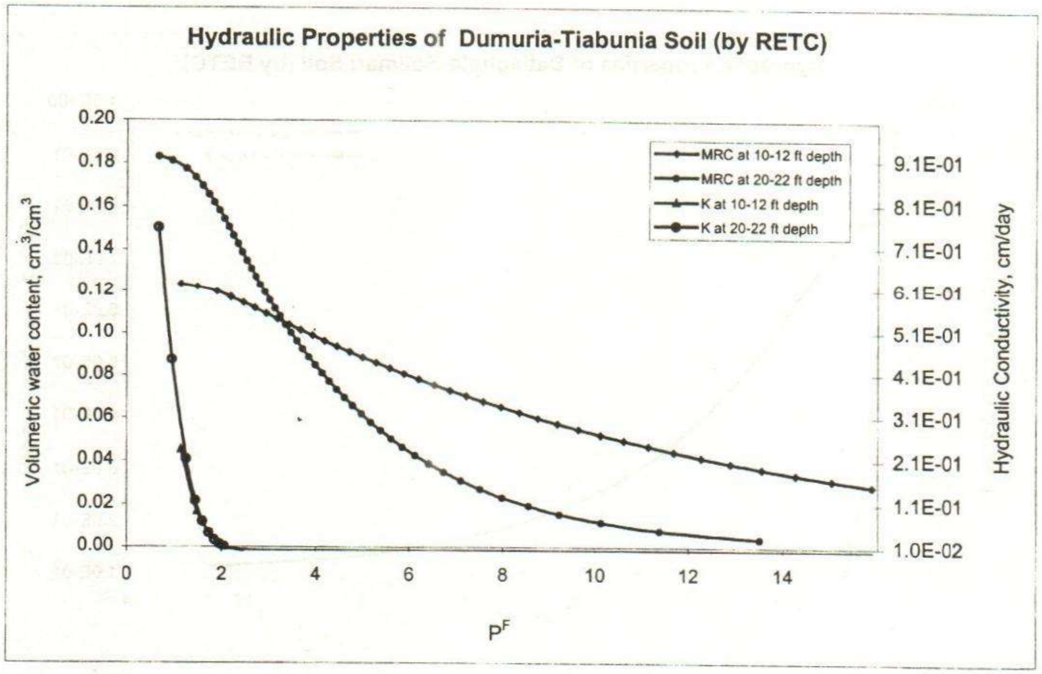

Fig.7: MRC curves and K vs. P^F plotting of Dumuria (Tiabunia) soil after parameter optimization by Computer software RETC

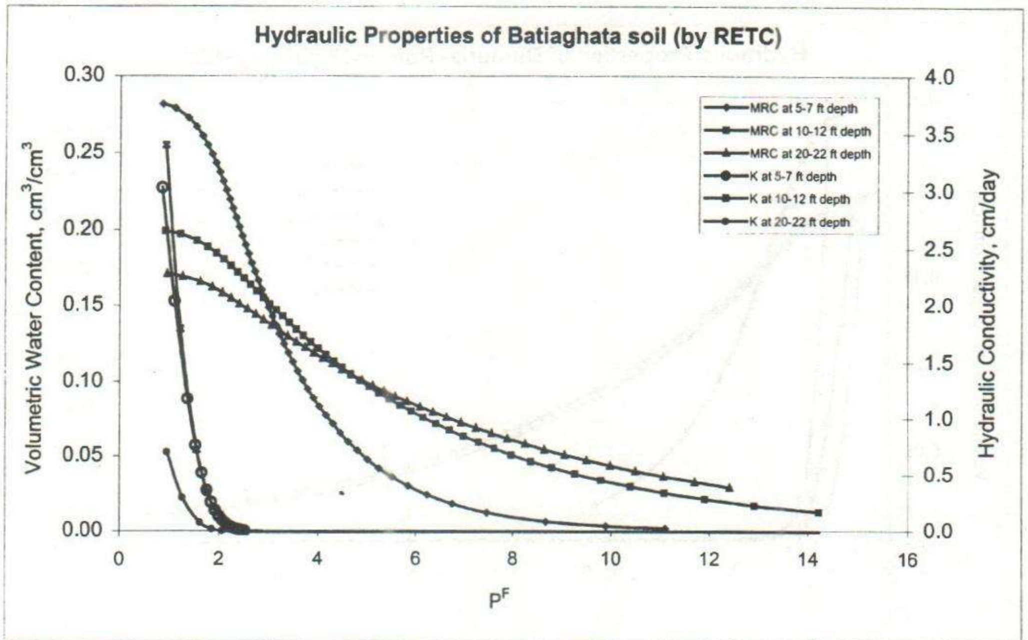

Fig.8: MRC curves and K vs. P^F plotting of Batiaghata soil after parameter optimization by Computer software RETC

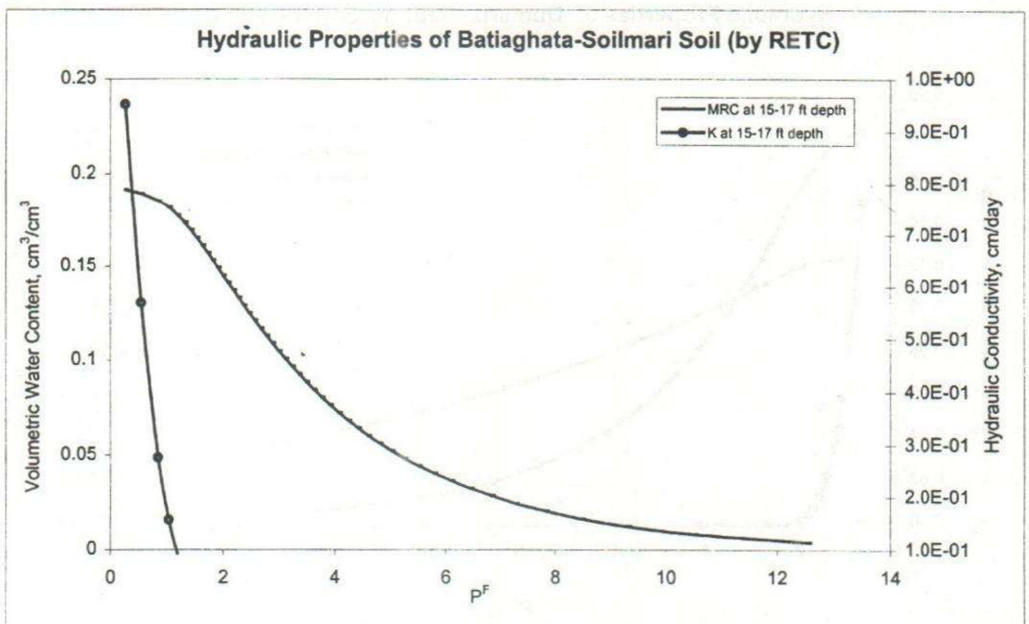

Fig.9: MRC curves and K vs. P^F plotting of Batiaghata-Soilmari soil after parameter optimization by Computer software RETC

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SEDIMENT INFLOW CHARACTERISTICS WITH THE IMAPCT Of 1998 FLOOD THROUGH THE RIVER BRAHMAPUTRA-JAMUNA

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Abstract

This paper provides an overview of suspended sediment inflow characteristics through the River Brahmaputra-Jamuna at the gauging station Bahadurabad. On the basis of weekly hydrological and sedimentological data measured by BWDB and RRI, total annual suspended sediment discharge have been calculated as to focus its variation through the years. In addition to that the impact of the catastrophic 1998 flood on the suspended sediment inflow through the river Brahmaputra-Jamuna at the gauging station Bahadurabad have been studied. This study describes the variation of fine and coarse suspended sediment particles inflow with water level and discharge at Bahadurabad. From this study it reveals that among all the study year the volume of total suspended sediment inflow is the highest in the catastropic flood year 1998.

Introduction

Bangladesh is a flat deltaic country formed by the sediment carried by three international river systems, namely, the Ganges-Padma, the Brahmaputra-Jamuna, and the Meghna. Jamuna is a part of the Brahmaputra, one of the largest of the world, flowing from the Himalayes in Tibet through Assam. The river system are being silted up each year, there by causing problems for irrigation, navigation, communication, flood inundation and maintenance of port & harbour.

Flood has become almost a regular phenomenon in Bangladesh causing serious damage to standing crops, properties and taking a huge toll of cattle heads and other livestocks. Bangladesh Inland Water Transport Authority (BIWTA) is spending a large sum of money for dredging riverbeds to maintain their navigability and Chittagong & Chalna Authorities to maintain accessibility to their ports and harbors.
Bangladesh Water Development Board (BWDB) is also facing many problems in keeping the intake canal of GK project from being silted up.

Bangladesh have been recently experienced with one of the catastrophic flood of this century in 1998. Although every flood that normally occur in this country brings huge amount of sediment. For the cause of geographical and environmental condition, in the rainy season the above mentioned river systems bring high discharge of water from the upstream. For that consequence, high discharge causes a significant river bank erosion and increases the amount of sediment into water. Mainly this sediment decreases the capacity of the channel by raising the bed level of the rivers. As a result, flood has become a regular phenomenon in the downstream region. The yearly sediment

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transport volume and inflow pattern is varying through the years. In this way, sediment transport is being a complex system in the alluvial channels specially in the downstream region.

This is why importance of sediment measurement and sediment study is being an essential issue for any kinds of river training program. Extensive and proper studies of sediment by sediment measurement throughout the whole year can help in a long-term solution of the sediment problem in the downstream region like Bangladesh.

The Brahmaputra-Jamuna and the Ganges-Padma river system create major channel network in Bangladesh. Sediment aspects of the Bangladeshi River are characterized by a fine sedimentary environment. Due to low threshhold velocity, the rivers are highly mobile with continuous reworking and deformation of their beds and banks transporting huge quantities of sediments. Among the three major rivers, the Brahmaputra river has the highest sediment calibre and transport the largest sediment load (RSP, 1996). Under these circumstances and because of very recent data availability at Bahadurabad, sediment inflow studies of Brahmaputra-Jamuna River are taken with the following objectives:

- To observe an overview of the suspended inflow characteristics through the river Brahmaputra-Jamuna at the gauging station Bahadurabad.

- To compare the hydrological parameters, suspended sediment discharge and sediment size of the 1998 year with those of the normal year.

Literature review

Channel network

In Bangladesh the channel network of the major rivers are formed by the Ganges, the Brahmaputra (Jamuna River in Bangladesh) and the Meghna rivers and several of their tributaries and distributaries: The remarkable aspect of the Bangladeshi rivers is that they form an intricate network influencing each other hydraulically and morphologically and debauching together into the Bay of Bengal. Based on an analysis of cross-sections measured by the BWDB, Klaasen & Vermeer(1988) stated that the Jamuna River is neither degrading or aggrading. This river, despite having a smaller drainage basin than the Ganges has a steeper slope, a larger discharge and sediment transport and higher sediment content.

According to RSP (1996) the Brahmaputra River drains the northern and eastern slopes of the Himalayes is 2,900 km long and has a command area of 573,500 km^2. It is a wandering braided river with an average bankfull width of some 11 km. From the literature review, it is found that the average flow, mean annual suspended sediment load, bed material grain size (d_{50}) of this river is 19600 m^3/s, 499 million tons and 0.22 mm respectively. The Ganges River has a finer sedimentary environment than the Jamuna. The median bed-material sediment diameter of this river is about 0.12 mm. Padma River bed-material sediment diameter varies from 0.09 mm in the lower reaches to 0.14 mm in the upper reaches. Mean annual suspended sediment transport through the Jamuna, the Ganges and the Padma Rivers according to various investigators are given in Table.1. The Gauging stations are located in the Jamuna River at Bahadurabad, the Ganges River at Harding Bridge and the Padma River at Baruria and Mawa as shown in Fig.1 (RSP, 1996).

Table 1: Mean annual suspended sediment transport in million tons through the Jamuna, the Ganges and the Padma Rivers by various investigators (RSP, 1996).

River	Coleman	CBJET	MPO	FEC	Hossain	RSP	BWDB
Jamuna	608	499	387	431	650	586	500
Ganges	479	196	212	338	480	549	450
Padma	-	581	563	-	-	894	694

Distributaries & tributaries

Major tributaries and distributaries of the Brahmaputra-Jamuna River include two right bank tributaries and two left-bank distributaries. The major right bank tributaries are the Teesta River and the Atrai-Gur River. Teesta is a coarse sand-bed braided river. There are two major left-bank distributaries of the Brahmaputra-Jamuna River; they are the abandoned course of the Brahmaputra River (Known as the old Brahmaputra) and the Dhaleswari River. In the true sense of the term, the old Brahmaputra River and the Dhaleswari River are the loop channels carrying a small part of the Jamuna river flow to the upper Meghna River. Both these rivers debauch into the upper Meghna River, after through some smaller distributaries (RSP, 1996).

Sources of sediment

According to the source, sediments can be divided into two groups viz. bed material load and wash load. The main sources of sediment are the materials of stream bed and the fine particles which come from the catchment of the river. But according to mode of transportation, the sediment can be classified into three categories, viz., bed load, saltation load and suspended load. Bed load is defined as sediment particles which move by sliding or rolling over and near the bed, generally in the propagating bed forms such as ripples and dunes. Saltation load is defined as sediment particles which make their journey by hopping and jumping as a result of drag force. These particles very frequently move as suspended load. Suspended load consists of sediment particles held in suspension by balancing their gravitational force with the upward forces due to the turbulence of the fluid (Haque, 1997).

Methodology

BWDB has a routine for the measurement of river stage, discharge and sediment samples at several gauging stations along the major rivers. The river system along with the gauging stations have been shown in **Fig. 1**. This study comprises the sediment characteristics as well as other hydrological parameters of the river Brahmaputra-Jamuna at bahadurabad to focus the sediment inflow characteristics from the upstream rivers. This study has been mainly concentrated on suspended sediment transport as bed load materials data are not available. At the gauging station fines and coarse sediment particles are separated after collection by a separator. Then the fine sediment portions known as bulk suspended sediment and coarse sediment portion (sand) and other hydrological data are sent to River Research Institute (RRI), Faridpur for laboratory analysis.

In advanced countries many improved methodology and sophisticated equipment are used for laboratory analysis of sediments. There are various methods for the determination of concentration such as Evaporation method, Filtration method and Displacement method. Similarly, there are several methods such as Visual Accumulation tube (VA tube) method, Bottom withdrawal tube (BW tube) method, Pipette method, Settling tube method and Hydrometer method for particle size analysis of sediment (Ref. Guy PR & ASCE Re. 54).

In our study evaporation method and Sieve-Pipette method have been used for concentration and size distributions respectively. In case of sediment size classification, MIT Standard classification method has been followed.

