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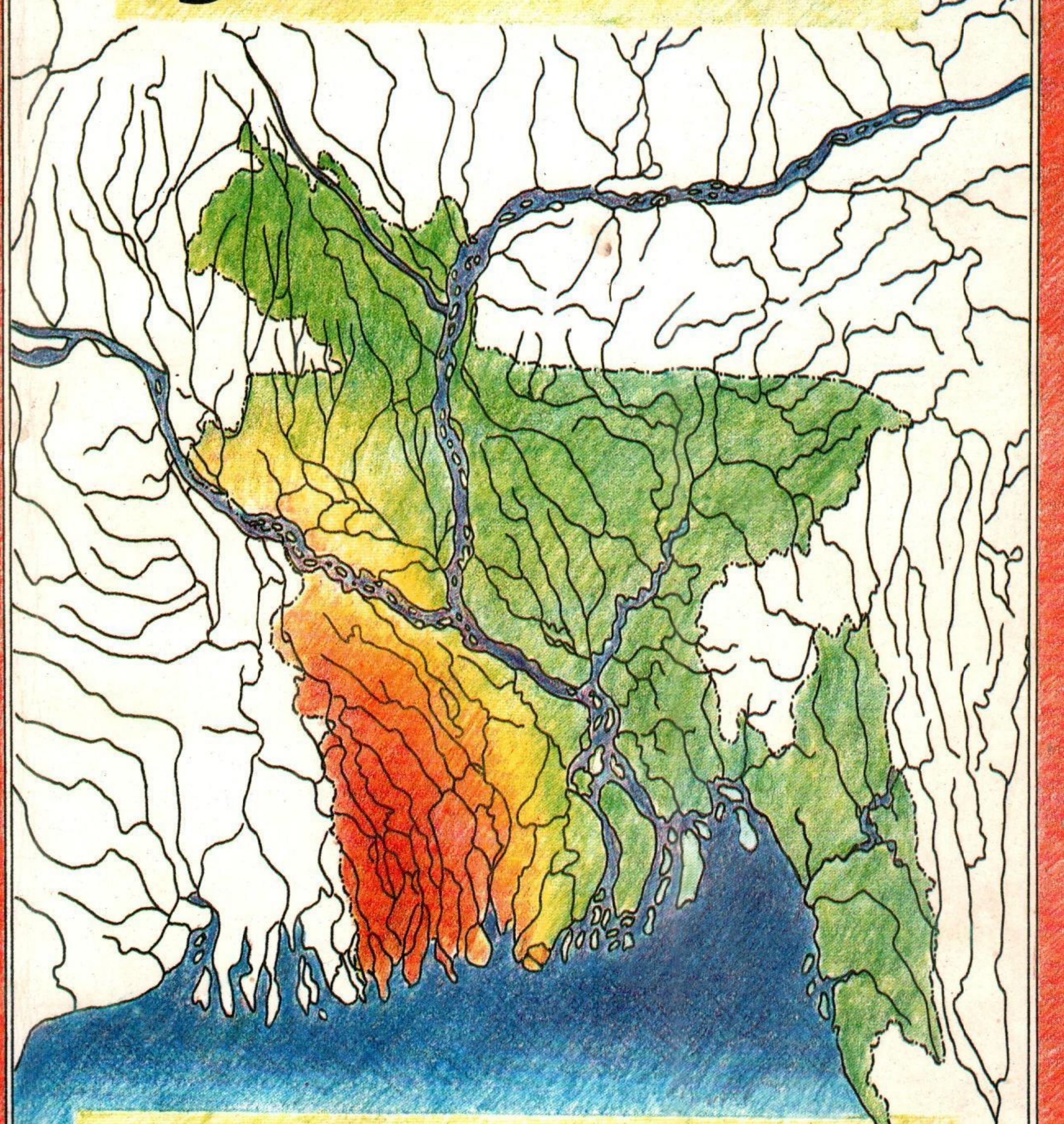

RIVER RESEARCH INSTITUTE

FARIDPUR

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TECHNICAL PAPERS

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ROLE OF THE RIVER RESEARCH INSTITUTE IN THE WATER RESOURCES DEVELOPMENT OF BANGLADESH

*Syed Anwar Yusuf**

Introduction

Bangladesh is located in the world's largest delta at the confluence of the three great rivers the Brahmaputra-Jamuna, the Ganges-Padma and the Meghna; the first two have the most dynamic morphology. Moreover,with hundreds of lesser rivers and rivulets (the total length about 22000 km) the river system forms a dense network. Exhibit-1 shows the regional location of Bangladesh with river catchment area and river network. Life in Bangladesh is predominantly riverine-agrobased. The average annual rainfall is 2500 mm, 80% of which is concentrated during monsoon (July-October). About 50% of the country lies within 12.5 m, with the southern part around 1.00 m above mean sea level. Exhibit-2 shows the general relief of Bangladesh with the area-elevation curve. Consequently, about 30% of the country is subjected to regular flooding every year causing extensive damage to life and property, maximum flooded area was 62% during 1988. Apart from river floods, coastal Bangladesh suffers severely from the tropical cyclonic surges during pre and post monsoon months generated in the Bay of Bengal which can reach heights of about 7.0 m as they approach the coast. The consequences are often disastrous and have costed hundreds of thousands of human lives. On the other extreme, during dry season scarcity of water in the rivers adversely affect agricultural, industrial and domestic use.

The geographical as well as the hydrogeological setting of Bangladesh demand comprehensiveness in the approach to plan the development and management of its water resources on a regional basis. This requires co-operation from the upstream co-riparian

* *Director General, River Research Institute, Faridpur.*

countries due to about 92% of the catchment area of the river system lying outside the country.

The management of waters has to be optimized on a seasonal basis with due consideration for the sometimes conflicting interests of flood control, bank-protection, irrigation, drainage and other agricultural and piscicultural requirements and industrial use. Thus, the planning and design of water management projects in Bangladesh require a delicate and difficult sharing. It requires both overall and detailed understanding of the complicated engineering and social problems which are interlinked with water. These interrelationships make many of the problems too complex for the traditional project-to-project type of planning. Management tools capable of integrating all the relevant processes on regional basis are required to address the problem.

River Research Institute

River Research Institute (RRI) is a national Institute of Bangladesh with the mandate of a Statutory Public Authority working under the Ministry of Irrigation, Water Development and Flood Control. It is to undertake multi-disciplinary programmes, tests, studies and researches in order to provide analytical and empirical tools for multisectoral planners and designers. On several instances it has also worked with upto such detail as is required for hydraulic design.

The World Bank, UNDP and DANIDA aided in the construction of infrastructures, procurement of equipment and development of technical know how.

RRI has three directorates and one Surface Water Modelling Centre to carry forward its National mandate.

Hydraulic Research

The Hydraulics Research Directorate conducts studies and researches in the following three discipline by physical scale models and suitable mathematical models.

River Hydraulics : With particular reference to flood control and drainage, bank protection and stabilization, sedimentation and navigation etc.

Estuarine and Coastal Hydraulics : With regard to tidal estuary, Coastal and harbour engineering, saline water intrusion and land reclamation etc.

Hydraulics of Structure and Irrigation : Concerning hydraulics of structure like barrage and dams, regulators and sluices, bridges and culverts and irrigation systems.

Following facilities have been created for conducting tests and researches on river engineering, estuarine and coastal hydraulics problems by means of physical modelling.

Open air model area :

The space available in this section is 32500 m^2 (250m x 130m) which is sufficient to run a total of 8 Nos (Exhibit-3) of physical models at a time. All the river engineering problems like Bank erosion, Flood control, Navigation etc. are dealt in this section. There are adequate facilities for pumping and recirculating of water for any type of scale model with maximum discharge being $2.00m^3$/s. In the monsoon, the open air model area are used under temporary sheds constructed over specific models.

Indoor model area :

For long term research and precise tests 2 nos model-sheds (100m x 30m each) having wave basin and river flume bed are available.

Model shed no.1

The 36m X 24m wave and current basin can be supplied with water through two pipes allowing for a maximum flow of 1.40 m^3/s having 0.8m water depth.

For the two 44m X 9m each river models it is possible either to supply any one of the river models by two pipes at a time allowing for a maximum flow of $1.4m^3/s$ or to supply simultaneously the two river models by one pipe each allowing for maximum flow of $0.7m^3/s$ in each facility with a water depth of 0.25m.

The 21m X 0.75m tilting glass flume is capable of flowing maximum $0.7m^3$ /s discharge with a water depth of 0.75m having maximum tilting slope of 1 : 70.

Model shed no.2
The following facilities are available in the Model Shed No.2.

- One river model with a test area of approximately 90m X 16m having a maximum flow capacity of $2.0m^3/s$.
- One river model with a test area of approximately 43m X 8m having a maximum discharge capacity of $1.4m^3/s$.
- One tidal river model with a test area of 43m X 8m having a maximum discharge capacity of $2.0m^3/s$.

Short description of the studies and researches conducted by physical modelling at RRI's present location since 1989 is given in Table-1.

Geotechnical Research
The researches in this field are mainly conducted concerning the problems connected to different structural materials through four research disciplines within the Geotechnical Research Directorate. Extensive laboratory facility (about 1000 sqm of floor space) has been established for conducting tests and researches. Short description of the jobs so far performed in this field at RRI's present location since 1989 is given in Table-2.

Soil Mechanics :
This discipline performs consolidation test, direct shear test, triaxial shear test, vane shear test, unconfined compression

test, compaction test, permeability test, CBR test, apart from simple identification tests to determine the characteristics of soils to be used as foundation and construction material.

Concrete Technology :

This discipline deals with quality control of concrete and concrete materials. It also runs trial mix design to attain a particular design strength with materials to be used in the construction.

Sediment Technology :

The main function of this discipline is to determine the quality and character of suspended sediment and bed materials transported by a flowing river and solve erosion-deposition problems.

Chemical Analysis :

This discipline is capable of determining 30 parameters of water (both inorganic and organic , specific and non-specific) such as pH, macro and micro ions, nutrients, BOD, COD etc.

Technology and Services Directorate:

Technology and Services (T&S) Directorate is responsible for rendering common and specialized services to the other two Directorates of RRI.

The services include :

Equipment and transport, Campus and workshop facilities, Library and publications, Computer facilities and Mathematical modelling facilities.

A small computer centre equipped with a modest resume of hardware and software facilities (Table-3) has been established for conducting the multitasking job under this directorate.

The T&S Directorate maintains the following important auxiliary installations also like the <u>Generator House</u> (3 standby generators with total capacity of 450 KWH) and the <u>Pump House</u>

(about 32 nos. 5 cusec. pumps) for supporting physical models and laboratory tests unhindered.

Surface Water Modelling Centre

From January 1993, Surface Water Modelling Centre (SWMC) has been transferred from the Water Resources Planning Organization (WARPO) to RRI as a unit located at Dhaka. SWMC has started to function since early 1980 under the DANIDA aided Phase-II of the Surface Water Simulation Modelling Programme (SWSMP). SWSMP owes its genesis to the National Water Plan Projects (NWPP) recognition that refined planning and management in the water sector requires a high level of analytical capability. The 1st phase of the SWSMP (1986-1988) was funded by UNDP and executed by World Bank. The SWSMP got added impetus following the two consecutive disastrous floods in 1987 and 1988 and so the phase-II was launched with a four year's ambitious programme (1989-1993) to establish 1-D mathematical models of the entire river systems in Bangladesh while creating and strengthening an indigenous capacity in mathematical modelling. Danish Hydraulic Institute (DHI) in association with the Bangladesh University of Engineering and Technology (BUET) has been the consultant.So far through these two phases, SWMC has developed -

a) A General Model
   Comprising the major river systems (Ganges,Brahmaputra-Jamuna, Padma and Meghna) with their major incountry tributaries and distributaries.
b) Five nos. Regional Model and several submodels comprising the regional river systems and subsystems respectively.

Through the extensive application of its models (tabular statement : Table-4) especially in several Flood Action Plan studies, SWSMP has established that the models are essentially very powerful tools for planning, design and management of water resources on an integrated multi-sectoral approach to flood control, drainage, irrigation, inland navigation, pisciculture, road and rail communication and environment.

Immediately after completion of the current 2nd phase, a third phase of three years duration is going to be commissioned from the 1st of January 1994 with the principal objective of consolidating the 1-D modelling technology and tying the loose ends of the phase-II with more emphasis on the morphology and flood forecasting aspects and introducing environmental and 2-D modelling. Most importantly, the third phase will be effectively directed to institutionalizing the SWSMP by undertaking multisectoral application works.

Conclusion

Through extensive research work, basic and job-oriented, RRI has been dedicated to develop local expertise as well as to adapt appropriate technology from the foreign expertise for providing the nation with the most suitable approach to the long drawn process of the most difficult water resources development objectives. It is expected that RRI with sufficient technological and financial support from national and international agencies will become the most covetted think tank of the country in the form of an international standard Hydraulic Institute.