Laboratory analysis and presentation of results

To study the sediment inflow characteristics of the river Brahmaputra-Jamuna a large number of suspended sediment samples collected by BWDB during the year 1984, 1985 and 1995-1998 have been analyzed in the Sediment Technology Laboratory of RRI. The samples were tested for the determination of concentration and grain size distribution following the procedure as mentioned in the methodology (RRI testing reports Sed-7(96), 4(97), 16(98), 8(99), 7(96), 4(97), 16(98), 8(99), Annual report 1984 & 1985). The stage and discharge data of the concerned river were processed. Monthly representative values are determined by averaging the weekly basis data and test results. The stage vs. time, discharge vs. time, stage vs. discharge, sediment transport rate vs. time and discharge vs. sediment transports rate have been presented graphically in **Figs. 2** through **6**. The comparison of particle size of suspended sediment of 1997 and 1998 is shown in **Fig. 7**. The flow characteristics, the sediment inflow characteristics and particle size characteristics of the Brahmaputra-Jamuna river have been presented in **Table 2, 3** and **4** respectively.

Discussion

Hydrological aspects

The graphical representation between the average monthly water level and time in the year 1984, 1985 and 1995 to 1998 is given in **Fig.2**, shows that almost every year highest water level has been found in the month of July. In the year 1998 water level was in the highest level from the month of June to September among all the study year. **Fig. 3** shows the relation between the monthly average water discharge and time in the year 1984, 1985 and 1995 to 1998. In the year 1998 water discharge was the highest from the June to October. The flow characteristics of the Brahmaputra-Jamuna river is given in **Table 2** and it is observed that the highest water discharge has been found in the months of July to September.

Table 2: Present flow characteristics of the Brahmaputra-Jamuna River at Bahadurabad

Year	Month of Min. Flow	Min. Monthly Flow (Cumec)	Month of Max. Flow	Max. Monthly Flow (Cumec)	Ratio to Max to Min Flow	Average Flow (Cumec)
1998	Feb.	3689	July	**79079**	21.44	**32740.1**
1997	Mar.	3986.5	July	44230	11.09	19730.6
1996	Feb.	3698.5	July	61153.7	16.53	21609.4
1995	Feb.	4321	July	67343.5	15.59	23348.3
1985	Feb.	3960	July	60500	15.28	20456.3
1984	Feb.	3950	Sep.	53250	13.48	21715.3

Based on the available data of all the study year the relation and graphical representation of river stage and water discharge with an equation has been given in **Fig 4**. It is obvious that there is a strong relationship between water level and discharge where coefficient of correlation is 0.99.

Sedimentological aspects

Variation of suspended sediment discharge with time is given in **Fig. 5,** shows that the excessive suspended sediment discharge have been occurred in the year 1998 & 1984 and after that in 1995. It is observed that the suspended sediment discharge depends on major two factors that are water discharge and sediment concentration. The summarized results of suspended sediment inflow at the gauging station Bahadurabad through the

Brahmaputra-Jamuna River presented in **Table 3**, gives an idea about the range of variation of the necessary parameters. **Table.3** also shows that the lowest sediment has been carried in the year 1985.

Table 3: Results of suspended sediment inflow through the Brahmaputra-Jamuna River at Bahadurabad

Year	Month of Max. Sediment Inflow (Amount in mtons)	Bulk Suspended Sediment Inflow (Million tons)	Coarse Suspended Sediment Inflow (Million tons)	Total Suspended Sediment Inflow (Million tons)
1998	August (315.01)	**853**	54.8	907.8
1997	August (115.89)	391.8	25.6	417.4
1996	May (91.62)	363.5	40.4	403.9
1995	May (256.86)	510.6	38.0	548.6
1985	Sep (77.92)	186.9	76.1	263.0
1984	Sep (460.8)	743.6	67.7	811.3

Based on the data available in the year 1984, 1985 and 1995 to 1998 the total annual suspended sediment discharge varies from 263 to 907.8 million tons. Among all the study year highest total annual suspended sediment discharge has been occurred in the catastrophic flood year 1998 which is far deviated from the year 1995 to 1997 but not from the year 1984.

The sediment rating curves by fitting a regression line between sediment transport (Q_s) in tones/day and discharge data (Q) in m^3/sec with the equation for the river Brahmaputra-Jamuna at the gauging station Bahadurabad is represented in **Fig.6** where coefficient of correlation is 0.88. This sediment rating curves can be used to determine the yearly gross amount of sediment being transported by a river.

The results of the monthly averaged percentage of sand & silt and D_{50} of suspended sediment samples for the year 1998 and 1997 of the Brahmaputra-Jamuna river have been presented in **Table 4** and the graphical representation of monthly averaged D_{50} of the 1997 & 1998 are shown in **Fig. 7**. From the **Table 4** and **Fig. 7** it is observed that the suspended sediment size in the year 1998 is relatively less than that of the year in 1997. This indicates that during the flood year fine sediment particles are transported more than the normal year in the River Brahmaputra-Jamuna. It has been found from **Table.3** that there is a wide variation in bulk-suspended sediment (Silt fraction) discharge. But coarse suspended sediment (sand fraction, also known as local bed material) discharge varies to a limited extent ranging from 25.6 to 76.1 million tons. On the other hand bulk suspended sediment discharge varies from 186.9 to 853.0 million tons. In that sense, sediment inflow environment have been mainly dominated by bulk suspended sediment or the fine sediment particles.

Table 4: Results of analysis of suspended sediment samples of the Brahmaputra-Jamuna River for the year- 1997& 1998

Months	Sediment Size Classification (Percent by Weight)				D_{50} in 1997 (mm)	D_{50} In 1998 (mm)
	in 1997		in 1998			
	Sand	Silt	Sand	Silt		
Jan.	N.A	N.A	47.0	53.0	N.A	0.058
Feb.	N.A	N.A	52.0	48.0	N.A	0.065
Mar.	62.0	38.0	39.0	61.0	0.088	0.050
Apr.	70.0	30.0	43.0	57.0	0.078	0.055
May.	64.0	36.0	43.0	57.0	0.148	0.057
Jun.	56.0	44.0	44.0	56.0	0.073	0.054
Jul.	83.0	17.0	46.0	54.0	0.115	0.058
Aug.	62.0	38.0	64.0	36.0	0.165	0.130
Sep.	81.0	19.0	50.0	50.0	0.195	0.058
Oct.	73.0	27.0	44.0	56.0	0.125	0.055
Nov.	76	24.0	N.A	N.A	0.135	N.A
Dec.	76	24.0	N.A	N.A	0.120	N.A

Conclusion

The yearly total amounts of sediments being transported by a river is varying during the year and through the years. With high floods much more sediment is transported than the years with minor floods. It has been found that the total suspended sediment inflow was the highest in the year 1998. It also seems that total annual suspended sediment inflow is in the increasing trend from the year 1996 to 1998. But the total value of total suspended sediment discharge in the year 1984 is also higher, it may occur due to sudden bank erosion and/or local scour or sampling mistake. Among coarse and fine sediment particle, the transport of fine particles is more important in this river because it occupies almost 80 percent of total suspended sediment discharge. This study reveals that every year a significant amount of suspended sediment is being entered through the great alluvial river system Brahmaputra-Jamuna. There is a significant variation in water and sediment discharge throughout the year. During the high floods the most of the sediments are transported and the fine sediments are higher in quantity due to over land flow through the catchment area and river bank erosion.

This inflowing sediment particles are creating many problems in the down stream region such as decrease the capacity of the channels, siltation at the bottom of any hydraulic structures which may cause much difficulty in operation, silt deposition in irrigation channel and some time sand deposited over the agricultural land during flood. In this way, details study of the inflowing sediment throughout the whole year and over the years are very much important for the future planning of any kind of river training and other engineering works.

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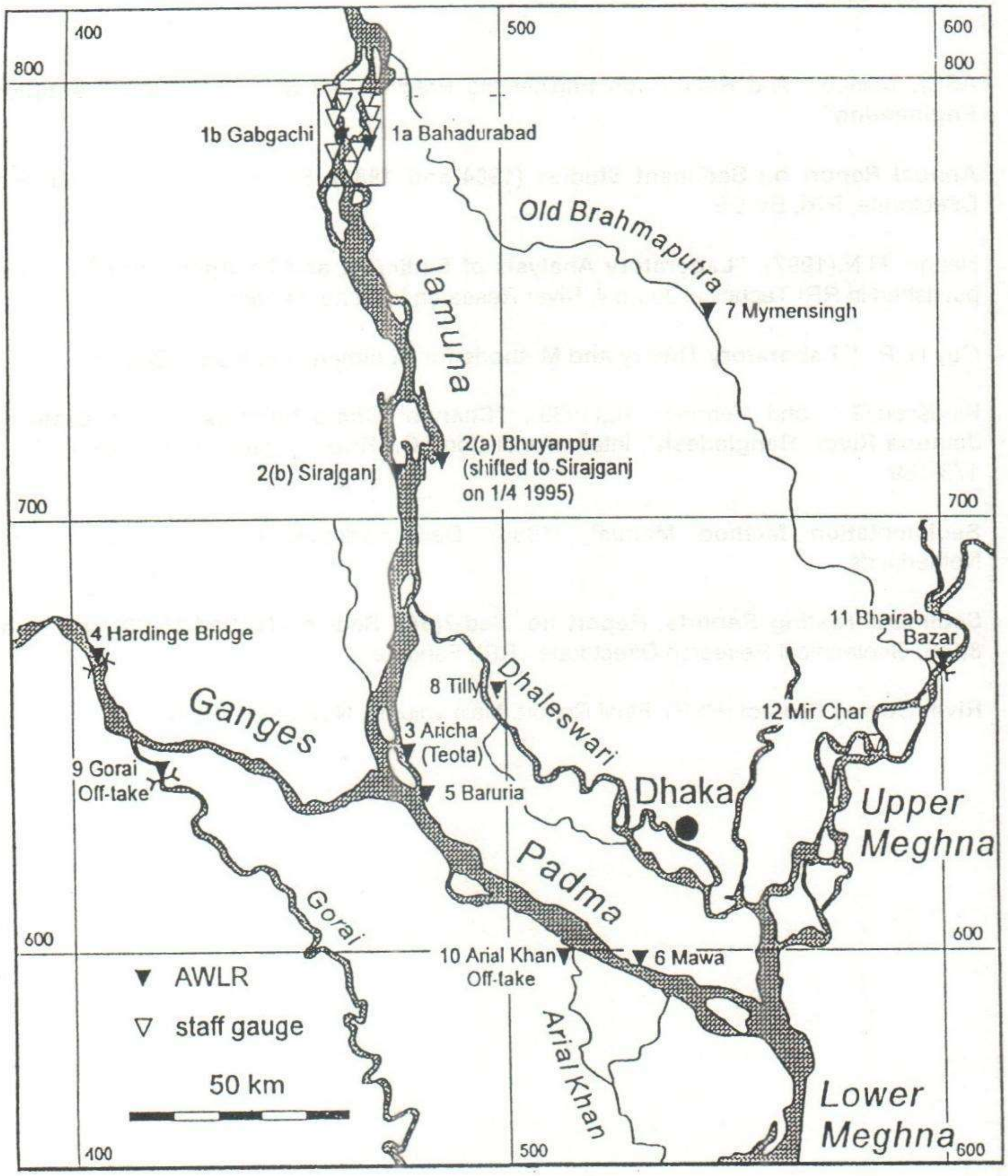

Fig. – 1: Major Rivers Showing the Location of Different Gauging Stations (Source: RSP, 1996)

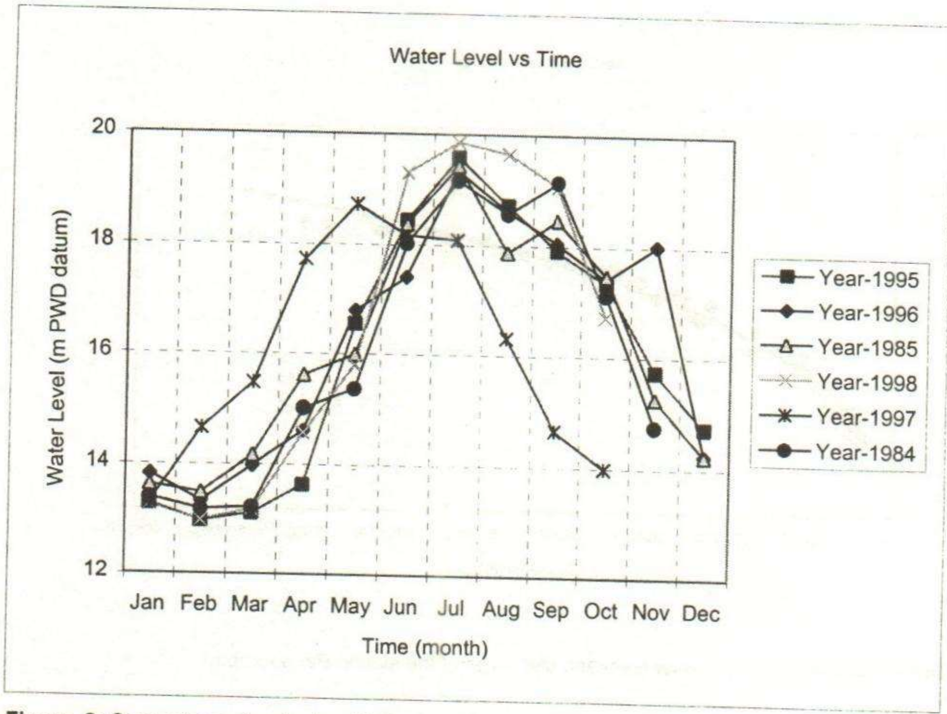

Figure 2: Comparison of water level between 1998 and the other study year at Bahadurabad

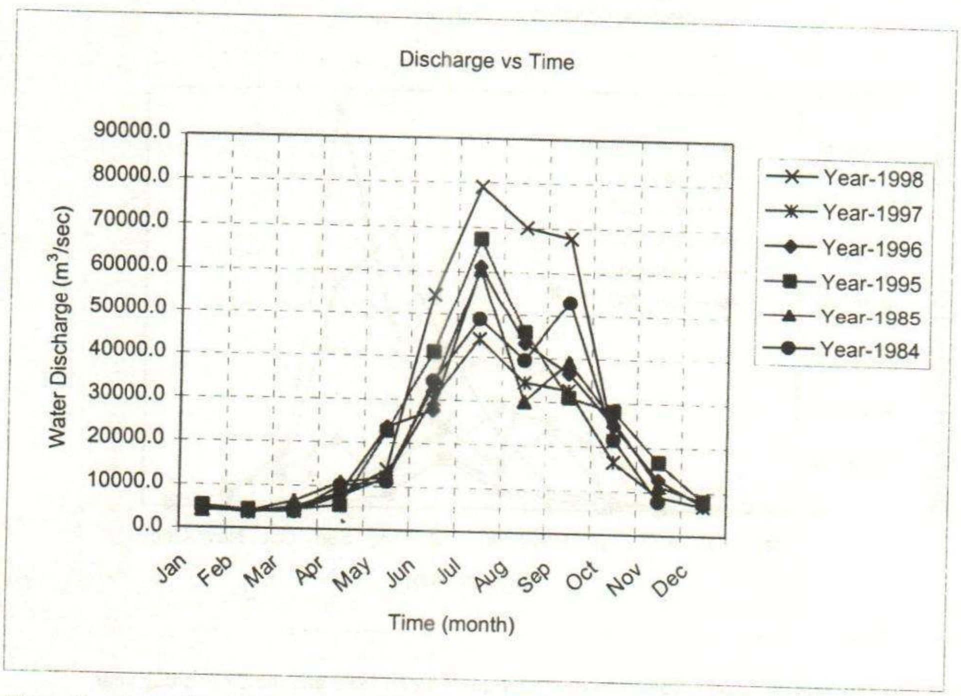

Figure 3 : Comparison of water discharge between 1998 and other study year at Bahadurabad