Table -1 : Hydraulic research Conducted by RRI

Sl.No.	Title of study/research	Year	Model	Client
01.	Model study of Jamuna Bank Protection and River Training (AFPM) Pilot project (FAP 21/22) at Bahadhurabad Extended.	93-94	An undistorted model with length scale ratio 1:75 to study protection of Bank with revetment.	Consulting Consortium (France,Germany,the Netherlands) FAP-21/22.,(FACO).
02.	Model study of Jamuna Bank protection and River Training (AFPM) Pilot Project (FAP-21/22) at Kamarjani Extended.	93-94	An undistorted model with length ratio 1:75 to study river training and bank protection with spurs/groynes.	-Do-
03.	Model study on the silt Trap of Teesta Barrage Project.	93-94	An undistorted fixed bed model with length scale ratio 1:30 to study the proposed 'Silt Trap' of Teesta Barrage Project in respect of trapping the suspended sediment before entering into the main Canal.	Chief Engineer Project-4, (Teesta Barrage Project), BWDB., Rangpur.
04.	Model study of Jamuna Bank Protection and River Training (AFPM) Pilot Project (FAP 21/22) at Bahadurabad.	92-93	An undistorted model with length scale ratio 1:60 to study the protection of bank at Bahadurabad from erosion of the Brahmaputra/Jamuna with revetment.	Consulting Consortium FAP-21/22.,(FPCO).
05	Model study of Jamuna Bank Protection and River Training (AFPM) Pilot Project (FAP 21/22 at Kamarjani.	92-93	An undistorted scale model in ratio 1:75 to study river training and bank protection with spurs/groynes.	-Do-
06.	Scale Model study for the Protection of Gorai Rly Bridge.	92-93	An undistorted model with length scale ratio 1:80, to study the protection of Gorai Railway Bridge from erosion of Gorai River with permeable spurs.	Bangladesh Rly through S.E/G.K Project BWDB., Kushtia.

SL.No.	Title of the study/Research	Year	Model	Client
07.	Model study on Bank Protection of Sangu River at Dohazari.	92-93	An undistorted model with length scale ratio 1:75 to study the protection of the Dohazari R & H bridge and the bank of sangu River near Dohazari R & H bridge with partly submersible boulder spurs.	Chief Engineer, System Rehab., Project, BWDB, Dhaka.
08.	Meghna River Bank Protection Studies at - 1)Bhairab 2)Chandpur 3)Eklashpur.	91-92	1)A distorted model with horizontal scale ratio 1:60 to study the protection of right bank of Meghna River at Bhairab Bazar Railway Bridge. 2)An undistorted model with length scale ratio 1:150, to study the bank Protection at chandpur from erosion of the Meghna with revetment/groynes. 3)An undistorted model with length scale ratio 1:150, to study bank protection at Eklashpur.	M/S. Hasconing,Delft Hydraulics & BETS, FAP-9B, (FPCO).
09.	Brahmaputra River Training studies on Revetment stability and falling apron.	91-92	1)An undistorted model with length scale ratio 1:50, to study the stability of bank parallel revetment in different flow conditions and the launching pattern of different types of falling apron. 2)Some general study with different undistorted models on different types of groynes and a typical ferry ghat layout to study some local design parameters.	M/S. Sir William Harcrow & partners Ltd. (BWDB).

SL.No.	Title of the Study/Research	Year	Model	Client
10.	Brahmaputra River Training studies at 1) Kazipur 2)Fulchari ghat. 3)Sariakandi. 4)Sirajgonj.	90-91	1)A distorted model with horizontal scale ratio 1:200 and vertical scale ratio 1:120, to study the protection of river bank at Kazipur from erosion of Brahmaputra-Jamuna river. 2)A distorted model with horizontal scale ratio 1:200 and vertical scale ratio 1:120, to study the protection of river bank at Fulcharighat from the erosion of the same river. 3)A distorted model with horizontal scale ratio 1:125 and vertical scale ratio 1:60, to study the protection of river bank at Sariakandi. 4)A distorted model with horizontal scale ratio 1:300 and vertical scale ratio 1:120 to study the protection of River bank at Sirajgonj.	-Do-
11.	Composite model study for Teesta Barrage Project.	89-90	A distorted model with horizontal scale ratio 1:250 and vertical scale ratio 1:48, showing all important structures of Teesta Barrage and adjacent areas to study the river training works after the construction of the barrage, the layout of closuredam, lead-cut, tail-cut, flood by-pass etc.	C.E./Project-4 T.B.P Rangpur.

SL.No.	Title of the Study/Research	Year	Model	Client
12.	Model study for the protection of Bhanga Mawa Road on Arial Khan River.	89-90	A distorted model with horizontal scale ratio 1:200 and vertical scale ratio 1:50 to study the protection of Bhanga - Mawa Highway (part of Dhaka-Mongla Highway) from the erosion of Arial Khan River.	Madaripur RH Division
13.	Model study for Bank protection at Chapra and Badanpur from erosion of River Gorai.	89-90	A distorted model with horizontal scale ratio 1:200 and vertical scale ratio 1:36, to study the protection of River Bank at Chapra and Badanpur.	S.E./G.K. Project, BWDB.
14.	Model study for protection of Haturia-Nakalia Bazaar from erosion of River Hurasagar.	89-90	A distorted model with Horizontal scale ratio 1:200 and vertical scale ratio 1:50, to study the protection of Haturia-Nakalia Bazaar from erosion of River Hurasagar.	C.E. Project-2. BWDB Pabna.
15.	Model study for the protection of Bhairob Railway Bridge from erosion of Meghna River.	89-90	A distorted model with Horizontal scale ratio 1:300 and vertical scale ratio 1:60, to find out the suitable location/orientation of the groynes/spur/Bank Revetment for protection of Bhairob Railway Bridge from the right Bank erosion of the Surma-Meghna River.	C.E./Bangladesh Railway.

Table-2: Geotecnical Research Conducted By RRI.

Sl.No.	Title of Job	Year	NO. of sample tested	Type of test	Client
01.	Soil Mechanics testing and studies.	1992-93 1991-92 1990-91 1989-90	10,579 Nos. 8,769 Nos. 17,222 Nos. 21,194 Nos.	Traxial compression test, Direct shear test, Unconfined compression test, Consolidation test, Grain size analysis by sieve and hydrometer, Natural moisture content, Atterberg's limit test, Relative density (Gs), Unit weight, Density index(ID), Compaction test, C.B.R. test, Permeability test, Loss-on-Ignition test, Field quality control tests such as monitoring field moisture condition in-site density and assessing percent compaction of earth structures and relevant studies.	Different Divisions of Bangladesh Water Development Board and other engg. organisations in Bangladesh.
02.	Sediment studies	1992-93 1991-92 1990-91 1989-90	861 3044 2206 1053	Determination of suspended sediment concentration, Particle size distribution, Specific gravity, Viscosity etc.	-Do-
03.	Water quality	1992-93 1991-92 1991-90 1989-90	- 16 8- -	Determination of pH, Ec-value, Free CO_2, CO_3, HCO_3, SO_4, Cl, Ca, Mg, Fe, Na, SiO_2, Boron, Mn, Potassium, Salinity, total dissolved soid etc.	-Do-
04.	Test of Concrete and Concrete Materials	1992-93 1991-92 1990-91 1989-90	598 938 743 794	A)Cement Test, B)Sand Test, C)Coarse Aggregates Test, D)Brick Test, E)Concrete Test, F)Design of Trial Mix, G)Quality control of concrete in the Field.	

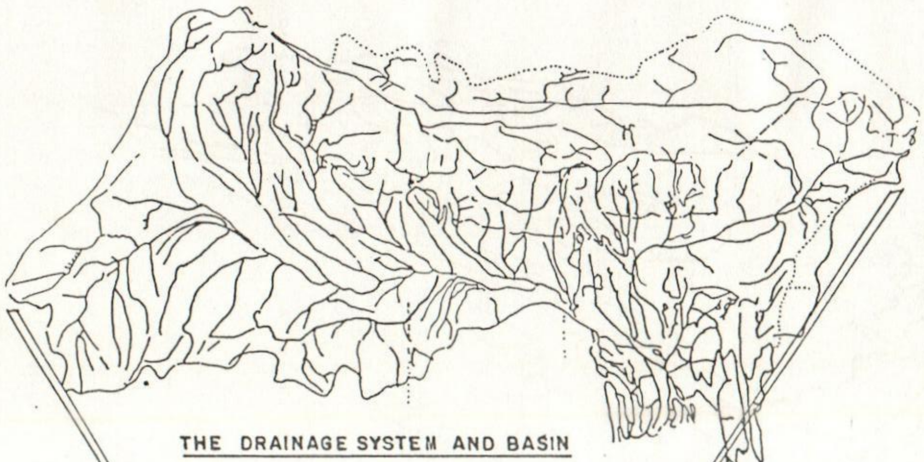
Table-3 : T&S Computer Facilities.

01. Hardware	a) ABC - 1 No. (125 MB, 5.25" & 3.5" High Density Drive) b) EPSON AX -5 Nos. (42 mb, 5.25" & 3.5" High Density Drive) c) IBM XT - 1 No. (23 MB, 5.25" Double Density Drive) d) TRITON - 1 No. (20 MB, 5.25" Double Density Drive)
02. Software	a) Languages Fortran 77 GW-Basic Turbo Basic Basica b) Packages & Utilities Wordperfect 5.1 Lotus 1-2-3 dBase III+ and dBase IV Quattropro 3.1 Harvard Graphics Grapher Mike 11 CPAV (Central Point Anti Virus) Nortons Utilities

Table -4 : Application of SWSMP Models

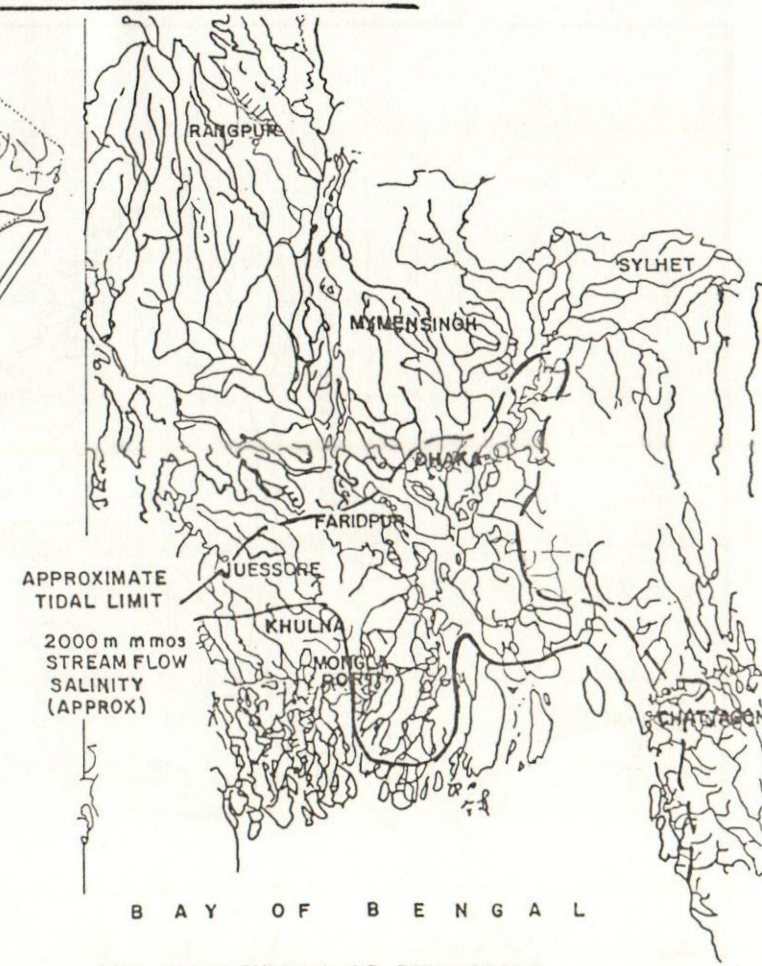
Sl.No .	Name of Application	Phase	Model	Client
01.	Mathematical Model runs on Gumti Phase-II Feasibility Study.	I & II	SERM	BWDB
02.	Flood Forecasting & Warning Systems Improvement.	I & II	GM	UNFP/WMO
03.	UNDP Flood Policy Study	I	GM	UNDP
04.	French Government Prefeasibility Study for Flood Control in Bangladesh.	I	SWRM	
05.	Coastal Embankment Rehabilitation Project-II.	I & II	SWRM	GOB
06.	Mathematical Model of the Drainage System of the Teesta Barrage Project.	II	NWRM	BWDB
07.	Dhaleshwari Mitigation & Land Use Master Plan (Part of the Jamuna Multipurpose Bridge Study).	II	GM & NCRM	
08.	Brahmaputra River Training Study.	II	GM, NWRM & JGP Model.	FAP-1
09.	North West Regional Study.	II	NWRM	FAP-2
10.	Jamalpur Priority Project.	II	NCRM	FAP-3.1
11.	South West Regional Study.	II	SWRM	FAP-4
12.	South East Regional Model	II	SERM	FAP-5.
13.	North East Regional Water Management Study.	II	NCRM	FAP-6.
14.	Flood Forecasting & Early Warning	II	GM	FAP-10
15.	Flood Modelling and Management.	II	GM & Regional Models by FAP studies.	FAP-25
16.	Tangail Compartmentalization Pilot Project.	II	NCRM	FAP-20.
17.	Environmental Study.	II	NCRM	FAP-16
18.	Greater Dhaka Food Protection	II	NCRM	FAP-8A
19.	Meghna Left Bank Protection	II	GM	FAP-9
20.	River Survey Project	II	GM	FAP-24.

REGIONAL HYDROLOGIC SETTING AND RIVER SYSTEM OF BANGLADESH

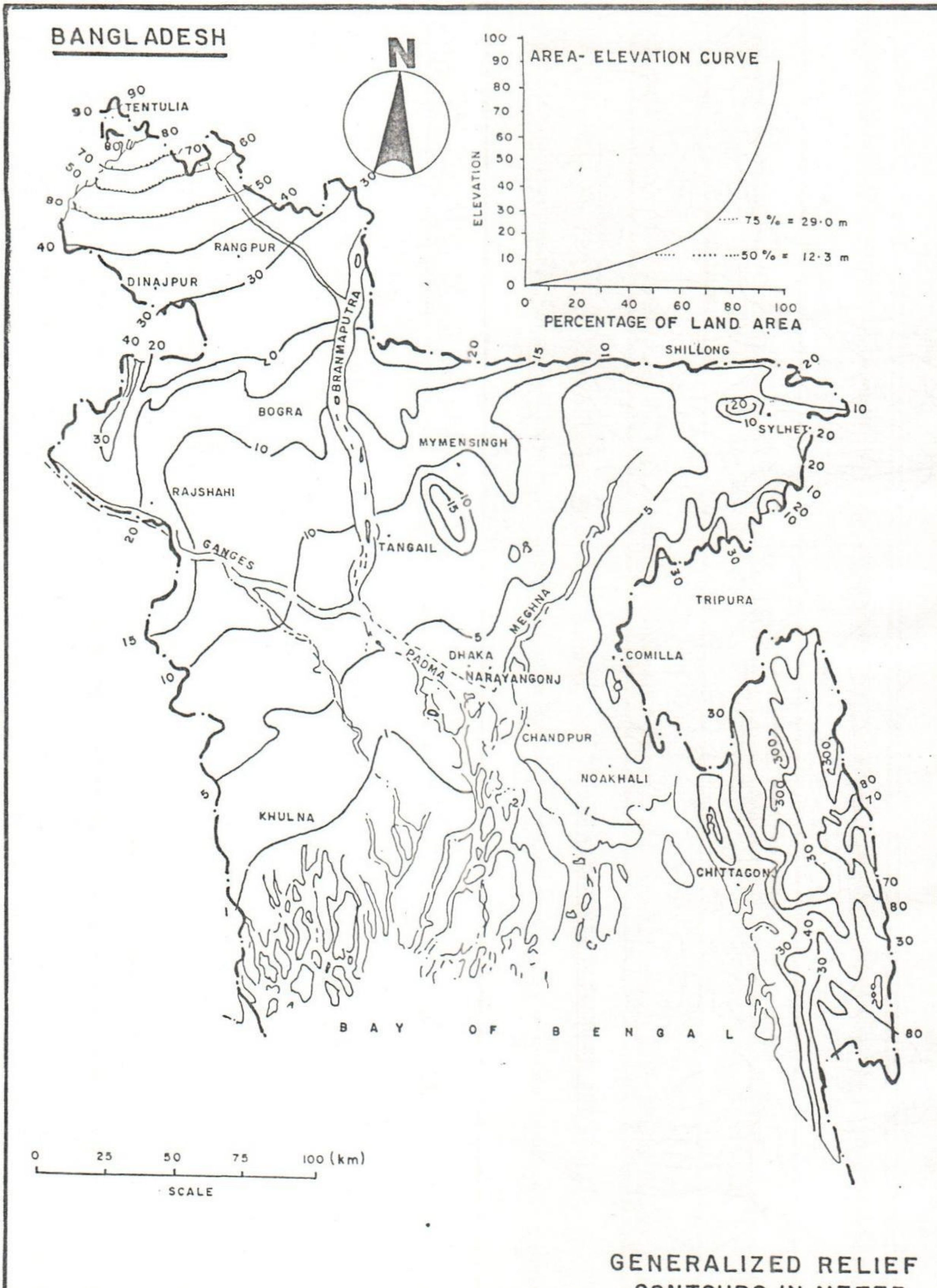


THE DRAINAGE SYSTEM AND BASIN

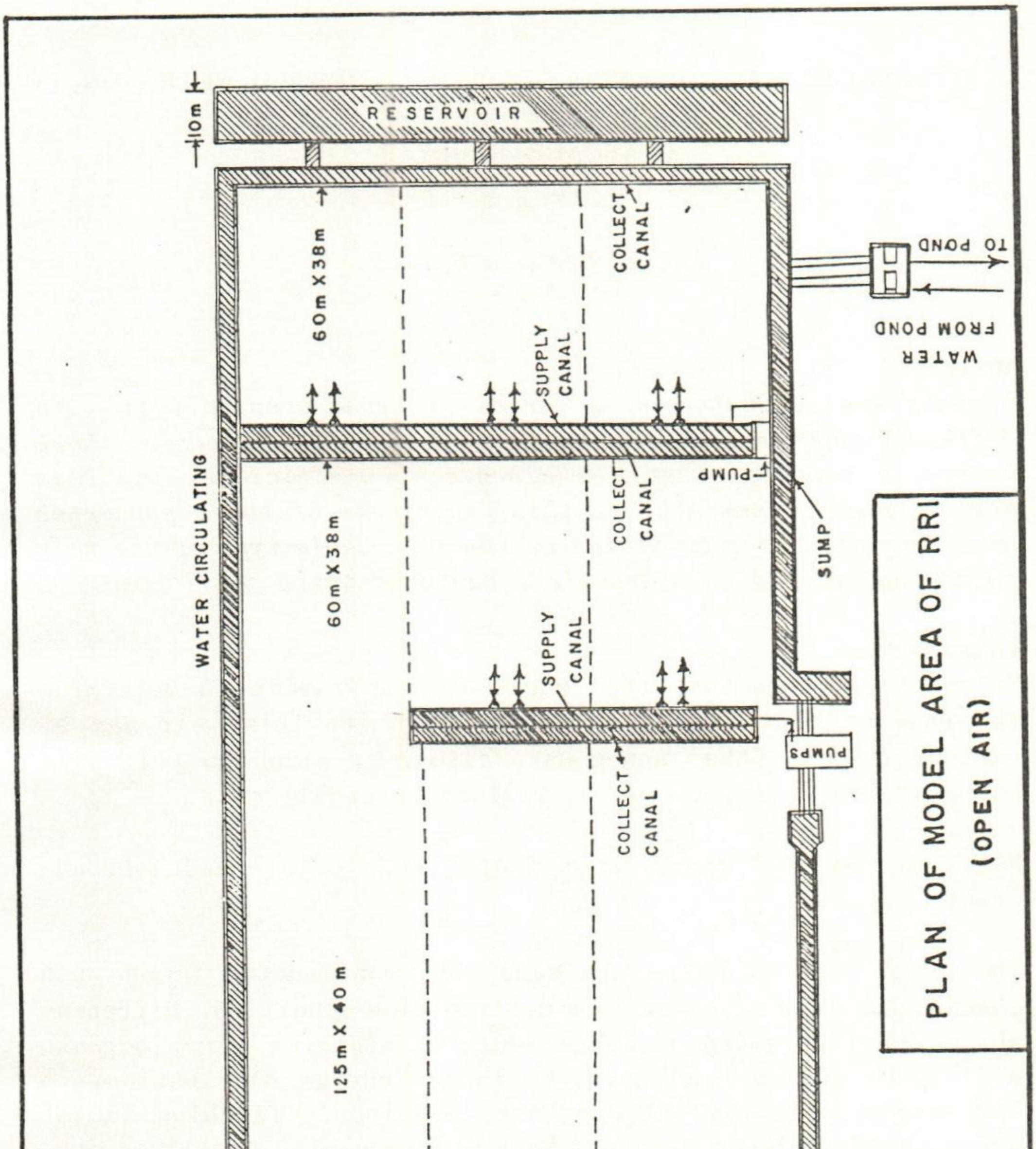
THE REGIONAL SETTING



THE RIVER SYSTEM OF BANGLADESH



GENERALIZED RELIEF
CONTOURS IN METER



PLAN OF MODEL AREA OF RRI
(OPEN AIR)

HYDRAULIC PERFORMANCE OF BROAD CRESTED WEIRS OF DIFFERENT SHAPES (A CASE STUDY)

*Shaikh Mostafa Hussain**

Abstract

A study was conducted on a series of broad-crested weir with different upstream and downstream slopes to compare their hydraulic behaviour against discharge co-efficient, upstream afflux, head losses etc. In this paper some of those parameter have been discussed to determine the acceptable type of the weir to be constructed in hydraulic structures where applicable.

Introduction

We generally know that the function of a weir is to determine the rate of flow of water passing over it. This weir may of various shapes. Other names have also been given to weir like wasteway, spillway, overwash, rollway, overfall etc.

Weirs may be of " sharp crest " of " thin edge " and " broad-crest ".

The broad crested weirs are generally constructed in an open channel for flow of water in a desired flow condition. Different shapes may be given to this weir to minimize or increase afflux, to reduce head loss, to induce energy dissipation, to increase co-efficient of discharge, to induce flushing out of accumulated sediment behind a barrage, regulator, spillway etc. Generally rectangular, trapezoidal and parabolic weirs are designed in such structures. A study was performed in RRI to see the hydraulic characteristics of different types of broad-crested weirs with a view to finding out the efficiency of

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* *Director/Chief Engineer, Hydraulic Research Directorate, River Research Institute, Faridpur.*

the under-sluice of the Teesta Barrage Project by Bangladesh Water Development Board.

Generally experimental results on weirs (broad-crested) with vertical or steeper upstream slopes are available in text books. But in the above-mentioned studies broad-crested weirs with very flatter slopes were tested in submerged flow condition.

In this paper the hydraulic performances of weirs with uncommon slopes have been discussed and the results of hydraulic characteristics have been tabulated in Table-I and Table-II for weirs of different configurations . This might be of interest to designers of the concerned field of works.

Brief description of the model

The model was constructed in a 36'-0" long and 3'-0" wide and 3'-0" deep steel-framed flume. One side of the flume was made of transparent plastic panels to facilitate visual observation in the flume from side. The model was made in an undistorted scale of 1 : 24. According to Froudian law of similitude the discharge scale was 1 : 2821 and the velocity ratio was 1 : 4.9.

To minimize friction on the steel-sheet side of the model, it was plastered with cement-mortar and then a very smooth cement finishing was given to it.

Two half-bays of the undersluice was reproduced in the model with one pier in the middle.

Water was introduced in the flume from a centrifugal pump and the discharge was regulated by a stop-valve. The discharge was measured on the downstream of the flume with the help of a 3'-0" long sharp-crested weir. This measuring sharp-crested weir was pre-calibrated in a calibration tank in the laboratory.

The ten different configurations of the broad-crested weir as tested in the model are shown in the Fig.2 in the following page.

It may be mentioned here that the weirs were tested against many other aspects, for example, downstream scour, pressure on the weir-glacis, sweep out condition of flow etc. but those parameters have not been elaborately discussed in this paper.

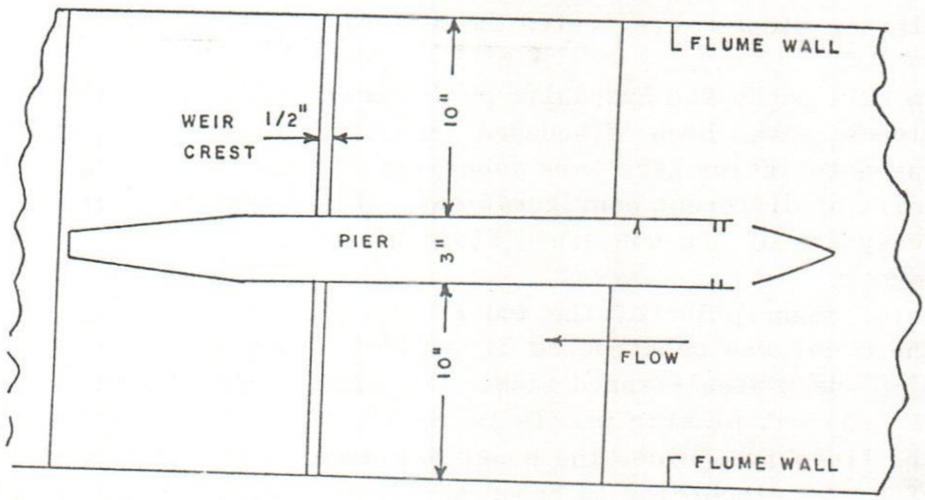


Fig.1: Plan of the broad crested weir in the flume

The upstream water level was measured at 1'-0" upstream of the crest of the weir and the downstream water level was measured at 2'-0" downstream of the weir crest. All the tests were done with submerged flow condition. Water passed over two weirs of 10" length (each) with a crest width of $^1/_2$ ".