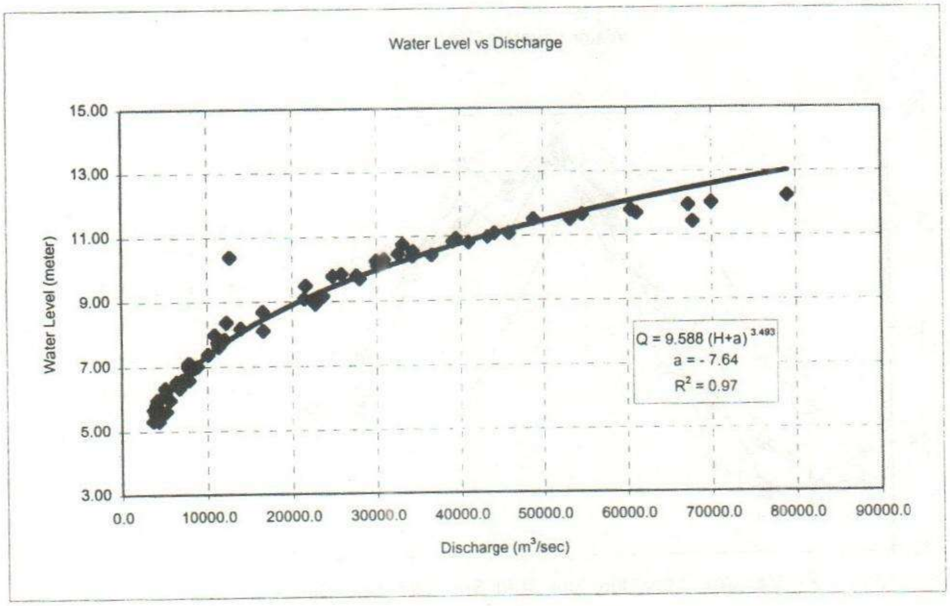

Figure 4: Relation between water level and discharge at the station Bahadurabad

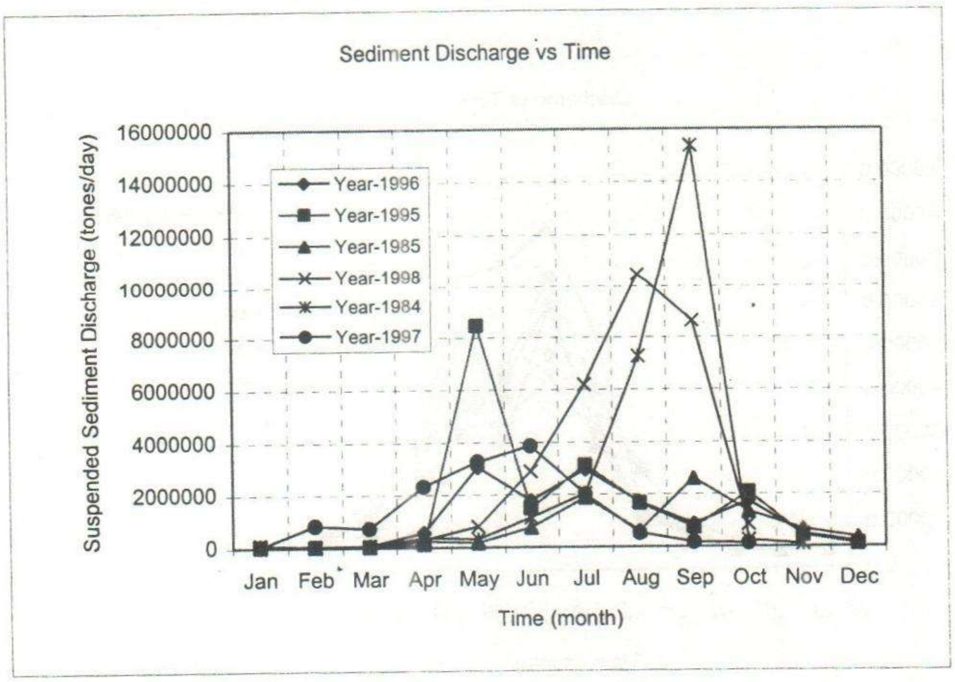

Figure 5: Comparison of suspended sediment discharge between 1998 and the other study year

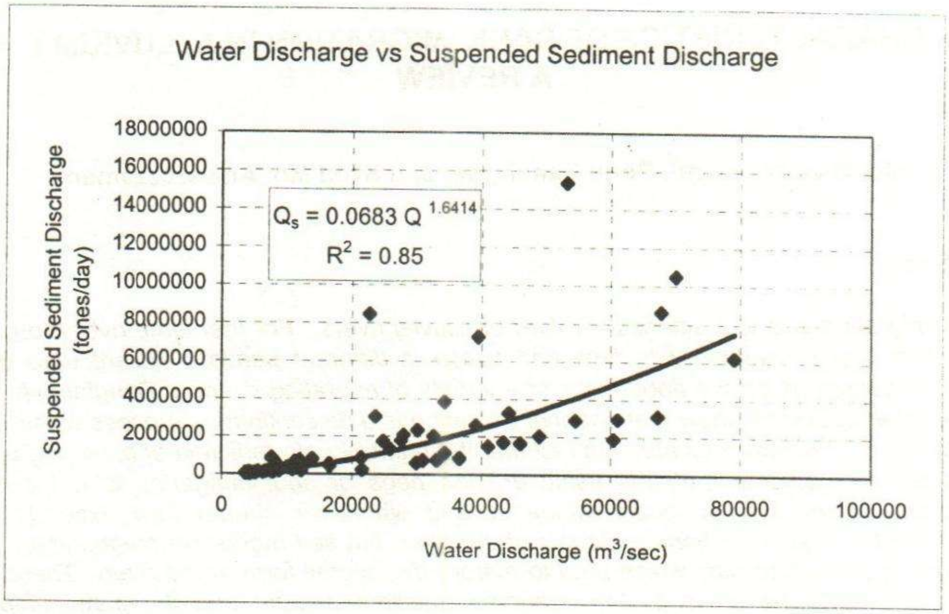

Figure 6: Relation between water discharge and suspended sediment discharge

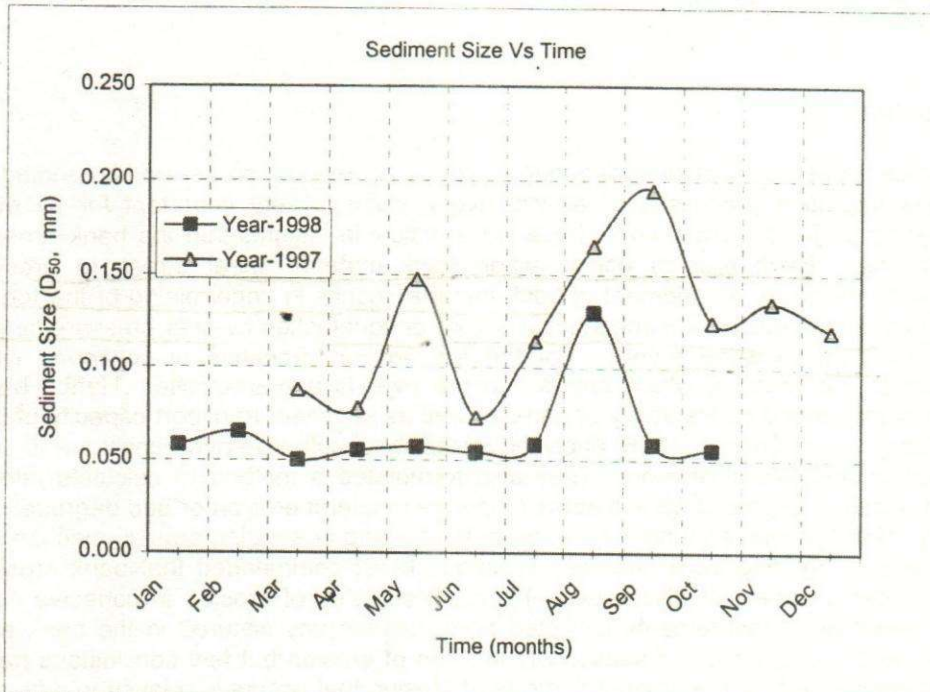

Figure 7: Comparison of suspended sediment size (D_{50}) between the year 1998 and 1997

CHARACTERISTICS OF BANK MIGRATION IN ALLUVIUM : A REVIEW

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Abstract:

Bank migration is a characteristic feature of alluvial rivers. For the same river the bank migration rate is variable with time and space in different bends. Recent data from forest successions on the floodplains of a variety of migrating rivers of Bangladesh and some other countries show that channel migration is a discontinuous process within any single bend. It clearly indicates the fact that the reliability of prediction of bank migration rates by conventional methods using erosion pegs or spot imageries is not out of question. Bank erosion occur wherever and whenever stream flow exceeds the threshold for erosion of bank materials of the river, but self-regulatory mechanisms are thought to come into play which tend to restore the overall form of the river. Therefore attention should be given to the assumed equilibrium state and its relationship to prevailing hydrologic and geologic conditions. In this paper a few past investigations regarding channel migration are reviewed and some techniques and considerations for predicting channel migration are pointed out in context of hydrology and geomorphology of Bangladesh.

Introduction:

In the recent past many researchers put efforts for developing proper understanding of the bank migration processes of alluvial rivers which is very important for all river engineering projects. Their works have led to valuable insights into the bank erosion mechanisms. Bank erosion occurs when bank material shear stress to erosion surpassed and fluvial entrainment of bank material results in undermining of the toe of the bank and subsequent soil-mechanical failure or liquefaction by over pressure in fine sand during falling water levels. For the toe erosion presence of continued river transport of the material which enters into the river is a prerequisite. Hence bank erosion is influenced by instability of bank as well as sediment transport capacity of the river. Osman and Thorne (1988) analyzed instability of cohesive riverbanks due to bed degradation and lateral erosion. They also formulated a method to calculate lateral erosion distance to predict bank stability response to lateral erosion or bed degradation. Hagerty (1991) presented fundamental aspects of piping or sapping erosion mechanism as a process affecting bank stability. Wolman (1959) commended that bank erosion occurs under a variety of discharges. He reported rates of erosion in cohesive river banks based on measurements collected from erosion pins secured in the bank and recognized the importance of seasonality in rates of erosion but few conclusions have been made concerning the amount and rate of erosion that occurs in relation to different discharge magnitudes. 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). Klaassen and Maselink (1992) used the approach of Hickin and Nanson for the Brahmaputra River in Bangladesh and found a negative correlation between the relative bend curvature (R/w) and the erosion

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rate . Brice (1974) found that bank erosion rate increased linearly with drainage area for a number of USA rivers. Daniel (1971) monitored the effect of erosional change on the form of the outer bank of meander bends along stream in Indiana.

The findings of the studies mentioned above made by various researchers make the engineers to recognize the fact that before implementing a river engineering project or any other development project along the river bank, the bank migration characteristics of the river concerned should be understood first.

Technical aspects for recognizing bank migration process

Bank migration particularly the migration of meander bends is a common feature of all alluvial rivers. 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 migration 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 of 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 analysed based on the knowledge of the relationship between form and movement and on identification of stable forms. The problem in analysing any aspect of river bank migration is to find a method which adequately demonstrates the properties of the course & 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. Generally two major approaches are used in the assessment and measurement of the bank migration of rivers. In first approach contemporary erosion of river bank is monitored by using erosion pegs. Such studies provide information on processes, rates, and temporal distribution of changes as well as on their spatial distribution. The second approach employs maps, aerial photographs, landsat and spot imageries, cross sections and other historical evidence to investigate spatial changes over longer periods of time, which is often beyond the scope of empirical observation. The study of bank migration can be benefited considerably from the wider availability of survey and particularly remote sensing. Information from available maps, from planimetric surveys and from the landsat and spot imageries it is possible to extract data representing riverbank migration. Some measurements of amount of change on individual bends can be made direct from the maps and spot imageries by measuring the conventional meander parameters such as wavelength, meander breadth and radius of curvatures between common points on the courses of different dates and comparing the values of the parameters obtained. But due to discontinuous nature of river bank migration time lapsed aerial photographs or spot imageries should not be used to measure short term river migration because it could result in enormous errors in prediction. However, if an extensive reach of river are to be studied from the same photographs or spot imageries and measurements taken for a number of bends, then an average value, of course, would only be representative of the flow conditions prevailing during the time interval when photographs or spot imageries are taken and not necessarily representative of the long term flow record.

Review of some previous studies:

A number of important studies were done by various researchers to develop insight into the bank migration process as well as to predict bank erosion rates. Approach used and outcome of those studies can guide very well to understand the bank migration process and to predict migration rate of an alluvial river. Hereafter some of those studies are reviewed briefly.

D. J. Hagerty (1991)

The author in his paper "piping/sapping erosion 1: Basic considerations" reported erosion of river banks by exfiltrating seepage which is a very widespread and significant bank erosion mechanism, but is rarely recognized. Such erosion is not consistent with common theories of tractive force erosion and can occur long after periods of high stage and in locations where deposition would be anticipated. The major cause of such unanticipated erosion is outflow of seepage with attendant removal of soil particles in the exfiltration zone, and consequent instability of undercut strata located above the zone of soil loss. Figs 1 (a-c) show a site where seepage flow out of a sandy layer carried sand out of the river bank, and the overlying more cohesive upper bank layer was undermined and collapsed. Such piping in river banks is most commonly noticed in alluvial soil deposits where the natural layering associated with alluvium favours concentration of flow in more pervious strata, and more cohesive layers tend to bridge over cavities, allowing conduits to form. For internal erosion to produce cavities there must be a free face or external plane from which seepage outflow can remove soil particles. A source of water is needed for the erosion mechanism to take place and flow must be concentrated in order that the exfiltration, characterized by the exit hydraulic gradient, will be sufficient to remove soil particles. The slabs or blocks of soil displaced during those failures will accumulate on the lower portions of the bank, together with the soil removed by piping directly. In order for the erosion process to continue, the accumulated failure materials must be removed. In the absence of continued river transport of the material eroded by piping away from the bank eventually reach equilibrium slopes, become vegetated, and remain stable.

M. Osman and Colin R. Thorne (1988).

In the paper 'River bank stability Analysis. 1: Theory' the authors presented a slope stability for steep banks and it is used in conjunction with a method to calculate lateral erosion distance. The calculation procedure of the failure plane angle, failure block width and volume of failed material per unit channel length is illustrated for the critical case. The analysis of cohesive soil erosion by flowing water developed at the Waterways Experiment Station in cooperation with University of California at Davis was applied to calculate the lateral bank erosion due to fluvial entrainment of bank material. The noticable side of this analysis is that calculation of the critical shear stress for erosion is based on the electro-chemical properties of the soil, pore water and eroding fluid. The erosion rate is computed from the excess sheer stress over the critical value. The lateral erosion distance Δw and the bed degradation distance Δz can be represented as:

$\Delta w = \Delta w\ (\tau_f, \tau_c)$

and $\Delta z = \Delta z\ (Q, R_h, S, \rho_w, \rho_s, D_s, g)$

where, τ_f = flow shear stress, τ_c = critical shear stress, Q = flow discharge, R_h = hydraulic radius, S = energy slope ρ_w = water density, ρ_s = sediment particle density and D_s = bed material size.