A sharp-nosed pier of 3" width separated the two weirs. Ten different configurations of the weir were tested to find out the most efficient one vis-a-vis afflux, head loss, flushing out (of upstream sediment bed) and the downstream scour. However, in this paper, the hydraulic behavior of the weirs in submerged flow conditions regarding discharge-co-efficient and head loss etc. have been evaluated. The following table (Tables) shows these characteristics :

TABLE-I FLUME DISCHARGE, Q=2.20 cfs (for two weirs)

SL. NO.	Weir configuration U/S slope	Weir configuration D/S slope	d1 FT	d2 FT	d1/d2* 100 % submergence	H FT	h FT	Head loss H-h FT	$H^{3/2}$ FT	Discharge coff. C	Afflux d2-d1
1	2	3	4	5	6	7	8	9	10	11	12
1	Parabolic		0.571	0.588	97	0.640	0.590	0.040		2.60	0.017
	(Haigh		0.612	0.621	98	0.670	0.640	0.030	0.548	2.42	0.00
2	1:5	1:3	0.571	0.595	96	0.643	0.618	0.025	0.516	2.56	0.024
	"	"	0.612	0.629	97	0.672	0.639	0.033	0.551	2.40	0.017
3	1:5	1:4	0.571	0.600	95	0.648	0.602	0.046	0.522	2.54	0.029
	"	"	0.612	0.627	97	0.670	0.643	0.027	0.548	2.42	0.015
4	1:4	1:4	0.571	0.590	97	0.637	0.603	0.034	0.508	2.60	0.019
	"	"	0.612	0.622	98	0.665	0.640	0.025	0.543	2.44	0.010
5	1:4	1:3	0.671	0.600	95	0.649	0.604	0.045	0.523	2.53	0.029
	"	"	0.612	0.634	96	0.675	0.637	0.038	0.555	2.38	0.022
6	1:1	1:2	0.571	0614	93	0.658	0.604	0.054	0.534	2.48	0.043
	"	"	0.612	0.646	95	0.687	0.641	0.046	0.569	2.33	0.034
7	1:1	1:3	0.571	0.596	96	0.643	0.604	0.039	0.517	2.56	0.025
	"	"	0.612	0.629	97	0.673	0.641	0.032	0.552	2.40	0.017
8	1:5	1:2	0.571	0.617	93	0.661	0.604	0.057	0.537	2.46	0.046
	"	"	0.612	0.646	94	0.687	0.642	0.045	0.569	2.33	0.034
9	1:6	1:2	0.571	0.617	93	0.661	0.604	0.057	0.537	2.46	0.046
	"	"	0.617	0.646	94	0.687	0.641	0.043	0.569	2.33	0.039
10	1:6	1:3	0.571	0.588	67	0.635	0.604	0.031	0.506	2.62	0.017
	"	"	0.612	0.625	98	0.669	0.642	0.027	0.547	2.42	0.013

Explanations:-

H. Total head on the uptream of the weir = $d2 + v_2^2/2g$

h. Total head on the downstream of weir = $d1 + v_1^2/2g$

Head loss = H-h

Afflux=Difference of water levels on the upstream and the downstream side of the weir.

C. Cofficient of discharge of broad crested weir = $Q/LH^{3/2}$

TABLE..II FLUME DISCHARGE= 4.25 CFS

Sl. No.	Weir configuration U/S slope	Weir configuration D/S slope	d_1 FT	d_2 FT	d_1/d_2 *100 % Submergence	H FT	h FT	Head loss H-h FT	$H^{3/2}$ FT	Discharge coeff. C	Afflux d_2-d_1
1	2	3	4	5	6	7	8	9	10	11	12
1	Parabolic		0.696	0.771	89	0.902	0.721	0.181	0.835	2.99	0.083
	(Haigh zinn profile).		0.717	0.788	91	0.898	0.779	0.119	0.841	3.00	0.071
2	1:5	1:3	0.696	0.771	90	0.887	0.766	0.122	0.857	3.06	0.075
	"	"	0.717	0.775	92	0.892	0.781	0.110	0.851	3.04	0.058
3	1:5	1:4	0.696	0.778	89	0.891	0.767	0.124	0.841	3.04	0.082
	"	"	0.717	0.785	91	0.896	0.770	0.126	0.848	3.02	0.032
4	1;4	1;4	6.696	0.773	90	0.891	0.765	0.151	0.891	3.04	0.077
	"	"	0.717	0.776	92	0.889	0.775	0.115	0.889	3.05	0.059
5	1:4	1:3	0.696	0.761	91	0.978	0.755	0.123	0.823	3.11	0.065
	"	"	0.717	0.766	93	0.882	0.768	0.114	0.828	3.09	0.049
6	1:1	1:3	0.696	0.813	86	0.918	0.759	0.159	0.821	2.91	0.117
	"	"	0.717	0.817	88	0.924	0.773	0.151	0.824	2.88	0.100
7	1:1	1:3	0.696	0.788	88	0.900	0.763	0.137	0.874	2.99	0.092
	"	"	0.717	0.792	91	0.902	0.770	0.432	0.888	2.98	0.075
8	1:5	1:2	0.696	0.808	86	0.914	0.763	0.152	0.872	2.93	0.112
	"	"	0.717	0.817	88	0.924	0.773	0.151	0.881	2.88	0.100
9	1:6	1:2	0.717	0.814	88	0.919	0.773	0.146	0.888	2.90	0.097
10	1:6	1:3	0.696	0.758	92	0.877	0.775	0.102	0.854	3.12	0.062
	"	"	0.717	0.763	94	0.879	0.784	0.095	0.857	3.11	0.046

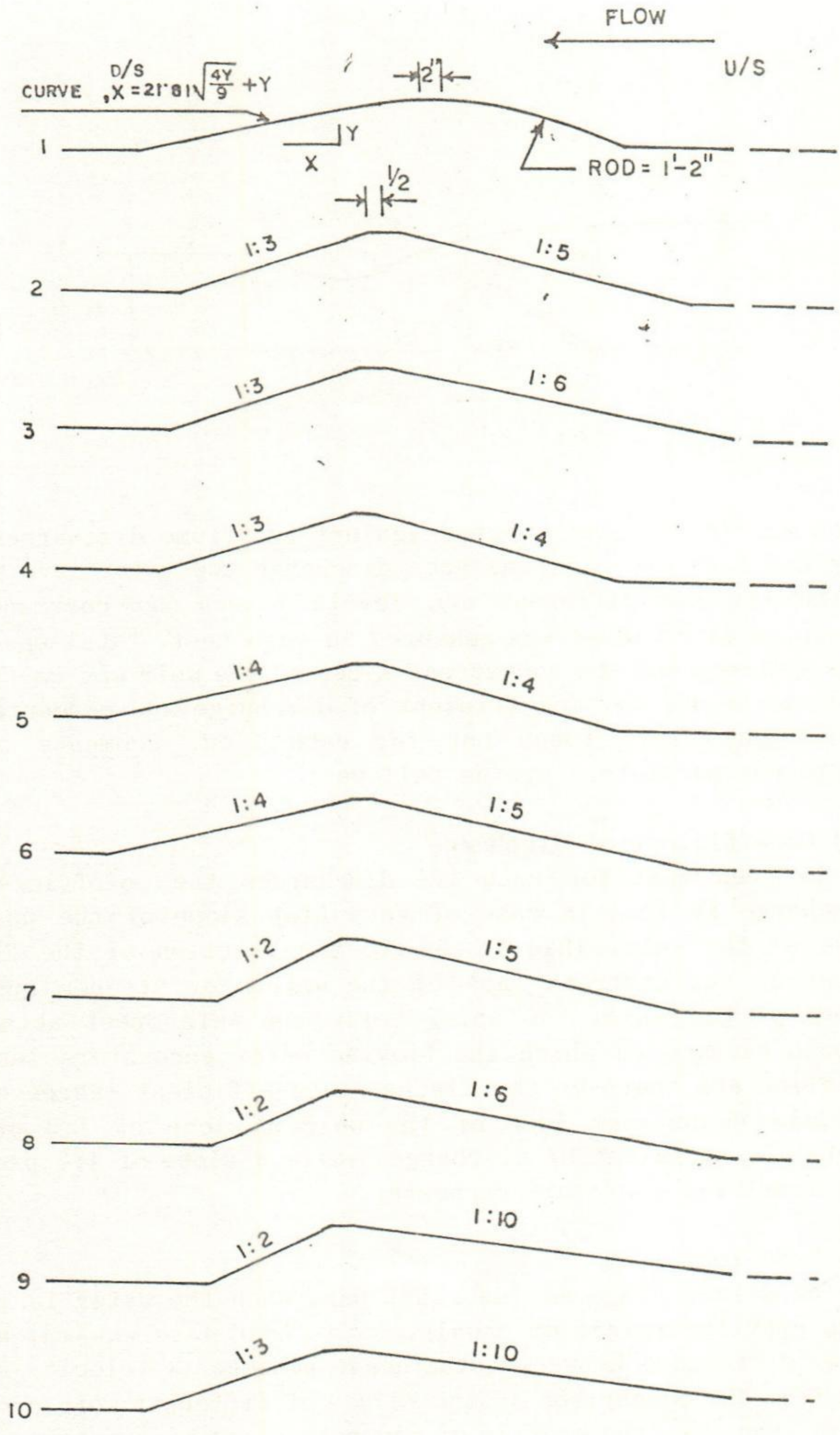
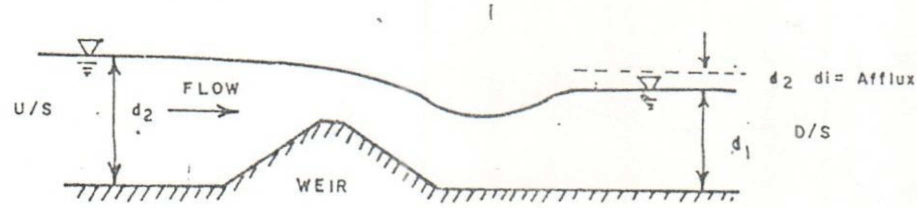


FIG.2 PROFILE OF THE BROAD CRESTED WEIR IN THE FLUME



Each weir profile was tested against two flume discharges 2.20 cfs and 4.25 cfs. And for each discharge two water levels were fixed (at two different d/s levels) and the corresponding upstream water level was measured in each test. Total energy on the upstream and the downstream sides of the weir was calculated and the head loss, co-efficient of discharge and percentage of submergence were found out for each flow. Comments on the different parameters are as follows :

(1) Co-efficient of discharge

It is seen that for both the discharges the co-efficient of discharge is less in case of very flat slope of the upstream face of the weir. This is due to the friction of the flowing water on the upstream face of the weir. For steeper upstream slope of the weir, the water below the weir crest acts as a smooth surface on which the flowing water encounters less bed friction and there-by the discharge co-efficient increases. As for the downstream face of the weir a slope of 1:3 produce higher co-efficient of discharge, while a slope of 1:2 produces the least value of this parameter.

(2) Head loss

The head loss plays an important part when the water is passed into gravity-irrigation canals. Total head loss was calculated (i.e. difference between total head: pressure + velocity head) and from the comparison of the values of different weir profiles it is seen that the profile of moderate slope (i.e. upstream 1:6

and downstream 1:3) gave the least head losses for both the higher and lower discharges. From the results it appears that the rate of discharge has a considerable influence on the total head on different types of weirs.

(3) Afflux

The word afflux means the difference between the upstream and the downstream water levels. This aspect of hydraulics is also very important due to the fact that when flowing water is arrested for irrigation, power generation etc., inundation on upstream side of the weir may occur due to excess afflux. From the results of this model test it appears that of all the tested weir profiles, the weir of moderate slopes of u/s 1:6 and 1:3 produces the least afflux.

Concluding Comments

From the comparison of the hydraulic characteristics of the broad crested weirs with different upstream and downstream slopes and the parabolic shape it appears that for the submerged conditions of the flows the moderate slope of upstream face 1:6 and downstream face 1:3 of a broad-crested weir the results of discharge co-efficient, amount of afflux and head loss are better against acceptability than those of others. From this conclusion more detailed tests were performed on the parabolic profile and the trapezoidal profile with upstream slope 1:6 and downstream 1:3. One of the tests was desiltation of the upstream pocket of the undersluice and in this test the trapezoidal profile was found to be taking less time for desiltation of the pocket.

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DEGRADATION OF SOIL AND ENVIRONMENTAL POLLUTION

*Engr. Nazrul Islam**

Introduction

The ever increasing Technology & pollution are pressurising land,water,air and environment. It is largely the human activities which contribute to the problem of soil degradation and environmental pollution. 25 million tonnes of productive topsoil is lost every year to desertification and land degradation leading to serious implication for agriculture. Modern Technology improves quality of life while impairing the quality of environment. Fertilizers help increase food production. Pesticides prevent diseases and crop losses and also provide some of the major safeguards for plants,human and animal health. Even though residues of many of these materials have adverse effect on soils and environment. Great benefit is derived from nuclear energy. Yet there is a great concern about the disposal of radioactive wastes into the environment which degrade and cause great hazards to the environment. The present discussion deals with the environmental pollution and soil degradation resulting from the use of fertilizer, pesticides and groundwater irrigation. Soil degradation and environmental pollution are now of major concern for a sustained development relating to health and agriculture,so as to prevent the damages due to environmental degradation. A focus on its various detrimental processes,causes and consequences is attempted.

Soil degradation and pollution

Soil degradation refers to a decline in the soil productivity through adverse changes in the nutrient status and soil organic

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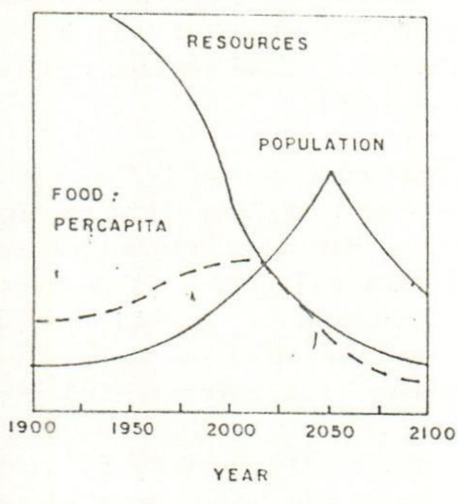
matter, structural attributes,and concentration of electrolytes and toxic chemicals (UNEP 1982).
Prevention of soil degradation is one of the greatest challenges facing mankind. Soil degradation is a major concern for at least two reasons; first it undermines productive capacity of ecosystem. Second,it affects global climate through alterations in water and energy balances and disruptions in cycles of carbon,nitrogen,sulphur and other elements. Its impacts on agricultural productivity and environment leads to political and social instability, enhances rate of deforestation, accelerates run off and soil erosion and pollution of natural water systems etc.

Causes of soil degradation and pollution

Main causes of soil degradation and pollution stem from indiscriminate human interferences in the natural ecological balance, from abuse and mismanagement of the soil and water resources and from farming land beyond its capacity to feed population and their animals and to make a profit from the land. Number of human being keeps on increasing while the natural resources remain finite. Thus per capita resources is gradually shrinking. This phenomena is represented by the figure below. Moreover short-sighted policies lead to degradation of the agricultural resource base.



Impact of chemicals and pollution

Chemicals,fertilizers and pesticides have played an important role in increasing agricultural production, but clear warnings have been given against over reliance on them. The run off of nitrogen and phosphates from excess use of fertilizers pollute water resources and as such damages aquatic and other lives. Excess use of chemical fertilizers may also cause nutritional imbalances in soil. Use of pesticides enhances productivity,but its over use threatens the health of humans, fishes and other lives. It also disturbs soil ecological balance. Long-term exposure to pesticides and chemical residues in food, water and even in the air is hazardous.

Physical degradation

Physical degradation refers to the deterioration of the physical property of soil. This includes erosion,sedimentation, compaction and hardsetting, crusting and laterization.

Soil erosion is one of the major causes of land degradation. This reduces crop production and increases water pollution. Tremendous amount of surface soil is removed by surface run off and to a lesser extent by blowing wind. Erosion is directly associated with the loss of plant nutrients and these plant nutrients are related to the yield. The organic matter and sediments load in water bodies destroys reservoirs and makes the treatment of water for domestic use more costly. Suspended solids from land erosion cause serious problems affecting fishes in the rivers (FAO 1986).

Mechanization of agriculture has led to a greater densification of soils due to movement of farm vehicles. The susceptibility of soils to compaction has also risen due to extensive use of inorganic fertilizers and over tillage. Intense rains further accelerate soil compaction. Groundwater containing oxides, sulphates & carbonates of Iron and calcium when used for irrigation may reduce soil porosity and make the soil compact depending on soil P^{H}.

Hardsetting soil causes the surface to harden sufficiently to prevent seedling emergence (Mullins et al,1987).

The formation of a thin surface called crusting causes most rainfall to be lost as run-off resulting in unsatisfactory environment for growth of crop.

Laterization means plinthite and related forms in degradation. Iron or aluminum rich layers rarely form natural soil, evolutionary process that removes silica and accumulate sesquioxides within the soil profile and protected by vegetation cover. These soil layers are often soft. The soil harden through iron-aluminum cementation into extensive rock-like sheets. These phenomena reduce the quality of the soil for agriculture. Their presence reduces the effective soil volume available for moisture and nutrient storage. They create obstacles for root penetration.

Biological degradation

Biological degradation is the reduction of organic matter in soil and the decreased activity of soil organism due to the processes resulting in environmental conditions deleterious to soil organisms. Soil structure degrades easily if biological activity falls bellow a certain minimum level. Reduction in organic fraction of the soil,which serves as the energy (carbon) source for soil fauna, results in decreased activity ,diversity and population of that fauna. Indiscriminate and excessive use of pesticides decreases faunal activity.

Soil devoid of macrofauna and with low level of organic matter undergoes a rapid deterioration in soil structure (Hamblin, 1985).

Chemical degradation

Chemical degradation is defined as the accumulated negative impact of chemicals and chemical process on those properties that regulate the life processes in the soil. Degradation of soil occur when the total nutrient reserve are inadequate for bio-mass production or when the rate at which the nutrients are mobilized is less than bio-mass demand.

Excessive soil acidity manifested in low P^H,low levels of exchangeable Ca,Mg,and K and toxic levels of Al is detrimental to plant growth and to microbiological process in the soil. Soil P^H values below 4 are extremely limiting to plant growth (Fay,1984) and are unfavorable for the growth of many bacteria and actinomycetes (Alexander, 1977).

Salinization and alkalization

The accumulation of salts and alkalis in soil threaten productive agricultural lands. Salt is one of the factor for pollution of soil. This type of pollution can be caused by several means and the main way is the inundation of soils by saline water from sea which brings salts and pollute coastal regions out side coastal embankment project in Bangladesh. Groundwater containing high amount of soluble neutral salts when used in irrigation may also impair soil quality and develop salinity. Again during dry season salt is brought to the surface by capillary rise of ground water which evaporates leaving the salts behind. Salinization is also caused by faulty irrigation by groundwater containing high amount of soluble salts.

Environmental pollution

Environmental pollution is the unfavorable alteration of our surroundings,wholly or largely as a by product of man's actions through direct and indirect effects of changes in energy patterns,radiation levels,chemical and physical constitution and abundance of organisms. Anthropogenic activities such as indiscriminate use of fertilizer and pesticides,disposal of industrial and domestic effluent and wastes, sludge, nuclear wastes etc. pollute soils, water and air. Such effluent and wastes containing a number of pathogens, organic and inorganic substances which are considered potential environmental pollutants. In urban areas a severe pollution is caused by detergents in wastes water in air emission (with heavy metals) from vehicles.

About 90 percent of cultivable land is used for the production of food crops. With the introduction of HYV resulted in the increased use of agro chemicals and irrigation inputs to harvest higher yields. Unfortunately the residues of these inputs in soil have great impact on the environment (Ahmed 1985). The increased use of fertilizers has raised nutrient pollution to soil and surface & ground waters. Excessive use of nitrogen fertilizer has profitably increased agricultural production but it has also an adverse impact on water quality in many areas.

Nitrate pollution in water and food are of great concern to human and livestock often causing methemoglobinemia or equivalent in case of livestock. Over supply phosphatic fertilizers to soil often causes eutrophication of natural water systems creating problem of algal boom.
The control of diseases and pests by pesticides have played a major role in improving yields and reducing post harvest losses for most crops. An increased use of biocides may kill predator species as well as pests and at the same time facilitate the developments of resistant strains of pests.

The pollution of soil by industry is characterized by the tremendous variety of organic and inorganic chemicals, which are potential pollutants. Inorganic pollution from industrial sources contain a number of elements such as mercury,cadmium,lead,arsenic,nickel,copper,zinc,molybdenum,manganese,fluoride and boron. To a greater or lesser degree,all of these elements are toxic to human and plants. Inorganic compounds from industrial sources can also be salts,acids and bases. Tannery industries use chromium salts which may be toxic even in low dose. Acids and bases from the industries have been introduced into soil (FAO 1979). There are also many kinds of chemicals that may enter the soil.

Conclusion

Use of chemical fertilizers, pesticides, industrial wastes and effluents from municipal areas contain heavy metals, toxic substances and in some cases high amount of ammonium and nitrate nitrogen. These effluents may also contain pathogenic microorganisms. They pollute natural water systems, ground water, sediments and land and thereby create serious environmental hazards. They endanger human health and aquatic lives. Both chemical and physical analysis of sediment and water samples are to be carried out to estimate the extent of pollution of the natural water systems, sediments and soil.

Full impact of use of fertilizer, pesticides and groundwater irrigation on the environment be fully considered and understood. Land degradation directly affects agriculture by diminishing crop yields,food supplies and water resources. Environmental degradation as a whole often seriously affect other sector of the economy. Thus measures are necessary to arrest significant direct and indirect adverse effect on land, ecology and environment.

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LABORATORY STUDIES OF BEDROUGHNESS AND SEDIMENT TRANSPORT-APPLICATION THEREOF

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Abstract

The paper within the framework of basic research programme dealing with flume studies of sediment transport, attention is focussed upon bed forms and their importance for an improved understanding of sediment transport and hydraulic roughness. Some aspects of the related investigations conducted in a four feet wide laboratory sand flume and thereby salient features of the experimental results are discussed.

Experiment showed interesting results of bed forms of fine and coarse sands as well as variations of friction factors and sediment transport. The friction factors of movable bed of sand varies considerably with hydraulic parameter and there was marked changes in the configuration of bed form. The sand dune formed across the channel are not uniform but weaved back and forth in the channel. It was further observed that the bed friction factor of fine sand decreases with the increase of concentration of suspended material. The data are also analyzed in terms of bar resistance and flow intensity of shear and the results follow closely the Einstein-Barbarossa bar resistance curve for natural channels. The experimental data plotted in the form of Einstein bed load function and the results indicated that flume data are comparable to Einstein curve.

The friction losses and sediment transport in an alluvial streams vary over appreciable ranges. The friction losses of streams is not constant but varies with bed configuration, depth and velocity of flow, fluid properties, bed material size and amount of sediment load.

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The law of transportation of sediment is intimately linked with the roughness problem. The depth, velocity and shear for any discharge are determined by the roughness which itself depends upon channel geometry, bed material and sediment load. On the other hand, sediment load or the concentration of sediment load depends upon flow conditions. The problem of sediment transportation and resistance to flow is closely linked together and is, therefore, studied together.

For more than century many experiments, especially in flumes, were carried out in order to investigate the various factors influencing sediment transport and hydraulic roughness. Sediment transport for bed can be expressed as a relation between the sediment transport on the one hand and the shear stress acting upon the river bed on the other hand. A part of the total shear is transferred to the river bed via the form drag of the bed forms, the remaining part of the shear stress is responsible for the sediment transport. Prof. H. A. Einstein enunciated the basis of bed load theory which in his own words are "The movement of bed materials is a complex function of flow duration, sediment supply and channel characteristics".

The suspended load of a natural stream includes both bed material load and wash load. Consequently, suspended sediment transportation theory can deal with the bed material load as pointed out by Chien. The analysis of suspended load can deal either directly with the total sediment transportation rate or with parameters in the equation for the concentration profile. If this concentration distribution is denoted by C(y) and the velocity distribution by u(y), then the sediment discharge per unit width for two dimensional flow is given by the integral

$$q_s = \int_{y}^{d} c(y)\, u(y)\, dy$$

where, y is the distance up from bed.

The hydraulics of uniform flow include basically the description of the velocity distributions and of the frictional loss for turbulent flow. Velocity distribution in open channel flow over a sediment bed is best described by Von Karman's logarithmic similarity law with the constants as proposed by Keulegan and is given by

$$U_y = \frac{2.3}{k} U_* \log_{10}(30.2 \frac{yx}{k_s})$$

$$= 5.75 U_* \log_{10}(30.2 \frac{yx}{k_s}) \text{------------------} (1)$$

where,

U_y = average point velocity at a distant y from the bed
U_* = Shear velocity = $\sqrt{(grs)}$
Y = distance up from the bed
X = a corrective parameter
K_s = equivalent roughness of the bed.

The distribution of sediment concentration in a vertical is given by,

$$C_y = C_a (\frac{d-y}{y} \cdot \frac{a}{d-a})^z \text{-------} (2)$$

c_a = point concentration at a reference level
d = total depth
z = exponent for suspended load

$$= \frac{W}{KU_*} = \frac{W}{\beta K U_*'} = \frac{W}{0.4 U_*'}$$

ß = ratio of turbulent diffusion coefficient for sediment to that for momentum
U_*' = shear velocity due to grain resistance only
= $\sqrt{(gr_b's)}$
W = settling or fall velocity of sediment particle,

Combining equation (1) and (2), sediment discharge per unit width is given by

$$q_s = \int_y^d C_a \left(\frac{d-y}{y} \cdot \frac{a}{d-a}\right)^z X 5.75 U_* \log_{10}\left(30.2 \frac{yX}{K_s}\right) dy \ldots\ldots (3)$$

Bed load function may be defined as $\phi = f(\psi)$

The quantity ϕ (intensity of bed load transport) involves the rate of movement, the particle size and immersed weight and a dimensionless function F of the fall velocity :

$$\phi = \frac{q_s \rho_s}{\sqrt{g(\rho_s - 1)} \cdot F D_{50}^{3/2}}$$

where,

$$F = \sqrt{\frac{2}{3} + \frac{36\nu^2}{g D_{50}(\rho_s - 1)}} - \sqrt{\frac{36\nu^2}{g D_{50}(\rho_s - 1)}}$$

The quantity ψ (Intensity of shear on bed particles) involves essentially the same particle characteristics and the bed shear $\tau = \gamma ds$

$$\psi = \frac{\gamma(\rho_s - 1) D_{50}}{\tau} = \frac{(\rho_s - 1) D_{50}}{ds}$$

q_s = sediment transport rate per unit width

ρ_s = density of sediment particles

D_{50} = particle diameter for which 50% by weight is finer

γ = sp. weight of fluid

d = Mean depth
s = Slope
τ = Shear stress of channel.