The initial soil erosion rate (R) can be determined from the following relation

$R = 223\times10^{-4}\ \tau_c\ e^{-0.13\tau_c}$ (gm / cm^2. min)

The initial lateral bank erosion rate is given by

$$dB = \frac{R}{\gamma}\left(m\ /\ \min\right) per\ \ unit\ \ area$$

The rate of soil erosion (R) is assumed to have and approximately linear increase with stress once the critical shear stress is exceeded. The actual erosion rate (dw) is

$$dw = dB * \left(\frac{\tau_f - \tau_c}{\tau_c}\right)\left(m\ /\ \min\right)$$

τ_c of the above equation is critical shear stress of undisturbed soil as a function of sodium absorption ratio (SAR), pore fluid salt concentration (CONC) and the dielectric dispersion $\Delta\varepsilon$, as shown in Fig. -2. If the duration of flow shear (τ_f) is Δt (min) then the lateral erosion distance (Δw) during this time is

$\Delta w = dw * \Delta t$ (m)

The stability relations for specific characteristic bank geometry were developed and used to predict the critical bank height, the width of failure block, volume of the failure mass and the starting point for subsequent analyses. In the method described above the processes and mechanisms of bank erosion and retreat have been greatly simplified in order to make them amenable to analysis. For instance, soil properties are assumed to be homogeneous, failure is taken to be catastrophic rather than progressive and fluvial erosion is treated separately in time to mass failure. The most serious shortcoming is that vegetation effects are not taken into account for explicitly. It is evident from theoretical and experimental evidence that vegetation does have a significant impact on bank stability and channel width both through its effects on near-bank flow hydraulics and on bank material properties (Hey and Thorne 1986).

Hickin and Nanson (1984)

The paper of the authors 'Lateral Migration Rates of River Bands' 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 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. The rate of channel migration (M) can be expressed by the qualitative statement

$M = f(\omega, \gamma_b, h, r, w)$

Where, ω = stream power per unit bed area, γ_b = the opposing force per unit boundary area resisting migration, h = bank height, r = bend radius and w = channel width

Dimensional analysis of the above equation yields

$$M = K \frac{\omega}{\gamma_b} \left(\frac{h}{w}, \frac{r}{w} \right)$$

The specific purpose of this study was to seek a correlation M (π, γ_b, α, ω) for which a dimensionally balanced expression is

$$\frac{Mh}{w} = K \frac{\omega}{\gamma_b}$$

An earlier study of the authors on the Beatton river showed that channel migration rates are strongly controlled by bend curvature (Fig. -3). A similar data analysis was made for all the rivers taken into consideration and a curve was developed displaying the same basic form (Fig. - 4). Then the entire data set where M>o was transformed to a reference curvature ratio. The curvature ratio adopted in the study was r/w =2.5 which represents the crest of the envelope curve in Figure 3 and 4. It permits the relatively simple transformation of the data points on both limbs of the distribution using the follow equation.

$M(r/w) = M_{2.5}.f(r/w)$

Where, $f(r/w) = 2/3\,(r/w - 1)$ if $1 < r/w < 2.5$

$f(r/w) = 2.5\,(r/w)$ if $r/w < 2.5$

During the data collection of the study it was considered from the field experience that five year flood corresponds to the morphological bankful flow. The slope parameter was the mean water surface slope at the 5 year flood. The maximum erosion rate occurs for r/w= 2.5 and increases with total stream power

$\Omega = Q_5 I \rho g$

The standardized channel curvatures ($M_{2.5}$) had been averaged to obtain a single representative migration rate for each reach. The relation between $M_{2.5}$, h and Ω discriminates on bank material type and the 45^0 separatix implies that $M_{2.5}/\pi$ = a constant for any given bank strength condition:

$$\gamma_b = \frac{\Omega}{M_{2.5} h}$$

γ_b has the dimensions of force/area and is a co-efficient of resistance to lateral migration. γ_b is a function of bank material size and can be obtained from the relation of bank strength to the textural character of basal sediment in the outer banks of the channel.

Klaassen and Masselink

The authors studied the bank erosion of the Jamuna River of Bangladesh and reported the findings in their paper entitled ' Planform Changes of a Braided River with fine sand as bed and bank material'. The river is a large braided sand bed river. The number of braids varies between 2 to 3 and the total width of the braided Channel pattern varies between 5 and 17 Km. The yearly erosion rates were studied based on cross sectional data and planform data derived from LANDSAT imageries. The bank erosion rates along curved channels for four different periods were observed. Information about tbe average number of occurrences for the four different periods appears in Fig.- 5. It shows in most of the cases erosion rate along the curved channels is between 0 and 500 m/year. The bank erosion rate might be upto 1000m/year under exceptional conditions. With respect to the influence of vegetation on erosion rate it was found from the study that influence of vegetation is negligible. It was also found that rotation and extension type of erosion mechanism was active in the Jamuna river .Translation type of erosion mechanism was absent because the chars and floodplain deposits exhibit hardly any cohesion. The bank erosion rates were analysed within the framework similar to the work of Nanson and Hickin (1985) for meandering rivers. The expression of bank erosion give by Hickin and Nanson was reduced to the following assuming that Chezy coefficient and overall bank resistance coefficient donot vary along the river.

$E = f$ (w, R/w)

The values of the relative curvatures (R/w) for the two years were averaged. However, if one value of R/w was smaller than 5 and the other was greater than 5, then the value of the bend with the smallest relative curvature was used. This is because it was considered that this bend was most active in the eroding process. A plot of E/w versus R/w shows that low relative curvatures lead to relatively fast erosion rates and vice versa. The value of "w" used in the study corresponds to low flow width. No sharp bend was found to have smaller erosion rates, as was observed by Hickin and Nanson for meandering rivers. Another important finding from the study was large channels (w > 1000m) demonstrated relatively smaller erosion rates whereas there was no significant difference in relative erosion rates between the smallest (w <500m) and the intermediate (500< w < 1000m) channels.

D. J. Hughes (1976)

The author investigated the rates of erosion around the meander arcs in relation to peak discharges, based upon the analysis of data recorded at meander locations in the river Cound catchment. Bank erosion was measured and discharge variations were monitored form January 1972 to September 1974 for a reach of the river having length of 1 Km. Erosion rates around the eroding arcs were monitored by using 92 bank pegs and measurement of distance from peg to channel margin were recorded at monthly intervals and more frequently during periods of high discharge. Profiles of the channel margin were measured every six months at 15 sites to show the nature of bank retreat in relation to the rates of erosion recorded at the pegs. A series of cross sections were taken at profile sites for indicating the total morphological changes of both bed and bank of the river in response to the discharges experienced. Discharge was monitored at the downstream end of the study reach.

It was seen from the study that the pattern of period rates of erosion is similar for each of the three individual arcs. The values of mean loss per site for each arc were also similar. During the study period two major peak discharges occurred. Recorded data showed that two floods together caused the greatest amount of erosion and contributed some 52%, 78%, and 47% of total erosion recorded in three arcs studied. Thus discharge magnitude appears to represent a threshold for major channel changes along the reach of the river. Of equal importance is the discharge required for a threshold of minimum activity below which little channel change takes place. 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 discharges representing the lower erosion threshold have a long term frequency of 10 to 12 times per year whereas the discharge representing the higher erosion threshold associated with widespread channel erosion occurs once in a period of 1.5 years.

Scope of the study on bank migration:

Infrastructure development along river and implementation of any river engineering project necessitate understanding of bank migration process and prediction of bank migration rates of the river concerned. Works of various researchers can be used as valuable tools to develop understanding of bank migration processes and possible future changes. However, regarding quantitative prediction of bank migration rates very few works are available so far. Hickin and Nanson (1984) estimated the yearly erosion for rivers in Canada. But the relations developed to make such estimate probably will not apply to all rivers outside of the region unless they are similar in terms of hydrology and geomorphology. Klaassen and Masselink (1992) used the Hickin and Nanson approach to analyse the bank erosion rates of braided Jamuna River and found substantial scatter in the plots of relative bend curvature versus the erosion rate. It was also seen that rates of erosion estimated according to Hickin and Nanson were smaller in an order of magnitude than what was actually observed. This reflects the necessity of inclusion of the duration of the yearly hydrograph in the analysis. The riverbank stability analysis made by Osman and Thorne (1988) to calculate lateral erosion distance is based on certain assumptions and simplifications. Therefore, in order to understand the erosion processes and to predict the migration rates of a particular river comprehensive study is needed based on available evidence and the available time spans over which change may be perceived. Attempt can be made to develop a deterministic model to predict future channel development and related bank erosion with reasonable accuracy by taking into account all relevant features in the analysis.

Concluding remarks

Bangladesh is the largest delta of the world. Since time immemorial the main land of this country was formed through alluvial deposit of the rivers. Many rivers with their tributaries and distributaries are snaking their ways through its landscape. As such, the role of these rivers in overall development of the economy of this country is of immense importance. It is important in terms of navigation, in terms of siting of infrastructures and industries along the riverbank, in terms of supply of nutrient rich sediment to the cropland, in terms of supply of irrigation and domestic water, in terms of hydropower generation and in terms of natural fishery. Implementation of water resources development projects and infrastructure development along the riverbank underscores the need for prior understanding of the river bank migration process. Otherwise the river may pose serious threat to the existence of the project. Enormous supply of sediment from the upstream, occurrence of a high magnitude flood and easily erodible character of bed and bank material can cause a major change in the river pattern at any time. Therefore, such possibilities should be assessed first before taking up a development project. However, due to very complex behaviour of these rivers, it is not easy to develop a deterministic model for predicting future channel development and related bank erosion. Clear understanding of the prevailing process may help reasonably in this regard. Available evidence from different sources can be used for analysis of the process. Measurement of changes by monitoring pegs in riverbanks is not practicable in hydrologic and geomorphologic reality of Bangladesh. However, personal observations and discussions with local people can provide valuable information regarding river stage and riverbank retreat during a particular flood event. The most important sources of evidence for detailed measurement and analysis of movement are maps, aerial photographs, spot imageries and cross sections of the river. The inclusion of the duration of the yearly hydrograph in the analysis will help to enhance the reliability of the prediction method. If chaotic behaviour of the river is found, a special study into the possible chaotic behaviour of the river system with the aim to identify this chaotic behaviour should be carried out to study its implications for the time span over which a reliable prediction of channel changes and bank erosion can be made (Klaassen and Masselink, 1992).

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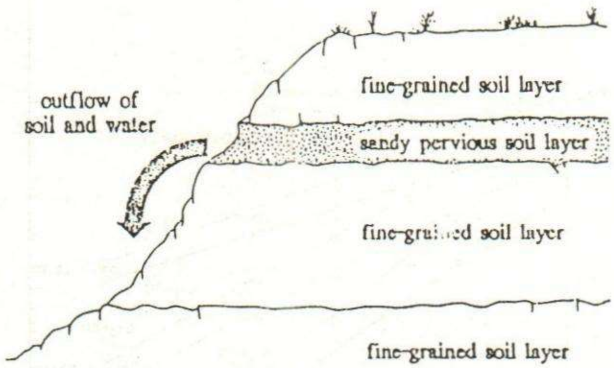

FIG. 1(*a*). Bank Erosion by Piping/Sapping with Subsequent Collapse: Seepage Outflow Initiates Soil Loss (after Hagerty, 1991)

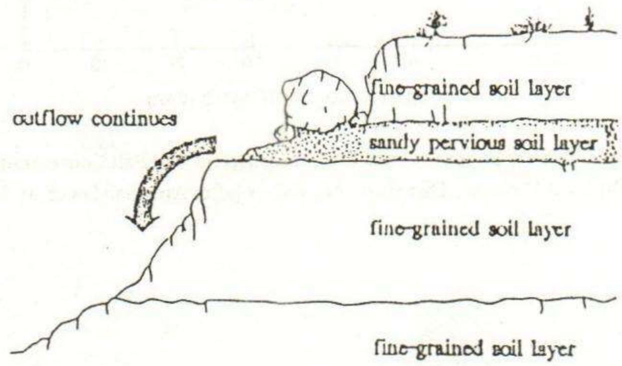

FIG. 1(*b*). Bank Erosion by Piping/Sapping with Subsequent Collapse: Undermined Upper Layer Fall, Blocks Detached (after Hagerty, 1991)

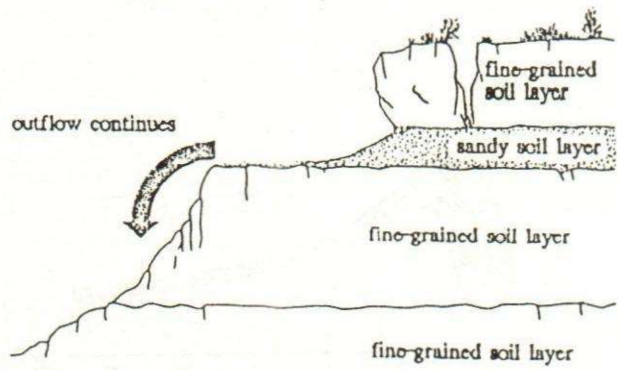

FIG. 1(*c*). Bank Erosion by Piping/Sapping with Subsequent Collapse: Failed Blocks Topple or Slide (after Hagerty, 1991)

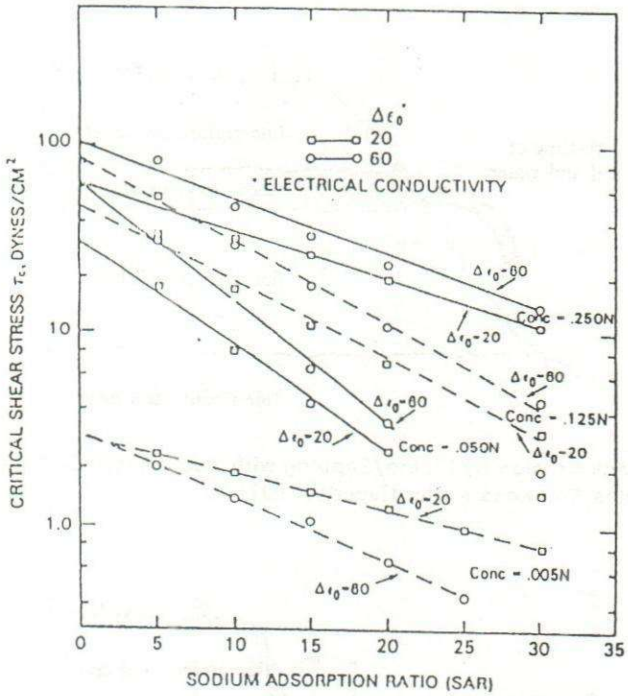

Fig. 2 : Critical Shear Stress τ_c versus SAR for Different Soil Salt Concentrations and Different Dielectric Dispersion $\Delta\varepsilon_c$ Values (after Arulanandan et. al. 1980)

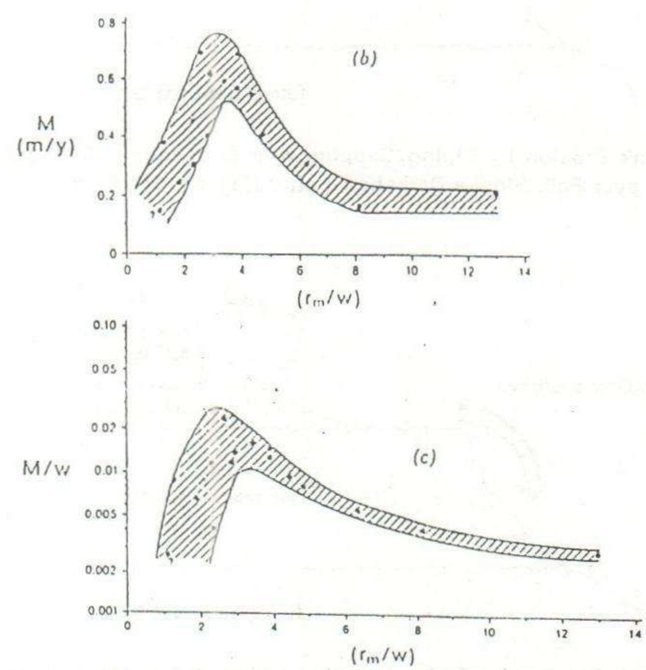

Fig. 3 : Channel Migration Rates versus Bend Curvature (after Hickin and Nanson, 1983)

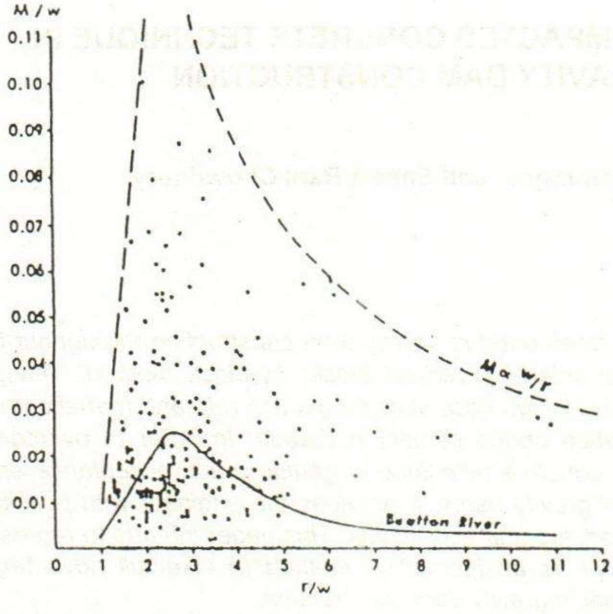

Fig. 4 : The Relation Between Relative Migration Rates and Bend Curvature Ratio for all Field Sites (after Hickin and Nanson, 1983)

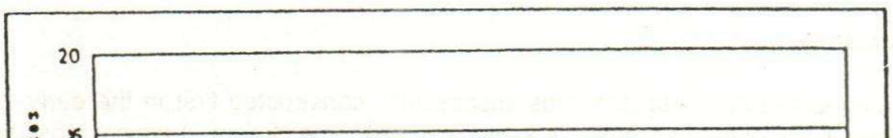

Fig. 5 : Average Bank Erosion Rates Along Curved Channels of the Jamuna River for the four Periods (after Klaassen and Masselink, 1992)

ROLLER COMPACTED CONCRETE TECHNIQUE IN GRAVITY DAM CONSTRUCTION

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Abstract

Conventional concrete has been used in gravity dam construction throughout the world for long. The dam is constructed by vertical block -by-block method. The problems associated with this method are high cost, slow production rate and formation of thermal cracks due to heat generation during cement hydration. In order to overcome these problems roller compacted concrete technique is getting much acceptance among the engineers in construction of gravity dams. It provides the engineer with possibilities for concrete dam designs, schedules and economies. This paper intends to represent roller compacted concrete concept as an innovative method to take full advantage of the characteristics of this material in gravity dam construction.

Key words

Roller compacted concrete (RCC), Thermal crack, Thermal stress, Pore pressure, Segregation, Workability.

Introduction

Roller compacted concrete dam was successfully constructed first in the early 1980's. Since then it has gained acceptance as a viable new type of dam. A typical cross section of RCC dam appears in **Fig.1**. Generally horizontal lift-by-lift method is adopted with RCC. The mixing, transportation, spreading and compacting of large volumes of material are concentrated into a short time interval. The concrete is brought to the dam site in trucks and compaction is done by rollers. This makes the rate of placement much faster and thereby reduces the cost appreciably. C. Teodorescu , et al (6) reported some important advantages of the use of RCC at Vadeni and Tirgu-Jiu spillway dams in Romania which include a reduction in cement consumption of more than 40 percent compared with the equivalent volume of conventional concrete that would have been required, elimination of formwork for concrete lifting,elimination of cranes for formwork operation and RCC placement and productivity related to the RCC volume increased at least twice. There are different opinions in RCC dam design particularly with respect to RCC mixtures. This is due to the fact that the development of RCC technology came from two distinct civil engineering disciplines, namely from geotechnical engineering and from concrete engineering. There are more than one way to approach any specific item of design and construction. Therefore, the basic properties of RCC combined with its rapid placement, construction and maintenance advantages have rendered it as a viable material for dam. With the passage of time roller compacted concrete technology has matured to the point where RCC dams are ready to take their rightful place among the major dam types.

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What is RCC?

RCC is a new type of construction material. It is concrete used in the construction process which combines rapid placing techniques of embankment materials and excellent material properties of concrete. It has a " no slump" consistency and is densified by external loading of the roller. It requires much lower water content than for comparable strength conventionally placed concrete and provides the ability to support compaction of spreading equipment. It is a simple material to produce and place and it requires no unique construction facilities or equipment.

Production of RCC

Material for RCC

The materials for RCC are cementitious materials, aggregates, water and admixtures.

Cementitious materials

The quantity and type of cementitious materials required for use in the RCC depend on size and shape of the structures, its required properties, the exposure conditions and the characteristics of the aggregate. The strength of RCC is dependent on the proportions of the cementitious materials. However, total heat generation within the mass is governed by the presence of total cementitious materials. Therefore, in case of structures with high strength requirement, cement with lower heat generation properties or replacement of a part of cement with pozzolans or fly ash can be considered to minimize heat generation. RCC containing pozzolans and fly ash generally results in reduction in the early strength levels of the concrete. However, it does not bring about any loss of long term strength.

Aggregates

The quality and properties of RCC are significantly affected by the selection of aggregates and control of aggregate grading. Size of aggregate does influence compactibility and segregation. Use of maximum size aggregate reduces the voids content of the aggregate and thus reduces the paste volume and potential for thermal cracking. However, increase of maximum size aggregate in RCC causes problems in mixing and handling RCC, including segregation.

Water

The water content is controlled to achieve maximum density during construction. The amount of water will correspond to a moisture level just below where pore pressure of the moisture develops and just above where segregation begins to be a problem.

Admixtures

The use of admixtures in RCC is effective in entraining air, reducing water and retarding set. Water reducing and set retarding admixtures result in significant delays in setting and heat generation. It can be considered useful in keeping a joint 'live" contributing to greater bond potential.

RCC mixture proportioning

There are two methods of RCC mixture proportioning (Hansen and Reinhardt, 1991) namely

1. Geotechnical approach mixture proportioning method and

2. Concrete approach mixture proportioning method

In geotechnical approach mixture proportioning method the "optimum" water content for trial mixes is determined either by observing the consistency of the mixes of varying water contents and by relying on past experience or by the moisture density principles , using impact compaction with a standard hammer dropped a prescribed number of times. After fixing the aggregate grading and water content, laboratory specimens are prepared with varying cementitious contents and compressive strength is tested at 3, 7, 14, 28, 90, 180 and 365 days. Thus a family of curves can be obtained indicating the effects of various cementitious contents on compressive strength at various ages. The cement content can be selected to meet structural requirements. With the selected cement content additional tests can be done with varying aggregate types and gradings.

Mix design methods based on concrete approach usually involve fixing all but one of the basic materials and then varying that component until the desired consistency or required properties are achieved. Each variable can be adjusted this way to optimize all mix components. All of the methods are based on a Vebe time indicating full consolidation of the RCC. The basic principle of these methods is that the volume of paste must exceed the voids in the aggregate. In high paste concrete approach method sufficient cementitious material is needed to achieve low permeability and bonding between lifts. However, the volume changes occurred due to heat generation by the cementitious material is also important to be taken into account. Following table shows the typical high paste RCC mix proportions used in the New Victoria dam (Wark and Mann, 1992)

Table 1: Typical high paste RCC mix proportions

Coarse aggregate (kg/m3)	1415
Fine aggregate (kg/m3)	740
Portland cement (kg/m3)	80
Fly ash (kg/m3)	160
Free water (kg/m3)	105

The table indicates that liberal substitution of pozzlans and fly ash for cement is possible which helps to reduce the problem of heat generation within the concrete mass. In this regard decisions can be made based on the specifics of the site. Depending on the cementitious materials content or the quantity of fines, the RCC water content will be between 90 and 130 kg/m3; this allows for placement with earthmoving equipment and compaction with heavy vibrating rollers (Sterenberg,1992)
The concepts of the Japanese RCD mix proportioning are to have a relatively wet mix with a high fines content and to keep the cement content as low as possible while being consistent with strength requirements. An example of RCD mix design appears in the following table (bulletin 75, ICOLD):

Table 2: RCD mixture design

Maximum size of coarse aggregate (mm)	150
Vibrating compaction (VC) value, seconds	20 ± 10
Water/ cement (including fly ash) ratio %	70-85
Sand/aggregate ratio %	150
Water (kg/m3) Cement and fly ash (kg/m3) Fine aggregate (kg/m3) Coarse aggregate (kg/m3)	90-110 120-130 657 1544

As much as 30 percent of the unit cement content is fly ash. It retards the set, helps to reduce thermal stress from the heat of hydration and enhances the long-term strength requirement. In order to enhance workability, the sand content is as high as 30 to 34 percent. The evaluation of the consistency of the RCD mix is done by varying the sand to aggregate ratio, water content and cementitious material content. The VC time is evaluated to provide the most workable mix along with observation for the segregation of the aggregates.

RCC placement and compaction

The RCC placement consists of the following operation:
- Water flushing and air blasting of the bedding concrete
- Placement of contact concrete and
- Placement of RCC according to specification

The contact concrete is a high slump concrete used to achieve bond between the layers. It should be placed on the lifts just ahead of the RCC placement. The placed RCC should be leveled by use of a bulldozer into several meter wide layers. The layer thickness ranges from 30 to 90 cm. In order to provide an even layer , the bulldozer should be used to level each heap individually rather than a series of successive heaps. It is important that each transported load should be released on the previous layer and that the bulldozer could push it forward. In this way, the materials of the heap are mixed again and any accidental segregation clusters can be avoided. Each RCC layer should have a crossfall gradient on the order of 20 H to 1 V to as flat as 50 H to 1 V falling from d/s to u/s. The crossfall is helpful to drain the surface of each layer during construction. It also provides additional sliding resistance along the potential plane of sliding between layers of RCC in the completed structure. RCC is rolled into a dense mass by external equipment energy. Compaction should be performed as soon as possible after the "no slump" concrete is spread. Full compaction can be achieved, however, any time before initial set. The number of passes required for the full consolidation of RCC should be determined. First one or two passes should be made without vibration and the rest passes should be made with vibration at a specified frequency. Overcompaction can lead to lower densities and should be avoided. **Fig.2** shows the typical RCC placement and compaction operation.

Control of temperature

Control of temperature can be achieved by reducing the cementitious material content.But such measures should be consistent with strength and permeability requirement of the concrete. Precooling of RCC to attain a expected placing temperature is also well predicted and it can be done by use of the following method:

- Winter production of RCC and stockpiling it in its cool condition
- Spraying water
- Using chilled mix water at 4^0c
- Injecting liquid nitrogen into the cement and fly ash during pneumatic transfer from the delivery tankers into the onsite silos

The placing program should be fast enough to achieve the full benefit of the precooling of RCC. Placement of RCC at night is used to reduce the placing temperature. Use of moderate heat cement and reduction of lift thickness can also be considered to control the temperature rise.

Control of cracking

The cracks are the result of tensile stresses within the dam caused by shrinkage associated with cooling of the RCC and stress concentration at abrupt changes in the foundation. The potential for cracking in RCC dams is unique to each site and dependent on the heat rise within the mass, the ambient temperature, the thermal , elastic and strength characteristics of the RCC. The development of temperature in the dam body is shown in **Fig.3**. Control of thermal cracking due to adiabatic temperature rise within concrete mass and stress developed during cooling can be largely achieved by strictly controlling RCC placement temperature, restricting the placing time and taking further precautions and measures which would severely restrict the construction of the dam (Hollingwoth and Geringer, 1992). RCC with high tensile strength and low modulus of elasticity results in high tensile strain capacity. High tensile strain capacity means reduced cracking potential. Since aggregate can have major effect on the tensile strength, modulus of elasticity and coefficient of thermal expansion of the RCC, potential for cracking can be minimized by appropriate selection of the aggregates. It is important to note that a concrete dam will crack at points of least resistance. RCC method of dam construction provides good scope to install joints at specific locations where cracks may be formed. By installing water stopped and drained joints partially or completely through the dam control of cracking can be achieved.

Control of seepage

Seepage control can be achieved by improving the design of the crack inducers and directors, stressing quality control in construction and care especially in the installation of water stops. Seepage is greatly reduced when there is effective bonding between successive layers of RCC. Treatment of the construction joints between RCC layers is used to improve the bond between the old and new layers. In order to achieve this, the surface of RCC construction joints are treated to eliminate some carbonation or dirty material and a thin layer of bedding mix is used. In order to ensure watertightness of the RCC dam an impermeable or relatively impermeable upstream face can be considered. A waterproof membrane at the upstream can be provided as an additional safety measure. The seepage through the RCC itself can be controlled by proper mix design. The mix design should aim at minimization of void ratio so that RCC will be less permeable.

Stability requrements

The stability of the RCC dam essentially means its stability against sliding and overturning. In the analysis of stability against overturning , considerations should be given to the materials used and the subsequent unit weight of the in place concrete along with the adopted cross section. In design and construction of the RCC dam the required stability against overturning can be achieved by varying the base width and cross sectional shape. For example, the downstream face of a dam can easily be flattened or steepened as required to satisfy the stability analysis. In the RCC dam the stability against sliding within the concrete section is dependent on the cohesion, the coefficient of internal friction and the average normal load on the potential failure plane. The shear strength along the interface between RCC layers will be less than that through the RCC mass unless special precautions are taken , such as limiting exposure time between layers, placing bedding mix between layers, cleaning the joints and increasing the paste content within the RCC mix.