Studies in 4 ft wide laboratory flume of RRI

One of the main objectives of flume studies in sedimentation is to develop a theory which will enable to predict the sediment load of a stream as a function of water discharge and other pertinent variables.

Sediment transportation is a complex problem which studied in 4 ft wide laboratory flume as a basic investigation as well as an applied problem in scouring sluices of diversion structures. The problem of transport of bed materials in the alluvial channels bifurcated from the principal rivers-the Ganges, the Brahmaputra, and the Meghna in Bangladesh has got much attention due to consequence of heavy maintenance cost. The annual cycle of these principal rivers are a long period of low flow approximately 7 months and much shorter period of very high discharge. All these rivers have unstable bed and unstable channel geometry. In view of the fact that large number of alluvial rivers flow past in Bangladesh with bed material of different grade of sand ranging from 0.1 mm to 0.47 mm. mean diameter, it was considered desirable to study the bed roughness, sediment transport and nature of bed movement of different grade of sand in laboratory flume. A flume of 84 ft long, 4 ft wide and 3 ft deep was used in the experiment.

Each series of tests consist of varying depth and discharge. All experiments were made in uniform flow. Uniform flow was established and depth, slope, temp., sediment concentration, bed forms and mean velocity were determined by the procedure described by Prof. Vanoni & Brooks published in "Laboratory studies of the roughness and suspended load of alluvial streams in December, 1957, California Institute of Technology Sedimentation Laboratory".

Tests were carried out in 4' wide flume with

1. Fixed bed-clear water
2. Movable bed with fine and coarse sand.

In these studies, no attempt has been made for the formulation of bed roughness and sediment transport which is still imperfect but the experimental data have been tabulated and plotted in different ways to understand and clarify some aspects of the problem. These flume data are compared with similar studies by Brooks, Vanoni, Nomico, Kennedy, Einstein, Barbarossa, Chien, Raudkivi, Hwang, Simons and Richardson, Engelund - Hansen etc.

The data are analyzed for bed friction factor, bed shear velocity and slope and sediment concentrations and the results indicate the increase with increase of mean velocity, rises to maximum and then gradually decreases and again slightly increases with increase of velocity.

Bed configuration

In the case of fine sand, fully developed dunes at low velocity are characterized by relatively short wave lengths and limited lateral extent. Dune pattern changed very gradually as the velocity increased. Further increase in the velocity, the sand becomes nonuniformly distributed in the flume, creating a reach with thick sand bed and another reach with a thinner sand bed. The bed becomes flat with further increase of velocity.

In case of coarse sand the bed undulations is entirely different. At low velocity the bed is covered with dunes of comparatively long wave. With the increase of velocity, the wave length becomes short and crest more sharp. Further increase of velocity makes the bed more rugged and the ridge weaved back and forth. Following Prof. V.A. Vanoni & Hwang another investigation is attempted to find a relationship between the friction factors of bed covered with dunes or ripples and the size and other geometric properties of these bed forms.

The resulting data were plotted and the results indicated the expected value of bed friction factor due to ripple of dune.

The bed friction factor was plotted with concentration of suspended sediment and in case of fine sand it was observed that the average concentration increased with decrease of bed friction.

For fine sand, the values of Manning's roughness coefficient "n" vary from 0.021 to 0.010 and for coarse sand the value of "n" becomes as 0.037.

Effect of sediment load on the velocity profiles

The effect of concentration on Von Karman universal constant (K) and hence upon the friction factor, depends on the settling velocity W as well as the concentration (C), so that for the same C, coarse material with high settling velocity has a greater effect on K and on the reduction in friction factor. By the same token, fine materials have low settling velocity and hence small damping effect even for large concentrations. With this idea in mind, K was correlated against the ratio of the power P^s to suspend the sediment to the power P_f required to over come hydraulic resistance to flow. Flume data was plotted according to the method of Einstein & Chien and correlation found in better agreement.

Bar resistance and flow intensity

The flume data are also analyzed according to method described by Einstein and Barbarossa interms of bar resistance, U/U_*'' and flow intensity of shear, ψ' and were plotted with Einstein's bar resistance curve for natural channel. The results appear similar to data already published by various authors on flume experiments.

Einstein's bed material discharge function

Another important relation presented by Einstein in sediment transport problem was his bed load function.

The experimental data are analyzed in the form of ϕ as ordinate against $\frac{1}{\psi}$ as abscissa. The plot of flume data are compared

in form to Einstein's bed load function indicating a continuous increase in rate of movement with increasing bed shear.

Applications

Prediction of future channel has a very great economic importance in river basin planning and development and in the operation and maintenance of river basin projects. For example, if a large dam or barrage is constructed on an alluvial bed river, all of the bed sediment normally transported will be trapped. The clear water released will tend to erode the channel bed downstream from the dam or barrage until a new equilibrium is established. Severe bed erosion may undermine costly installation such as bridge piers, diversion structures, sewerage outlets and bank protection works.

By application of the bed load function to an existing channel it is possible to estimate the rate of bed sediment supply. With the bed sedimentation rate a function of the discharge, the long term transport i.e. the average annual sediment transport, can be predicted only if the long term flow rates can be predicted. Most sediment problems can be solved satisfactorily if at least the flow duration curve for river section is known. Based on extensive measurements, it was found that the sediment transport relationship of Engelund-Hansen generally gave reliable results which describes the morphological phenomena fairly well, reads as

$$\frac{q_s}{D_{50}{}^{3/2}\sqrt{g}\Delta}-0.084\frac{ds^{5/2}}{\Delta D_{50}}.\frac{c^2}{g}$$

q_s = sediment transport per unit width
D_{50} = diameter of grain (of the bed material) for which

50% by weight is finer

$$\Delta = \text{relative density} = \frac{(\rho_s - \rho)}{\rho}$$

s = slope, c = chezy's coeff.; d = water depth,
g = accln. due to gravity.

Sediment transport (Q_s) over the entire width,

$$Q_s \text{-} 0.084 D_{50}^{\ -1} g^{-1/2} \Delta^{-2} C^{1/3} B_s^{\ -2/3} S^{5/3} Q^{5/3},$$

in which B_s = sediment transporting width of the river and
Q = river discharge per unit time

Wherever the hydraulic engineer deal with the flow of untreated surface water, he must consider to what extent his plans and design will be affected by the scour, transportation and deposition of sediment in the channels, reservoir or estuaries through which the flow will pass. Various aspects of the sediment problem are often encountered in the design of irrigation and drainage canals, the improvement or stabilization of rivers for navigation and flood control, the planning and design of the reservoirs, the maintenance of harbour channels, the purification of public water supplies and the control of soil erosion on watershed areas.

Conclusion

The results point out that the variation of bed material plays a significant part in bed roughness and sediment transport problem and in case of large wide channels of some of the world's biggest river systems like the Ganges, Brahmaputra-Jamuna where the hydraulic and sediment characteristics are different along the width of channels, the problem of bed roughness and sediment transport would be more complex. Due attention should be given to a more fundamental flume study of braided river like Brahmaputra which will aid in understanding the processes involved.

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FLOOD CONTROL IN BANGLADESH AND THE STUDY OF RIVER MORPHOLOGY

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Introduction

The disastrous floods of 1987 and 1988 have proved that small-scale flood control projects are inadequate for protection when severe flood engulfs a region. Though the small-scale schemes help to achieve some agricultural benefit,they take up substantial land for development,tend to aggravate flood levels in unprotected areas,and do not contribute significantly to general flood protection. Several of these small-scale projects have suffered extensive damages,the project infrastructures being washed away by the flood. This could not have happened had there been adequate flood protection works along the major rivers.

From the environmental consideration total elimination of flooding is neither feasible nor it is desirable. With this end in view,the national and international experts have already formulated a 26 component Flood Action Plan (FAP) in 1989 which mainly envisages embankment on both sides of major rivers, river training,channel improvement,protection of infrastructure for major towns and key installations,improvement in flood forecasting and warning as well as other non-structural measures including watershed management,afforestation,disaster preparedness,flood proofing etc.

Because of the unique geographic and hydrologic separation created by the major rivers,FAP has considered five planing regions(Drawing-1) for study. Construction of embankments along the major river will have difficult-to-predict consequences in the upstream as well as downstream reaches of the river. As such the Coordination Advisory Team (CAT) of

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FAP-25 considered the Morphological Study as a key element of Inter-region Study. Therefore,it is necessary to analyze morphological and hydrological effects of proposals emerging from the regional studies on the main river systems. The information could be provided to the individual regional studies,by which the physical,environmental,economic and social consequences of such changes could be analyzed. The consequences thus derived could in turn be communicated back,through the Inter-regional Study,to the relevant FAP components.

River system of Bangladesh

Morphological characteristics

The major rivers of Bangladesh are the Ganges,the Brahmaputra,the Upper Meghna and combinations of this three rivers,the lower Meghna. The Ganges and Brahmaputra river merge together into the Padma river,which in turn joins with the Upper Meghna to form the Lower Meghna. The Lower Meghna empties in to the Bay of Bengal. The Ganges-Brahmaputra and Meghna Basins are shown in Drawing-2. The rivers are characterized by wide meandering braided at places and are of very flat gradient. The principal geomorphic feature of much of Bangladesh is the deltaic deposits of alluvium brought down by the rivers from the large catchment lying outside the border of Bangladesh. The materials brought down by the Ganges and Brahmaputra rivers are mostly fine sand with lesser amount of silt and clay. Many rivers in Bangladesh are affected by significant bank erosion but no one suffers as severely and extensively as does the Jamuna river;only a portion is shown in the Drawing-3.Bangladesh through its intricate network of water courses drains an area of about 2 million square km of which only about 8 percent lies within its territorial limit. The long-range technical problem is further complicated by the great magnitude of peck flood flows (1.42 lakh cumec) and the enormous sediment load of the rivers,which in total is about 1.1 to 1.6 billion ton annually (Hossain,1993). Sediment concentrations of the Ganges (approx.3600 mg/l)and the Brahmaputra (approx.1200 mg/l) are amongst the largest in the world.

River-flood plain interaction

In a drainage flood plain,rainfall inundation heads up behind the embankments temporarily clogging the silt denundation out flow into the river. On the other hand breaches in the existing embankment can appear anywhere and aggradation and degradation of the mouths of spill channel causes variations in the flow characteristics at these points. Changes at offtakes and spill channels not only affect flow volumes and riverspill from year to year,but within years also,as the sand bars that form at the mouths are gradually degraded. Such exchange of silt laden water between river and flood plain changes the land topography.

The annual flooding of the flood plain has certain advantages, but the magnitude of flood water regularly causes bank slumping and erosion. As a consequence, every year a sizeable population is affected by erosion of their houses and agricultural lands and often by becoming displaced.

Structural intervention and consequences

River training

River bank stabilization on the major rivers in Bangladesh has been accomplished at one location only (Hardinge Bridge on the Ganges river). With the continued use of rip rap, the Ganges river has been narrowed and confined within a channel under the Hardinge Bridge for more than 80 years. Bank protection at other locations on the major rivers cannot be considered that successful over a long period of time nor can they be compared with the aforesaid effort. The problems associated with river stabilization on the major rivers are mainly due to very deep scour depth (18 to 40 m) and relatively large velocities (greater than 3 m/sec) during flood flows. Onthe other hand, the low flow equilibrium has already been destabilized and is continuously being aggravated by increased upstream withdrawal in the extra flood time. To adopt suitable river training works and improved construction practice also require special study due to high sediment carrying capacity of the rivers and loose finer boundary materials.

In the braided channels, areas of local bank protection can be rapidly outflanked by the changes in the thalweg. There are problems also in maintaining construction quality.Proven bank stabilization materials are expensive to obtain. The equipment required to place bank stabilizing materials on a large scale is not readily available within the country. Under such conditions,a bank stabilization programme will require expenditure until entire reaches of the river are stabilized.

Dredging and resuscitation of rivers

Available data suggest that the dredging of the main rivers has received less priorities earlier. But for channel improvements, dredging of the main rivers may be necessary in association with river training works. Considering the immediate need for flood mitigation,salinity reduction and navigation facilities,dredging of confluences,especially the offtakes should receive priority.

The problem of Gorai mouth siltation persists due to very low flow of the Ganges in the dry season. The nouth has been dredged several times to prevent channel degradation and salinity intrusion. But the efforts have not been very successful due to dredging capacity falling far short of the requirement.A similar situation prevails at the offtake of Arial Khan. The mouth of old Brahmaputra becomes silted up in the dry season and dredging has been done yielding no significant benefit. Bangladesh Inland Water Transport Authority maintains a fleet of dredgers at Daulatdia and Aricha to maintain the navigation route.

Heavy siltation in the intake channel of the main pumping station of the Ganges-Kobadak project has limited the supply of irrigation water to the system to a fraction of design value. Dredging in intake channel has not been successful.The rate of sedimentation has exceeded the rate of dredging.

Minor rivers which are distributaries of the major rivers depend on the major rivers for their low-flow discharges. The off-take ofsuch minor rivers must be maintained in order to assure regime low-flows in the minor rivers.