Conclusion

It is clear from the above discussion that since RCC provides a wide range of design and construction opportunities, the problems associated with conventional concrete method of gravity dam construction can be largely reduced by adopting RCC technique. Moreover, rapid construction and placement facilities of RCC result in low cost and time saving. It is a quite simple technique of dam construction and use of locally available materials can be considered.

It is obviously a matter of further investigation to take full advantage of RCC techniques in dam construction aiming at achieving of desired structural performance and also economy. Selection of the best design is very much complex because the various production rates and material properties that are possible with RCC. There are more options to evaluate and the factors influencing the final decision are closely interrelated. Arguments that make one design favourable at a particular site may be the same arguments that make it undesirable at a different location or for a different set of circumstances. Therefore, importance should be given to the solution that is site specific and based on sound judgement.

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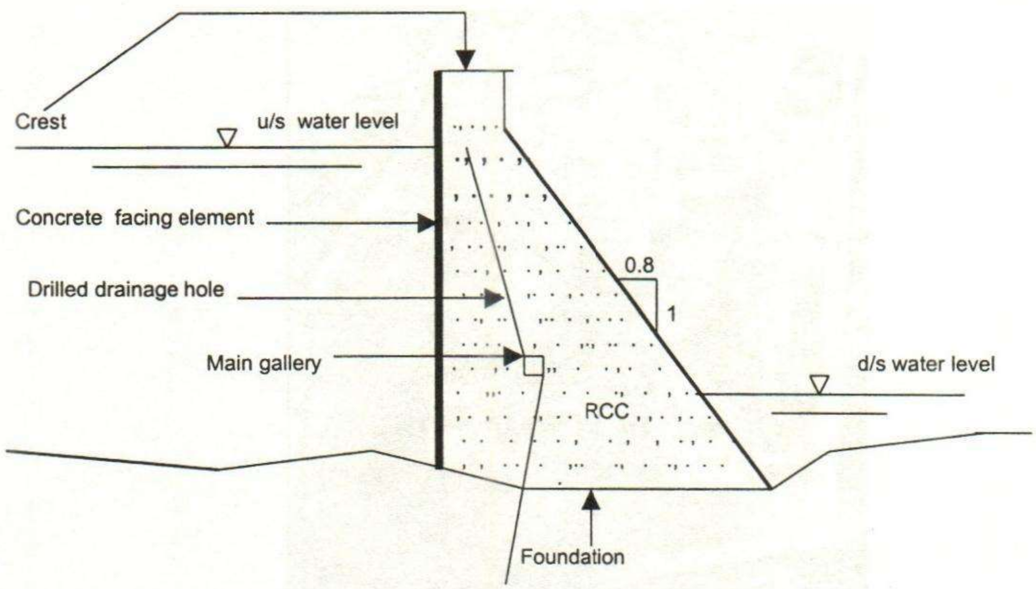

Figure 1 : Typical cross section of a RCC dam

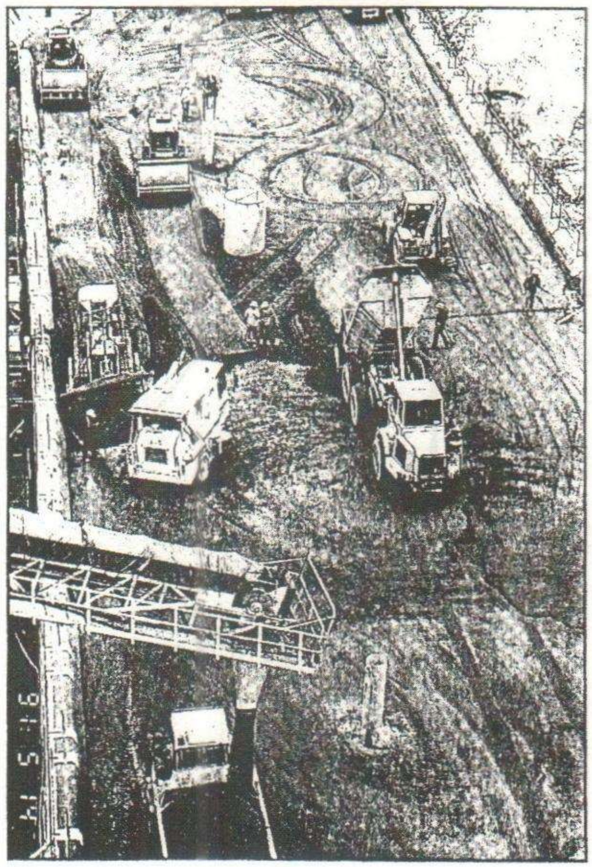

Figure 2: Typical RCC placement and compaction operation

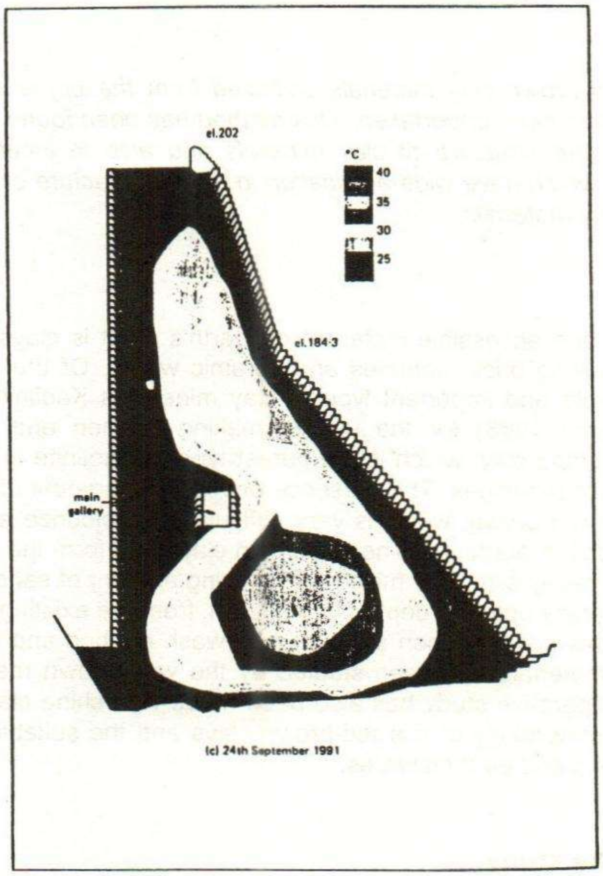

Figure 3: Development of temperature within RCC dam
(Source: New Victoria dam Australia)

INFRARED STUDY OF RED BROWN CLAYS OF DHAKA CITY

P.K. Roy[1], T.Hossain[2], S.H.M. Shamim Kabir[1] and Sheela Rani Chowdhury[3]

Abstract

Infrared study of red brown clay materials collected from the upper deposit of some areas of Dhaka city has been undertaken. This method has been found to be a potential tool in investigating the structure of clay minerals and also in locating some of its important properties which have wide application in the manufacture of various types of ceramic and refractory materials.

Introduction:

The most abundant and accessible materials on earth's crust is clays which form the basic material for making brick, potteries and ceramic wares. Of the various types of clays the most suitable and important type of clay mineral is Kaolinite (Al_2O_3, $2SiO_2$, $2H_2O$) (Ralph E. Grim 1968) for the use of making earthen and ceramic wares. Generally the white china clay, which is the purest type of Kaolinite in natural state, is widely used in ceramic industries. The presence of very little amount of iron oxide turns its colour into red or red-brown, which is very difficult to decolourize and thus it is not widely used for the above purposes. The red-brown clays that form the upper deposit of Dhaka city consists mainly silty Clay mixed with varying amount of sands and grits. The deposit extends generally upto the depth of about 7 m. from the existing ground surface. The clay portion of this soil has been separated by wash method and in this paper the quality of this clay materials has been studied by the well known method of infrared spectroscopy. A comparative study has also been made with china clay and this study will give an idea of minerology of this red-brown clays and the suitability of its use for manufacturing ceramic and earthen wares.

Methodology of the study:

Clay particle of the soil samples were separated by wash method and infrared spectra of the samples were recorded in the 400 - 4000 cm^{-1} kBr pallet technique using IR - 470 double beam spectrophotometer.

Experimental Details:

The soil samples were collected from eight different sites of greater Dhaka city and the clay portion, the particles size of which is less than 2μ is separated by wash method. The clay particles so separated were dried at a temperature of about 80^0C. Infrared study depicting some characterizing quality of this clay mineral has been studied. The infrared spectra of these samples and china clay were recorded in the 400-4000cm^{-1} kBr pallet technique using IR-470 double beam spectrophotometer with base line 0

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transmission expansion-5 and scan time 7 minute. The resolution is 1.5 cm^{-1} at 1000 cm^{-1}, 3 cm^{-1} in 3000 cm^{-1} region and the accuracy is ± 4 cm^{-1} in 4000-2000 cm^{-1} and ± 2 cm^{-1} in 2000-4000 cm^{-1}. The spectrograms of these samples are shown in Fig.-2 and the observed frequencies of individual samples are presented in Table-1.

Results & discussion:

The infrared spectra of five samples 1-5 shown in Fig.-3 are found nearly similar and all of them correspond to the well ordered Kaolinite structures as per comparison with the spectrum of China clay. The clay mineral can be characterized by the general form (Prasad J.1965).

$(Si_4O_6)[O_4(OH)] [(Al\ Fe)^{3+}{}_{4-2n} X^{2+}{}_{n}] (OH)_{6-4n}$

where n represents the degree of order [calculated as (OH) deficiency divided by four] (Keeling PS. 1963). In case of well ordered Kaolinite $n \rightarrow 0$; for large value of n i.e. n=1 the structure becomes disordered . The spectrogrames of rest of these samples 6-8 shown in Table-4 are not found as similar to those of China clay at higher frequency bands but nearly similar to those at lower frequency bands.

According to Keeling (Keeling PS. 1963) the bands at 3690cm^{-1} and 3620cm^{-1} can be attributed to outer and inner hydroxyl groups and 3690cm^{-1} band has a maximum intensity which indicates well ordered Kaolinite while 3620cm^{-1} does not change in intensity. The ratio of the intensities 3690cm^{-1} and 3620cm^{-1} is approximately 1 which indicates well ordered Kaolinite. Had this ratio been 0.25 it would have indicated disordered state (Ramaswamy K & Kamalakkannan, 1987). The absorption bands at zone 3500 – 3400 is least for ordered Kaolinite and only one band exists at 1625 – 1610 cm^{-1} for the samples.The existence of 1620 cm^{-1} corresponds to H-O-H deformation of hydroxyl group and is well reflected in all the samples studied. The weak bands at 1379cm^{-1} also indicates H-O-H deformation of hydroxyl group.

The strong distinct absorption bands at 1100,1030 and 910cm^{-1} indicate well ordered Kaolinite structure. According to Miller (Miller J.G. 1961) well ordered clays are characterized by the presence of 920cm^{-1},1120 cm^{-1} and 1030 cm^{-1} bands. Of these 920 and 1120cm^{-1} are due to Al-(HO) vibrations.

The degree of disorder can be ascertained from the behavior of these absorption band also. They are very distinct in well ordered clays but becomes weaker and are replaced by single band at 1030cm^{-1} as the disorder increases. In the samples 5, 6 & 9 the peaks 925cm^{-1} and 1120cm^{-1} merge with main peak with a broad absorption around 1030cm^{-1} which indicates the disordered behavior. The spectrogram of China clay shown in fig. 2 indicate sharp peaks displaying highly well ordered clays. The influence of iron in clays , shows peaks between 915cm^{-1} – 820cm^{-1}. The frequency between 1150 and 400cm^{-1} according to Former & Russel are called lattice vibration. In this range the layer Al-(HO) vibrations occures at 910$cm^{-1}$and below 795cm^{-1} (Si-O-Al) vibration present. Mixed SiO deformations and octahedral sheet vibrations indicate peaks at 430cm^{-1}

Conclusion:

From the analysis of infrared spectrograms of clays of different location it is concluded that the clays at Mirpur, Uttara, Dhanmondi, Mohakhali and Tejgaon are well ordered clays and are highly suitable for manufacturing ceramic, refractory and earthen-wares. The little iron contents which changes the colour of clay to red-brown not harmful for manufacturing coloured ceramicwares.

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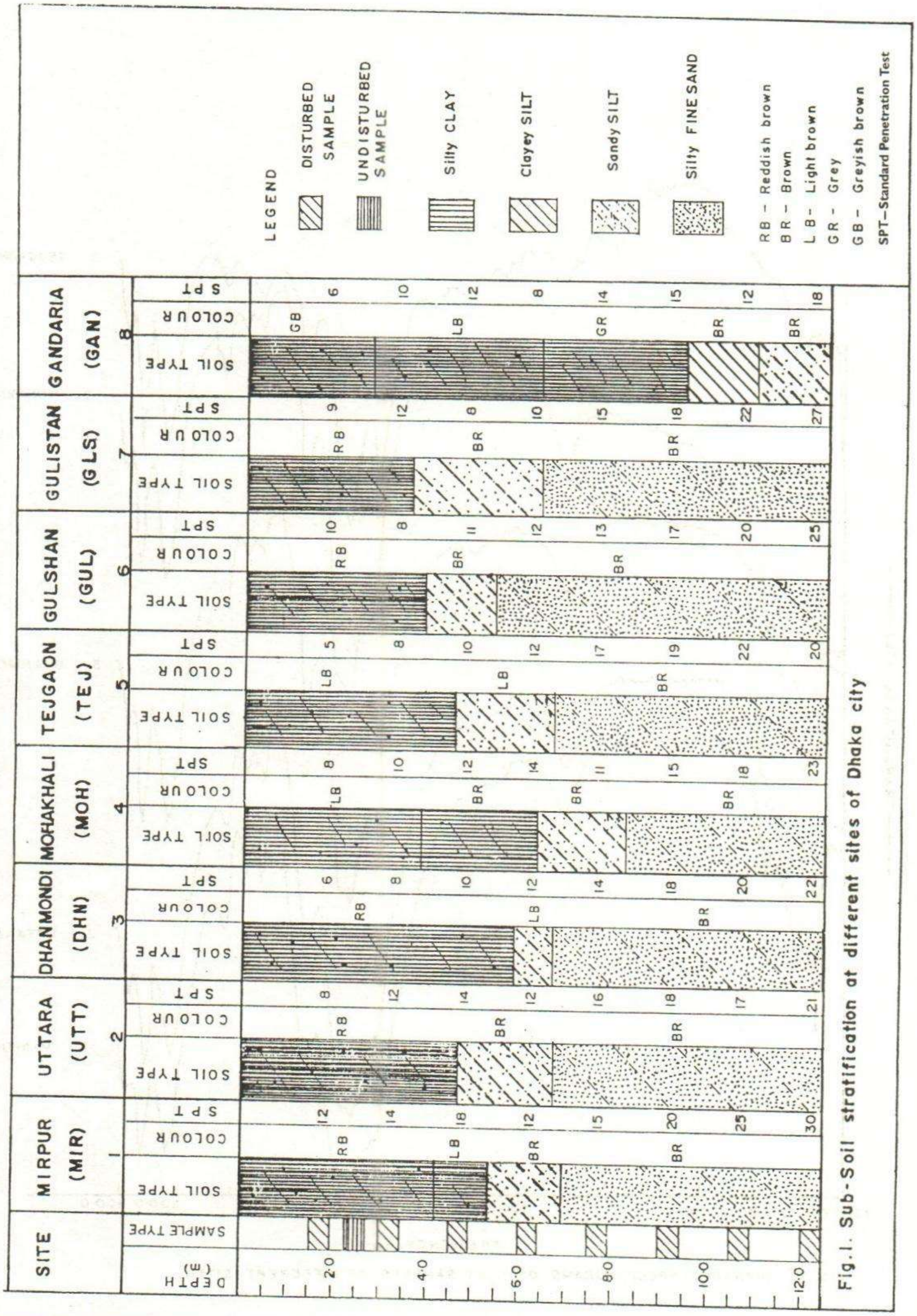