Drainage impediment

The system of embankments on interlinked rivers from polder like areas, which are to be drained efficiently through the tributaries as far as possible by gravity drainage with restrictive use of pumped drainage. The monsoon water if held back after harvest of Aman to the bankfull stage of channels and in basins could help pisciculture and irrigation of adjoining lands. It would also retard moisture depletion to the benefit of non-irrigated crops. A large number of projects have constructed rural infrastructure with little consideration of their effect on the drainage.As a result natural drainage is impeded.

Embankment failure

Embankment breaches have occurred in almost the entire length of the original Brahmaputra and Teesta frustrating their flood protection objective.
Construction of extensive embankment in the upper catchment has increased the rate of travel of flood wave as well as have increased the maximum discharge and flood levels. Thus the potential threat of increased damage from sand casting due to breach of embankment has also increased. The determination of the setback distance of the levee systems has also become more difficult, in consideration of its impact on the morphological process during the engeering life of the levee.

Changes in the river morphology

Published results of several studies after the 1988 flood indicate that morphological changes may occur to the natural state of the river of various magnitude and timescale as a result of engeering intervention on the flood plain and along the river bank. Jamana bridge project Phase-2 study (1989-90) setup a mathematical 1-D morphological model of the Jamuna from Dibrugarh in India to the confluence with the ganges. The study produced simulation results of a 75 year timescale with respect to several changes in scenario occurring in full at the start of simulation viz: increased denudation of the catchment,damming of the tributaries, construction of levees, sea level rise, diversion of siltfree water in the upstream etc.

China-Bangladesh Joint Expert Team (1989-91) also conducted a similar model study which says upstream of Kazipur the bed level will degrade by 0.1 to 0.3 m while downstream beyond this point deposition of upto 1.0 m with consequent stage rise of 0.6 m will occur. The annual sediment transport may increase by about 27%-50% of the current average annual sediment transport at Bahadurabad (about 500 million tonnes). This increase,the study considered,not likely to have an unfavourable effect on the Padma morphology due to its higher transport capacity.

Recently completed River Training Studies of the Brahmaputra river (FAP-!, 1993) utilized a 1-D morphological model (JGP) developed by the Surface Water Modelling Centre (SWMC) comprising the Jamuna (Brahmaputra), the Ganges and the Padma. The JGP has then been applied to test the impact of different interference scenarios (over 100 years simulation period) viz.,constriction ranging from 6 km (about 50% of the existing) to 4 km just downstream of the Teesta confluence, complete levee system on both bank, Jamuna multipurpose bridge construction,50% increased denudation and sea level rise (0.5m) due to the Green House effect.

Morphological studies

Approaches

River morphology applied to alluvial rivers has to deal with flow, governed by complex process of interaction between the sediment transport and the hydrodynamics. There have been several different approaches in the way these process have been analyzed. Several empirical relationships have been developed on the regime concept; analytical and stochastic methods have been developed on meandering, on power approach and thresholds, rational approach and on formation of alternate bars, bifurcation and confluence. One major drawback of these relationships are that,these describe equilibrium condition. A further limitation of these methods is the fact that the change through time can not be predicted but only an estimate of the final equilibrium condition. On the contrary many mathematical models in use at present are one dimensional and thus can not model plan-form and channel shapes, however these are able to simulate other variables (viz: sediment transport and longitudinal slopes) with time and to predict final equilibrium slope. Thus combination of all the analytical approaches will have to be combined with the mathematical modelling (DHI, 1993).

Sediment transport modelling

Sediment transport modelling is a very special problem in fluvial hydraulics. Scale models can only model the immediate neighbourhood of the structure being studied and when large scale and long term consequences of man's intervention are concerned (hundreds of kilometers of river courses,tens of years of morphological variations), analytical and mathematical model study is the only solution. Another issue that limits the accuracy of physical modelling is the scale distortion which is almost unavoidable whenever it involves sedimentation.

The sediment transport models so far available are all developed for non-cohesive sediment in steady uniform flow and are not applicable to cohesive sediments, which normally constitute the wash load and is independent on the flow characteristics. Cohesive sediment transport is modelled by advection dispersion phenomenon.

However, following have been found so far in connection with different **studies with the Bangladeshi rivers.**

Major rivers: For the Ganges upto the confluence with the Jamuna, Hossain(1989) found Colby's method to give satisfactory results about bed load. Hossain (1989) also developed a simple power type regression equation which gave results varying from 56% to 20%. He found good co-relation between suspended sediment discharge and the average flow velocity. The river cross-section shape also appeared to be well correlated with concentration of the suspended sediment. (Hussain, 1989).

Under the Surface Water Simulation Modelling Program, Phase II (SWSMP-II), Engelund-Hansen formula was used due to insufficient data. Grain sizes from bed material samples taken from the three major rivers show considerable seasonal and spatial variability even within a single cross-section (Ref. /Interim Report-II of SWSMP-II), which contradicts the use of the available formulae due to being based on single grain size.

FAP-1 study has come up with the findings based on dune trackings that dunes in Brahmaputra migrate much faster than would be expected from any known bed load formulae. It concluded that more than 50% of the suspended sediment load contributes to the dunes movement.

Regional rivers: In the South West Region a significant amount of work is concerned with the cohesive sediment load carried by tides. SWSMP -II has recently developed a model based on only a very limited sediment data by the principle of advection dispersion transport phenomena, which is at the stage of demonstration now. It will have to be further developed with more data before it can be utilized.

For the non cohesive sediment transports a mixed result has been obtained by SWSMP-II . For lower transport rate Van Rijn, Engelund-Hansin and Engelund-Fredsoe give fair agreement, but for the higher transports Van Rijn's has been recommended with note of caution suggesting its unsuitability in matters of suspended load. In all other regions,Van Rijn's formula has generally been found to be the best suited formula for application for application, even though the available data usually is not sufficiently detailed to justify the application of a model with such complex description. It is probably due to the origin of formula being for small grain size than the other formulae.

Data cllection, analysis & interpretation

Foregoing paragraphs reveal that sufficiently comprehensive investigations, studies and research is yet to be made on the very important aspects of the rivers viz.nature, behaviour and response under different physical conditions,natural as well as man made, such as embanking, bank revetment and training works, etc. Data collected so far by several agencies, have come up with methodologies suited to their specific needs. Surface Water Modelling Centre (SWMC) in association with FAP and Bangladesh Water Development Board (BWDB) hydrology has collected considerable amount of hydrological, morphological and hydrographic data at several locations. But they are much too little for morphological modelling of rivers undergoing rapid morphological changes with seasonal and spatial variation of topography, topology and sediment compositions. Therefore,more intensive field campaigns are to be made with due care that errors are minimized.

A very important issue, as have been surfaced in the modelling work, is the applicability of the collected data.It requires a coherent package of parallel laboratory analysis,data cataloging and calibrating the models. A very well organized laboratory and computer analysis procedure needs to be commissioned. A model procedure adopted by the Surface Water modelling Centre has demonstrated to be reasonably effective.

Mathematical modelling

Mathematical modelling of alluvial channels has advanced with the progress in the physics of fluvial processes and computer techniques. The applicability and accuracy of a model depend on the physical foundation, data quality and numerical technique employed. Existing nathenatical models are almost all based upon the idea that it should be possible to simulate hydrological flow conditions and the concomitant change in longitudinal profile of a river over a period of 20-50 years using a chosen sediment transport formula.

Until recently development of the hydraulic models has adopted rigid bed friction slope (determined on Manning numbers). Work is now progressing to incorporate mobile bed modelling by incorporating morphology(mostly sediment transport).

SWSMP experience so far is that, morphological models on a regional scale is at present a very difficult task because of the complexity of the regional hydraulic models and incomplete understanding of the sediment transport phenomena,particularly at the enormous bifurcations. Raw data on topography also have to be processed with a systematic judgement to remove the localized perturbation and produce an equivalent channel for running the large scale models. Originating from the regional sediment transport models,work on several morphological submodels have recently been initiated.One very important comprising Jamuna, Ganges and Padma(JGP) has been utilized in the FAP-1 studies.

Conclusion

All the above methods, may it be deterministic or stochastic requires that rigorous testing and calibration of the constituents equation describing physical relations are made against the field data.

With due regard to the most important consideration in order to tackle a gigantic task like free-of adverse impact on river morphology, FAP has rightly adopted the staged approach. This way successive occurrence can be compared with the prediction on which the design has been based and subsequent adjustment made to the next stage of development. Thus a regular comprehensive field campaign at key points must be instituted which should at limited scale, continue in to the operation and maintenance period.
In the absence of any other better tool, the present approach will be continued by SWSMP. However, such models will be based on major assumptions, which need to be studied and validated. In this connection the ongoing River Survey Projects <u>FAP-24 can play a pivotal role.</u>