Fig.1. Sub-Soil stratification at different sites of Dhaka city

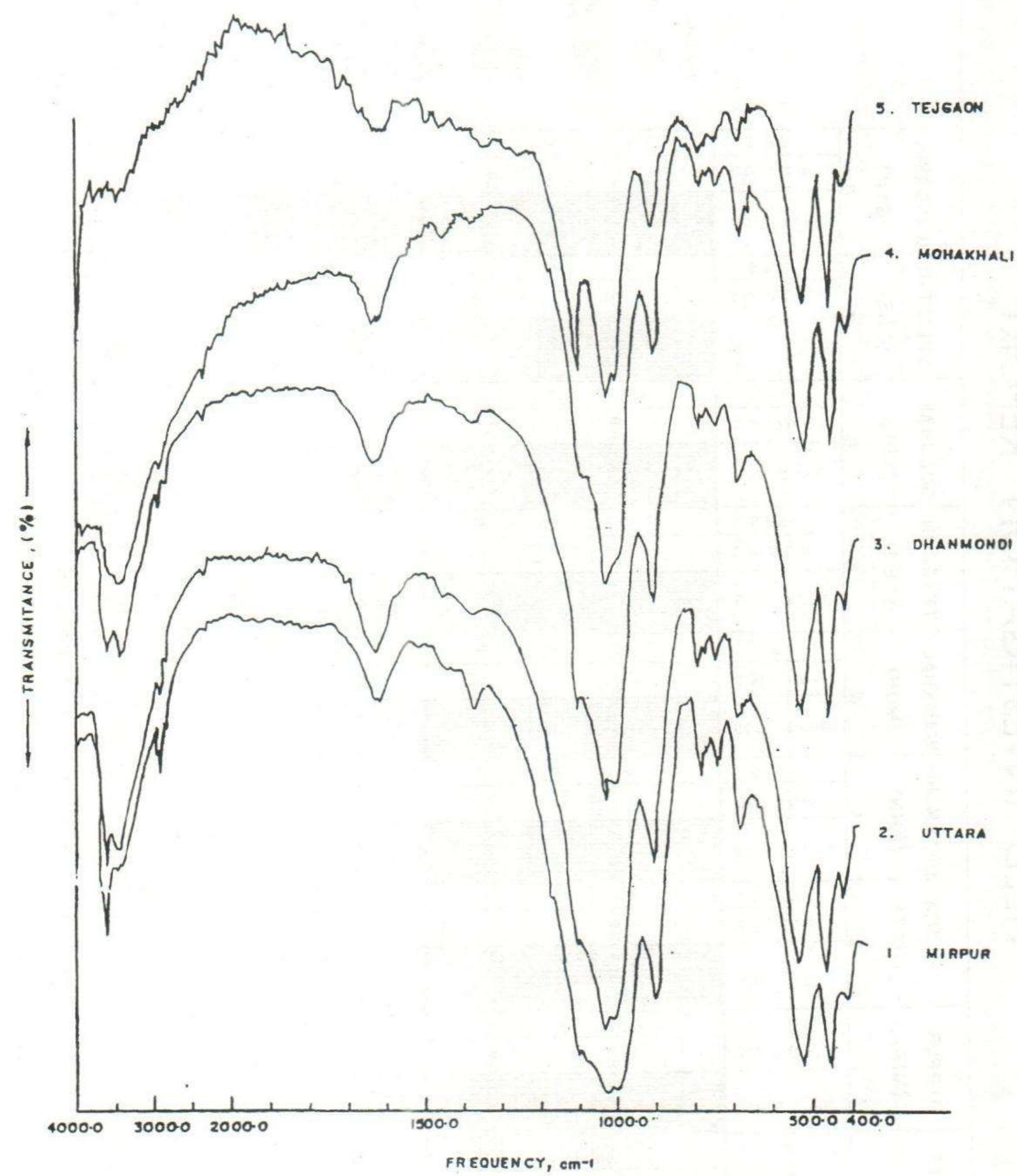


FIG. 2 INFRARED SPECTROGRAMS OF CLAY SAMPLES OF DIFFERENT SITES

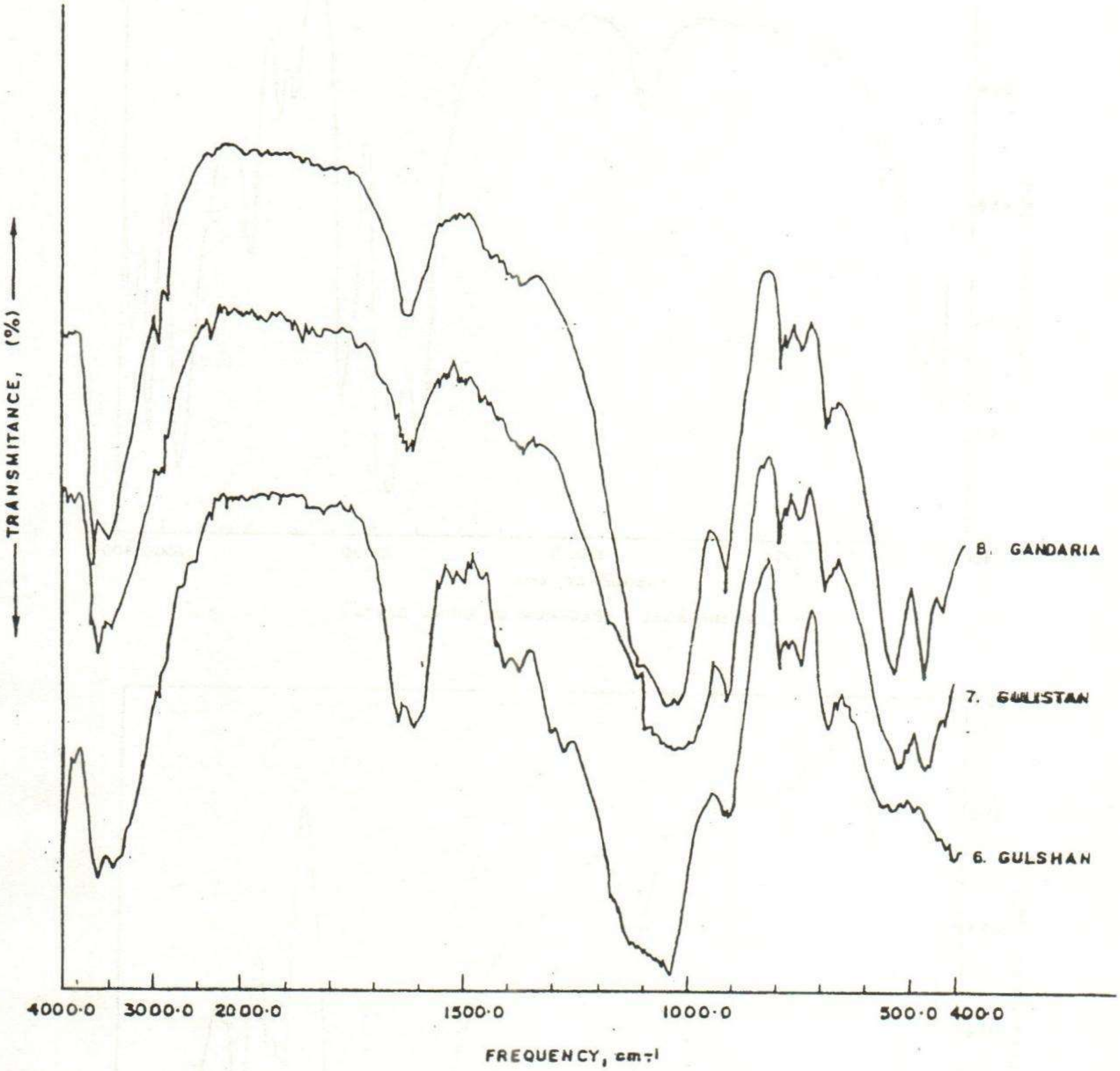


FIG. 3. INFRARED SPECTROGRAMS OF CLAY SAMPLES OF DIFFERENT SITES

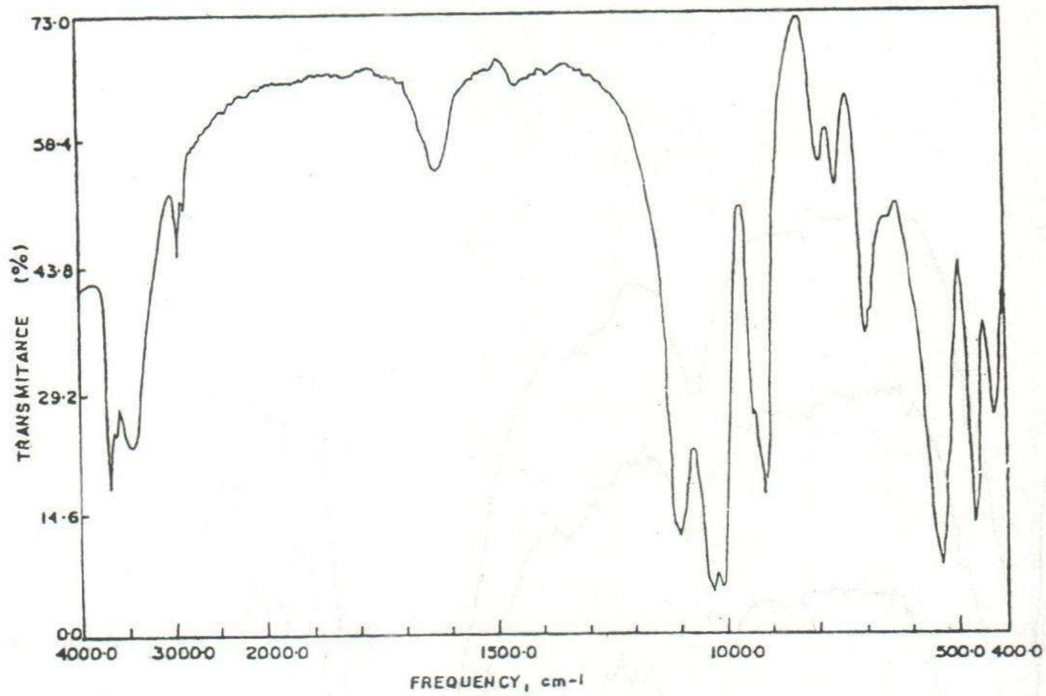
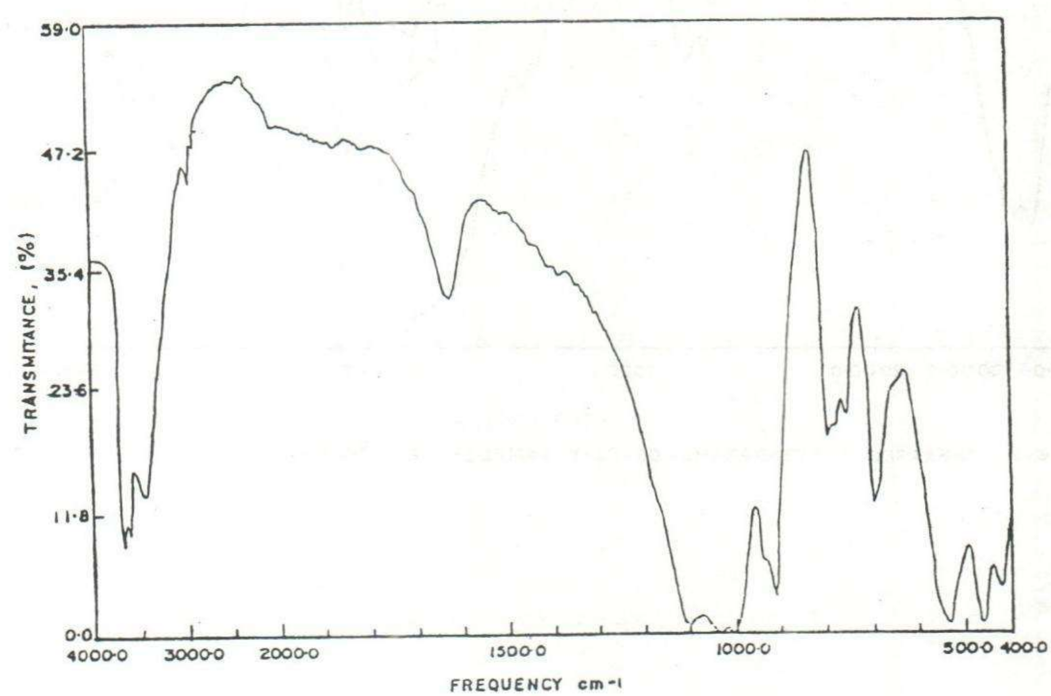


FIG. 4 INFRARED SPECTRUM OF CHINA CLAYS



FIG, 5 INFRARED SPECTRUM OF BIJOYPUR CLAYS(<0 002mm)

TABLE-1

INFRARED ABSORPTION SPECTRUM OF RED BROWN CLAYS OF DHAKA CITY

(Absorption Frequencies in cm^{-1})

Mirpur 1	Uttara 2	Dhanmondi 3	Mohakhali 4	Tejgaon 5	Gulshan 6	Gulistan 7	Gendaria 8	China Clay	Well Crystalised *	Poorly Crystalised*	Tentative assignment
3690	3690	3690	3690	3730	-	-	3690	3690	3695	3695	O-H stretching of hydroxyl sheet. O-H stretching of inner hydroxyl group.
3620	3620	3620	3620	3620	3620	3620	3620	3620	3619	3620	
3420	3420	3420	3420	3420	3420	3420	3420	3420	3410	3400	
2910	2950	2910	2910	-	2910	2950	2910	2910	-	-	
2350	2350	2850	2350	2350	2350	2350	2450	2850	-	-	
-	-	-	-	2160	-	-	-	-	-	-	
1620	1620	1625	1625	1610	1635	1620	1620	1620	1620	1620	H-O-H deformation of hydroxyl group Al-(OH) vibrations (Si-O) stretching Al-(OH) vibrations
1375	1375	1375	1385	-	1380	1375	1375	1450	-	-	
1110	1100	1100	1100	1100	-	-	-	1100	1103	1104	
1030	1030	1030	1030	1030	1030	1030	1030	1030	1036	1035	
1005	1005	1010	1010	1005	-	-	-	1005	1010	1009	
910	920	910	910	910	910	910	910	910	913	914	
795	795	795	795	795	795	795	795	795	791	795	(Si-O-Al)
750	750	750	750	750	750	750	750	750	753	755	
690	690	690	695	695	690	690	690	690	700	700	
535	535	535	535	535	535	525	535	535	542	540	Mixed SiO deformation and octahedral sheet vibrations.
465	460	465	465	460	465	470	465	465	472	470	
420	430	420	420	425	425	420	425	425	432	430	

* Source : Prasad J. 1965

ESTIMATION OF WATER LOSS IN EARTHEN CANAL

S.M.Anwaruzzaman[1], Md. Azizul Haque Podder[1] and Md. Abul Ala Moududi[1]

Astract

This study was undertaken to estimate the water loss in the earthen irrigation canals of four shallow tubewell projects situated in Shalikha and Sangramshimul mouzas of Madhupur thana of Tangail district. The water loss in the study area was found ranges from 3.10 percent to 6.95 percent of the pumped discharge with an average of 5.20 percent and coefficient of variation of 15 percent. As a matter of fact, the canal conditions under all the projects are in good condition; and hence the loss rate is relatively low in comparison to those reported elsewhere.

Introduction

Water losses in irrigation distribution networks with earthen canals occur in various ways; of these steady state losses, transient losses, and wastage are the major components. The steady state losses include seepage in to bed and bank soils, normal infiltration and excess seepage into holes and cracks, visible leakage through and over the banks (over topping and leakage through banks and closed outlets) and evaporation from water surface. The transient losses include initial seepage into dry banks in excess of other short-term leakage resulting from bank washouts, breaches and broken outlets.

In the farmer's field, water losses in irrigation canals in Bangladesh vary due to uncontrollable cracks, holes and burrows (Khair and Dutta, 1983).

Very recently, modern irrigation is a great problem due to frequent fuel price hike and scarcity of irrigation water during the dry season. Expansion of irrigation practice is causing the shortage of available water in the dry season for irrigation; and fuel price hike has also seriously threatened to maximize crop production with the minimum use of irrigation water. But water loss through the canals in minor irrigation projects is of great concern. As reported, this loss is generally as high as 50 percent of the total water pumped diverted, although it varies from soil to soil. Canal water loss causes extra expenditure to pump water and also reduces the command area as well as the conveyance efficiency of irrigation. Farmers are facing these problems which not only do reduce irrigation coverage but also complicate the management at the farm level. Therefore, on-farm water control measures for avoiding water losses in the irrigation distribution systems are realized even by the farmers.

Before planning an irrigation scheme, it is therefore needed to know the extent of water loss incurred in irrigation distribution system. The above research investigations reveal that water losses do occur in the irrigation distribution system resulting in wastage of not only fuel and water but also it deteriorates the environment as a whole. As a matter of fact, water is not a free goods, but it appears from the real world situation prevailing in Bangladesh that water suppliers have been mining the groundwater as much as they can within their capacity without concern to its harmful effect of the environment. This situation

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must not be allowed and precaution must be taken to prevent the environment from degrading.

This study was undertaken with an overall aim of assessing the extent of water loss in the minor irrigation projects and thereby to apprise irrigation planners to incorporate it in the future national irrigation policy. To fulfill the overall aim, the following objectives were set;

(i) to estimate the pump discharge, and
(ii) to estimate the water loss in the canals.

Description of the study area

This study is based in Shalikha and Sangramshimul mouzas of Madhupur Thana under Tangail district (fig.1). Four Shallow Tubewell (STW) were chosen under the irrigation projects to estimate the water loss in the irrigation canal networks are shown in Fig.1. All the projects are largely dominated by silt loam soil with a relatively flat topography (0-5% slope) falling under the Young Brahmaputra Flood Plain. The lands can be categorized as medium high land to high land dominated by the former one. This study area is suitable for growing cereals by HYV boro and amon together with vegetables. The average annual rainfall and evaporation in this area are about 2000 mm and 950 mm, and intensities of those are high in August-September and April-May, respectively (Bangladesh Bureau of Statistics, 1987). The daily average temperature ranges from 15 to 30^{0}C and average daily humidity ranges from 60 to 95 percent.

Experimental methods

Theoretical consideration

The water losses incurred in the irrigation canals of the four STW were measured using the following formula (Skogerboe, 1973) :

$$W_L = \{(Q_p - Q_d)/Q_p\}*(10000/d) \qquad (1)$$

Where,

W_L = Water loss in percent of the pumped discharge per 100 meters length of the canal;

Q_p = Pump discharge in lps; and

Q_d = Discharge at distance of 'd' meters from the pump in lps.

Estimation of discharge

The flow in the irrigation canals may be either free flow or submerged flow depending on the transition submergence S_t (the value of submergence at which the discharges passes from free flow to submerged flow or vice versa). If the submergence S(defined as the ratio expressed as a percentage of the downstream depth, h_b to the upstream depth, h_a) is less than S_t, the flow is free. In this study all the flow was free and the following equation given by Skogerboe, 1973 was used.

$$Q = C_1(h_a)^{n1} \quad (2)$$

Where,

Q= flow rate in cfs

C_1= free flow coefficient

h_a = upstream flow depth in ft; and

n_1= constant depending on length of the flume

The value of C_1 is again given by

$$C_1 = K_1W^{1.025} \quad (3)$$

Where,

K_1= flume length coefficient = 5.21 (taken from graph; Skogerboe, 1973)

W= throat width = 0.50 ft.

Under the free flow condition for this "15*60" cm cutthroat flume, the modified formula incorporating constant becomes as follows

$$Q = 2.52\ h_a^{1.98} \quad (4)$$

Discharge of the pump was measured as far as possible close to the discharge box following the standard rules; and thereafter loss incurred in the canal section was incorporated to determine the pump discharge.

Data collection and presentation

During the field investigation the pump discharge, command area, length, density and dimension data of the canals were collected and are presented in table 1. The flows at different sections of the canals were measured by cutthroat flume of the above mentioned STWs for the estimation of water losses.

Results and discussion

It can be observed from table-2 that the water loss in percent of the pumped discharge per 100 meters length of the canals varies from 3.10 to 6.36 with an average of 5.34 and coefficient of variation (CV) of 17 percent in the case of STW-1. Whereas, it ranges from 3.40 to 6.80 with an average of 5.06 and CV of 21%; 4.10 to 6.38 with an average of 5.15 and CV of 19%; and 3.10 to 6.95 with an average of 5.23 and CV of 13% in the cases of STW-2, STW-3, and STW-4, respectively. Although it is seen that the average water loss rate is more or less equal within the projects, the loss rate over a certain length even in the same canal varies remarkably. The loss rate determined in this undertaking reveals that it is relatively low in comparison to those reported elsewhere (Dutta 1985, Dutta and Sarker 1986, Jenkins 1981). But this loss rate was expected to be low because the core research team made an effort to improve the water distribution system by constructing the discharge box, aligning the irrigation canals, making the bed slope according to the land topography and above all by compacting the canal subgrades. In addition to that it is worth to mention that most of the canals are dug reasonably into the original land. The water loss in percent of the pumped discharge for the canals in all the irrigation projects is found to be more or less linear means that the rate of loss is uniform over canal length.

Finally, an attempt is made to express the cumulative water loss as a function of the distance from the pump (fig.-2). The simple linear regression analysis has yielded the following equation.

$$Y= -0.3633+0.0542X \quad (5)$$

Where,

Y= Water loss in % of pumped discharge; and
X= Distance from the pump in meter.

The higher correlation coefficient (r^2 = 0.9838) shows a better relationship between the loss and distance.

Conclusions

This study was undertaken to evaluate the level of water loss in the irrigation canal network in the four STW projects. The following conclusions are made out of the study.

(i) The pump discharge is by and large below the rated discharge resulting probably from low rpm and overall maintenance of the engine.

(ii) The cross-section of the canals vary widely within the irrigation projects. Moreover, the canal density warrants further improvement to reduce the water loss as well as the operation and maintenance cost of the irrigation unit.

(iii) The average water loss in the study area in percent of the pump discharge per 100 meter of the canal is 5.20 which can be graded as fair.

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Table-1: Characteristics of shallow tubewell projects in the study area

Tubewell identi-fication	Command area(ha)	Average Pump Discharge(lps)	Canal Length(m)	Canal density (m/ha)	Average cross sectional area (m^2)
STW-1	4.25	11.64	1010	238	0.1062 (0.069-0.131)
STW-2	3.16	11.18	1010	319	0.1062 (0.069-0.131)
STW-3	3.64	11.93	1100	302	0.1009 0.054-0.149
STW-4	6.67	12.51	1350	202	0.0876 (0.05-0.126)
Average	4.43	11.81	1117.5	265	0.0994
C.V.(%)	35	5	14	21	28

Table-2: Water loss in percent of the pumped discharge per 1000 meters length of the canals in the study area

Distance from Pump	STW-1				STW-2				STW-3				STW-4		
(meter)	M_1	M_2	M_3	M_4	M_1	M_2	M_3	M_4	M_1	M_2	M_3	M_4	M_1	M_2	M_3
50	6.36	4.82	6.36	3.1	5.03	6.8	6.8	3.4	6.38	4.86	5.18	5.2	4.64	6.24	3.1
100	5.58	4.81	6.36	3.95	4.29	6.53	5.72	4.11	6.29	5.53	4.1	4.94	4.48	6.95	3.93
150	5.79	4.81	6.3	3.72	4.75	5.96	5.43	4.95	6.31	5.75	4.85	4.85	4.11	6.61	4.69
200	5.89	4.93	5.54	3.95	4.88	5.68	5.28	4.88	6.21	5.83	4.39	4.81	4.2	6.44	4.67
250	5.94	4.71	5.32	4.09	4.83	5.76	5.44	4.83	6.17	5.87	4.42	4.72	4.25	6.3	4.34
300	5.93	4.7	4.95	4.44	4.8	5.98	5.55	5.07	6.12	5.87	4.46	4.69	4.29	6.21	4.34
350	5.71	4.86	5.3	4.44	4.75	5.83	5.63	4.98	6.1	5.87	4.88	4.66	4.28	6.12	4.56
400	5.89	4.98	4.98		4.74	5.84		4.7	6	5.83	4.79	4.79		6.06	4.75
450	5.86		4.98		4.69			4.69	5.96	5.8		4.74		5.67	
500			4.98					4.53		5.63		4.87			
550												5.02			
Average	5.34				5.06				5.15				5.23		
C.V. (%)	17				21				19				13		

Note: M_1, M_2, M_3 and M_4 refer to main canals.

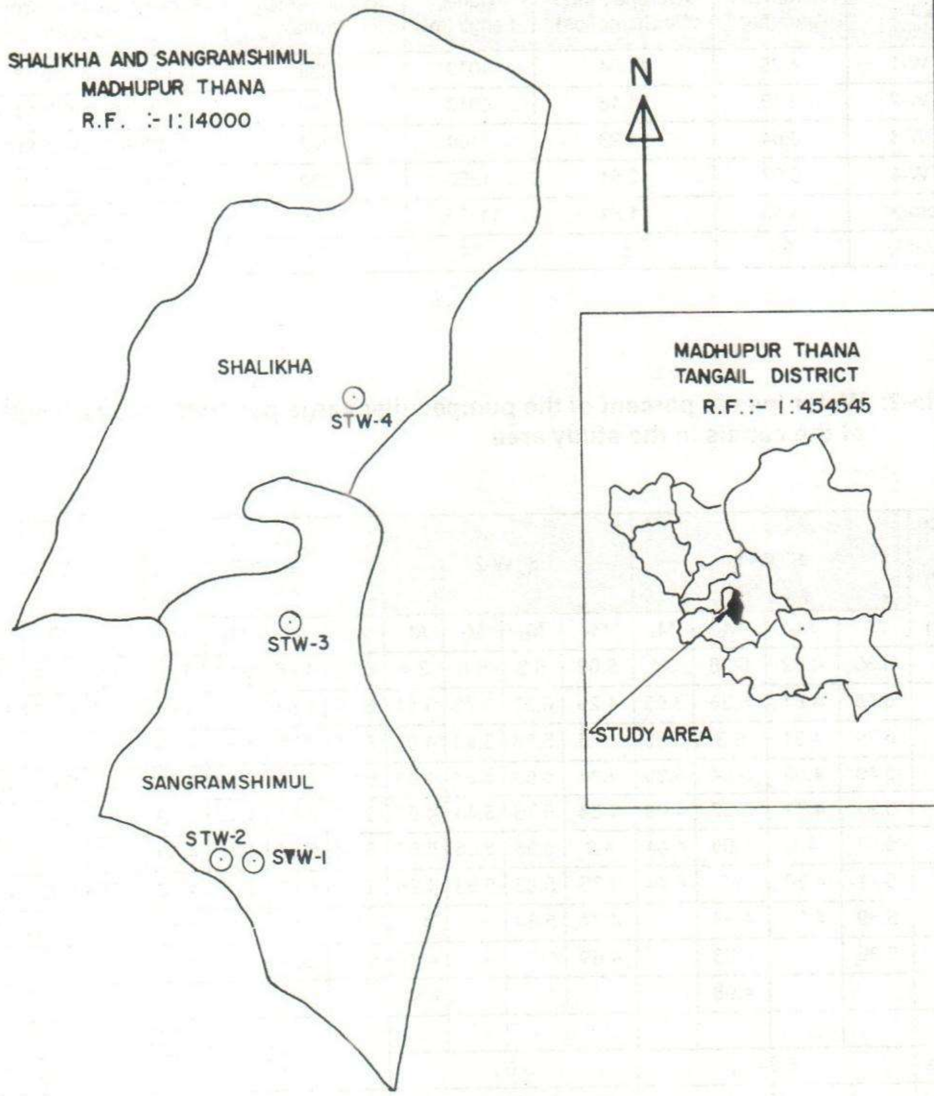

FIG. I : LOCATION OF THE STUDY AREA

(SOURCE : MOUZA MAP)

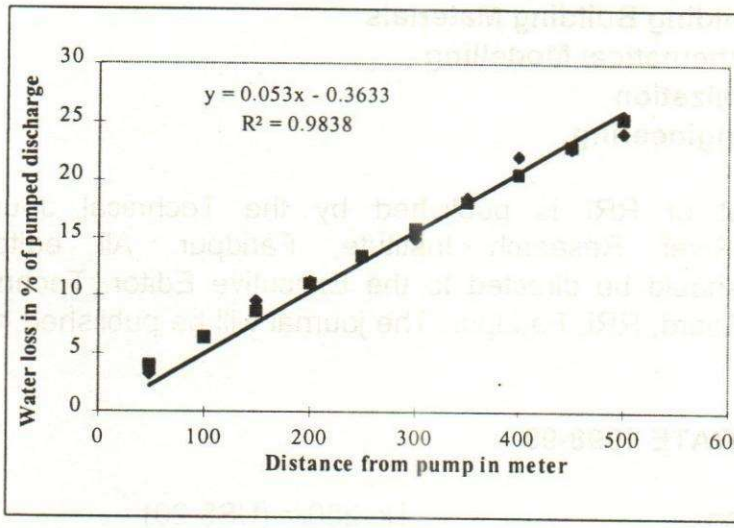

Figure 2: Water loss in the earthen canal of the study area

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