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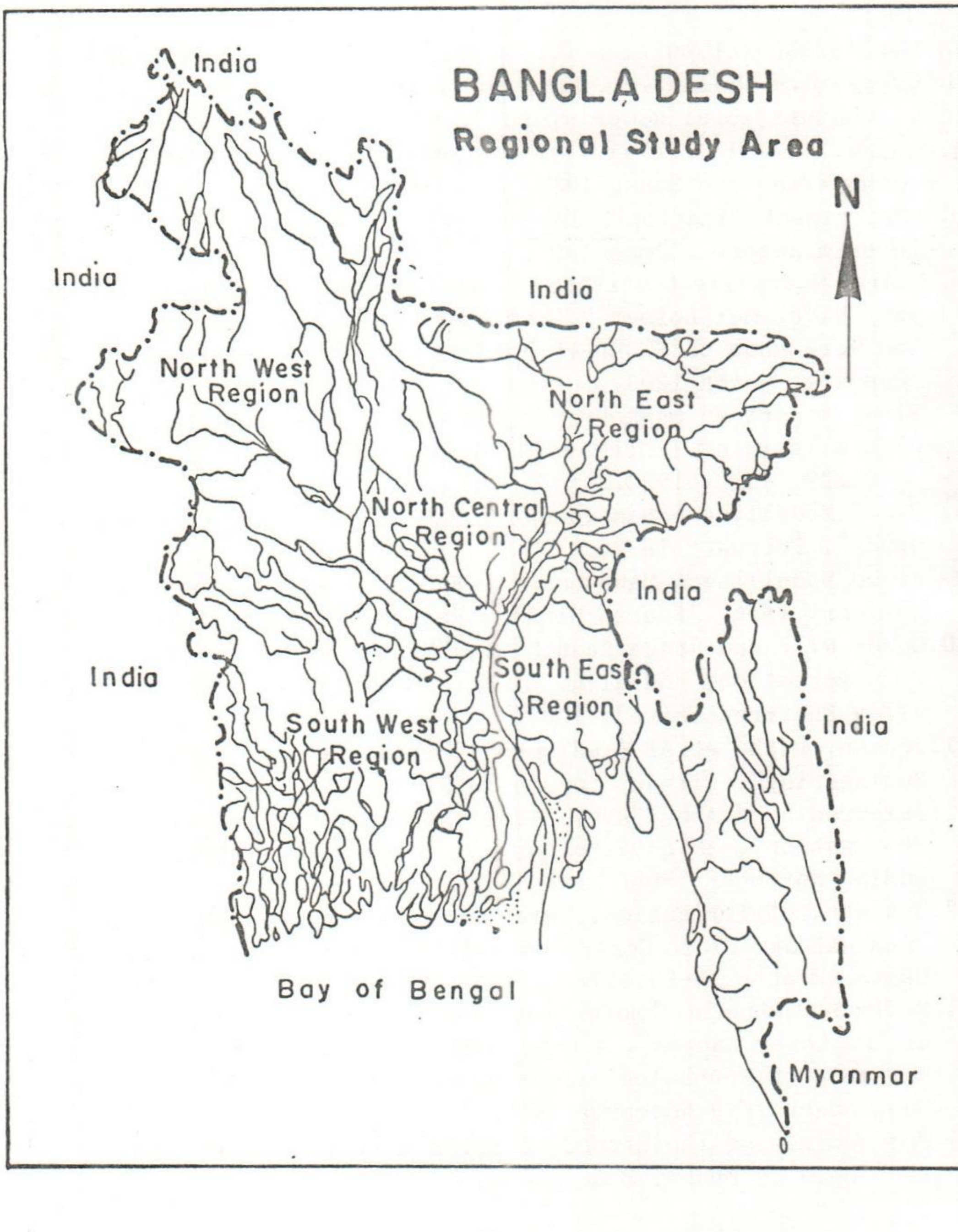
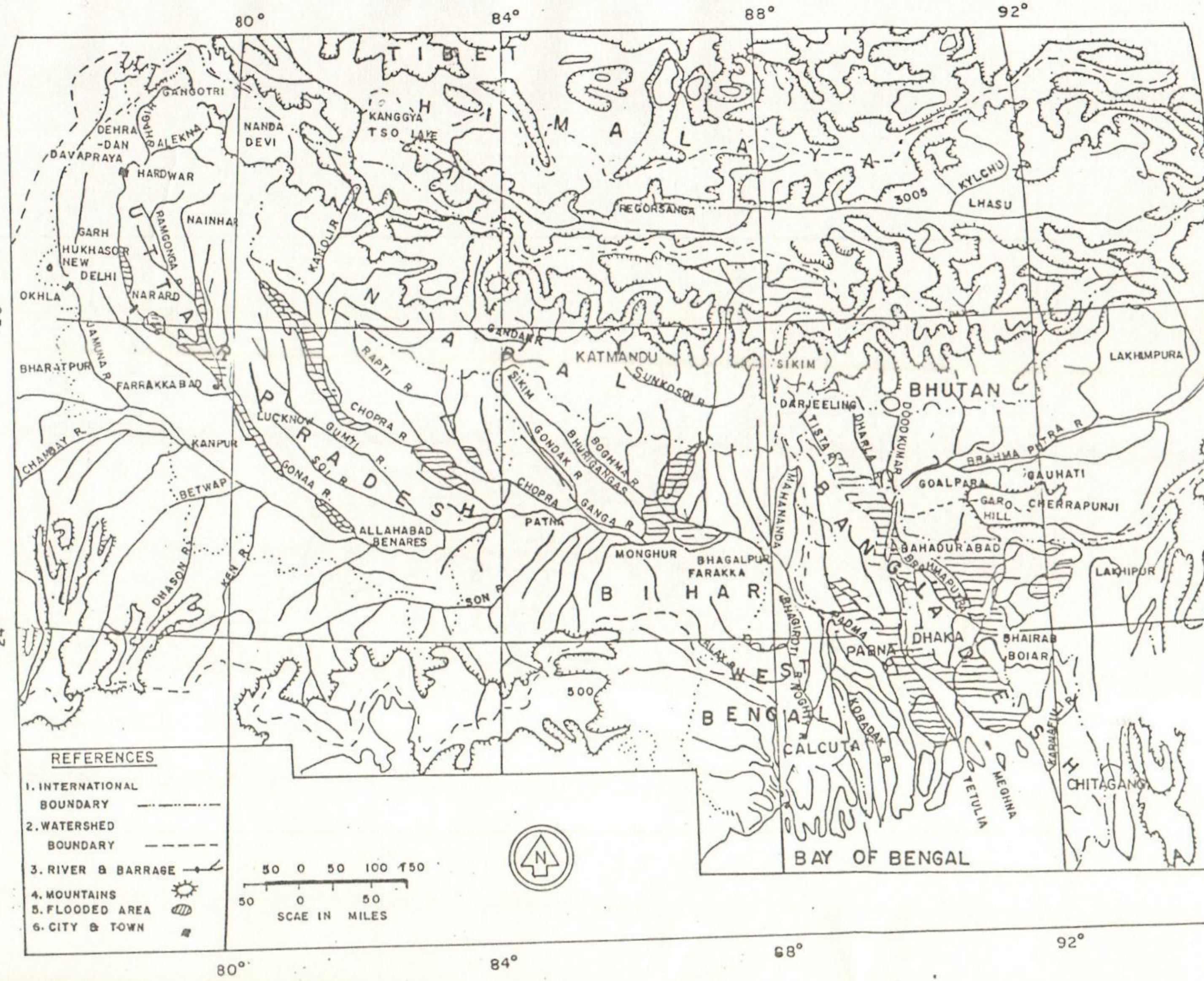
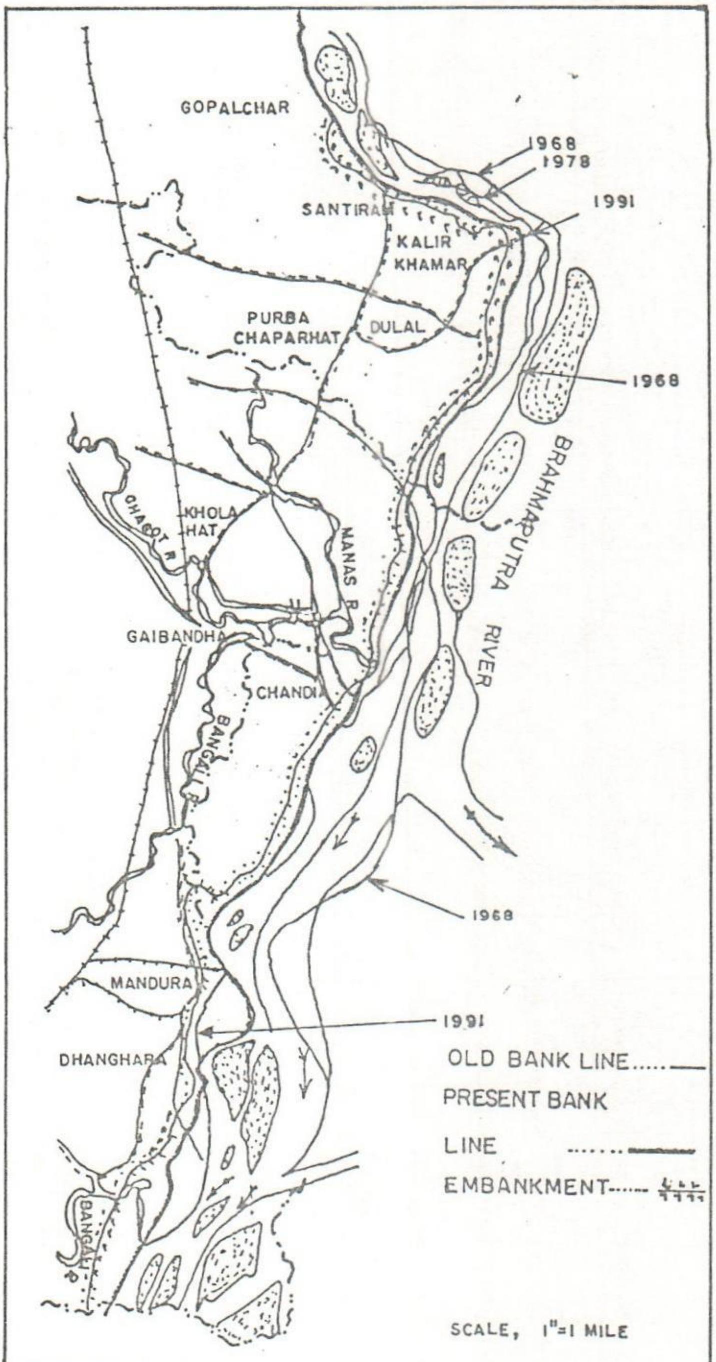
DRAWING. I


BANGLADESH
Regional Study Area
N
India
India
India
India
India
India
North West Region
North East Region
North Central Region
South East Region
South West Region
Bay of Bengal
Myanmar

DRAWING. 2. GANGES - BRAHMAPUTRA AND MEGHNA RIVER BASINS



DRG. 3. MIGRATION OF JAMUNA RIGHT BANK



DETERMINATION OF BASIC DESIGN SOIL PARAMETERS FROM ATTERBERG LIMIT TEST

*Md. Hanif Mozumder**

Abstract

A wide variety of problems associated with soil mechanics are solved by combination of theoretical knowledge and knowledge of the properties of soil under consideration obtained from laboratory tests. In some cases, for preliminary calculation purposes, some of the essential information can be approximated using reliable estimations.

The aim of this paper is to evaluate whether it is possible to estimate the unconfined compression strength and compression index and other physical properties of soil for the calculation of bearing capacity and settlement of cohesive soil only from Atterberg limit tests.

In this paper, correlation of unconfined compressive strength with liquidity index; compression index, C_c , with liquid limit, LL; compression index, C_c , with natural moisture content, w, are presented for the regional soils of Bangladesh. Atterberg limits also give the information on the consistency of soil and some physical properties of soils.

The main achievement of this paper is that foundation analysis on cohesive soils can easily be started from Atterberg limit tests based on some empirical correlations in context of Bangladesh soils.

1. Introduction

A structure may fail by both shearing failure of soil or

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excessive settlement of the underlying soil. Shear failure is the major cause of failure of many of structures. So, bearing capacity analysis by using shear strength of the soil is of great importance. Many attempt such as Triaxial compression test, Unconfined compression test, Direct shear test have made for the determination of soil strength.

Settlement analysis is an important subject in soil engineering and it is more pronounced for foundation on soft clays. There are three main types of settlements, viz, immediate settlement, primary consolidation and secondary consolidation. Among the three components of the total ultimate settlement, the most important is the settlement due to primary consolidation determined using the parameters of one dimensional odometer test.The test is performed in a variety of ways and the results are also interpreted differently to estimate the necessary soil parameters.

The most commonly used test parameters for the estimation of total ultimate settlement is the compression index, C_c , determined from the e-logP curve of a consolidation test. But both for the bearing capacity analysis and settlement analysis triaxial compression test, unconfined compression test and consolidation test is performed in the laboratory and these are laborious and time consuming and expensive too. But within very short period and less effort and more economically how we can analyse the mentioned points in the case of Bangladesh soils using only Atterberg limits test.

In this paper, correlations of unconfined compressive strength with liquidity index; compression index, C_c , with liquid limit, LL; compression index, C_c , with natural moisture content, w; are present for the regional soils of Bangladesh.Atterberg limits also give the information on the consistency of soil and some physical properties which is shown in Table-2.

2. Literature review

2.1 Atterberg limit

In 1991, the Swedish agriculturist Atterberg divided the entire ranges from liquid to solid state into four stages

(1) The liquid state
(2) The plastic state
(3) The semi solid state
(4) The solid state

The Atterberg limits which are most useful for engineering purposes are; liquid limit (LL), plastic limit(PL), and shrinkage limit (SL). The limits are expressed as percent water content.

Consistency of cohesive soil is shown as below.

```
---------------------------------------------------->   Moisture
decreasing
Liquid  ! Plastic     ! Semi solid     !   Solid
state   ! state       ! state          !   state
        !             !                !
        !             !                !
        LL            PL               SL
```

Liquid limit , LL:
The liquid limit is defined as the moisture content, in percent, at which the soil changes from a liquid state to plastic state. LL=50, means over 50% water content the soil particle remain in suspension and below 50% water content the soil is in plastic state which can be remolded without development of cracks.

Plastic limit, PL:
Plastic limit is the water content (in percent) corresponding to an arbitrary limit between plastic and the semi solid states of consistency of soil. It is the minimum water content at which a soil will just begin to crumble when rolled into a thread

approximately 3mm (1/8") in diameter.

Shrinkage limit, SL:
Shrinkage limit may be defined as a maximum water content below which the soil will not causes to decrease in volume on further drying.

Plasticity index, PI:
Plasticity index is defined as numerical difference between the liquid limit (LL) and the plastic limit (PL) of soil.

$$PI = LL - PL$$

Liquidity index, LI:
Liquidity index express as a percentage.

$$LI = \frac{w - PL}{LL - PL} = \frac{w - PL}{PI}, \text{ Where } w = \text{natural water content.} \quad \ldots\ldots(1)$$

Consistency index, CI:

$$CI = \frac{LL - w}{LL - PL} = \frac{LL - w}{PI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

$$CI + LI = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3)$$

2.2 Consistency of soil
By consistency is meant the relative ease with which a soil can be determined. This term is mostly used for fine grained soils for which the consistency is related to a large extent to water content. Consistency denote the degree of firmness of the soil which may be termed as soft, stiff or hard.

3. Presentation of result:
Quite a large number of cohesive undisturbed soil samples are tested annually for the determination of their q,q,LL,Pl and plasticity index (PI). Range of results of q_u, w, LL, PL, and PI of the undisturbed soil samples collected from some 48 structure

sites located in the district of Borguna, Comilla, Kustia, Rangpur, Dinajpur, Pabna, Chittagong, Noakhali, Dhaka, Mymensingh, Rajshahi, Sylhet, Jessore, Faridpur and Khulna are presented in Table-1. The presented q_u-values correspond to peak values.

Attempt have been made to correlate the value of q_u -with the values of LI and natural water content w, and theses are shown in Figs. 1 and 2 and average curves are drawn. In Fig. -1 the best fit straight line gives the equation $q_u + 9LI = 12$ ....(4) and the value of correlation coefficient 0.63. Thus knowing the values of w, LL, PL and PI, of a cohesive soil stratum of Bangladesh, the value of consistency index, CI and the value of q_u for that stratum can be estimated from equation (2) and (4). Using the value of CI, obtained from equation (2), consistency, N-value, q_u and other physical properties of soil can be approximated from Table-2.

Fig. 2 indicates as asymptotic relation between q_u and w, from this curve by knowing w only, q_u can be estimated. But limitation of this curve is that overall q_u -w relation is plotted up to 50% moisture content and for the clays of Bangladesh differ appreciably.

In this paper, clays of the district of Dhaka, Rangpur Noakhali, Jessore and Faridpur as tested in RRI have been considered. The values were analysed statistically to obtain correlations with the calculation of the values of correlation coefficients. Following correlations have been obtained for the clays considered.

Location	Equation	Correlation co-efficient.
Dhaka	$C_c = 0.01\ (w - 8.37)$ $C_c = 0.005\ (LL - 9.88)$	0.90 0.67
Rangpur	$C_c = 0.01\ (w - 17.65)$ $C_c = 0.005\ (LL - 12.95)$	0.96 0.66
Noakhali	$C_c = 0.009\ (w - 3.67)$ $C_c = 0.008\ (LL - 4.08)$	0.96 0.80
Jessore	$C_c = 0.01\ (w - 3.67)$ $C_c = 0.008\ (LL - 26.8)$	0.96 0.96
Faridpur	$C_c = 0.013\ (w - 13.85)$ $C_c = 0.01\ (LL - 20.79)$	0.99 0.90

TABLE-I

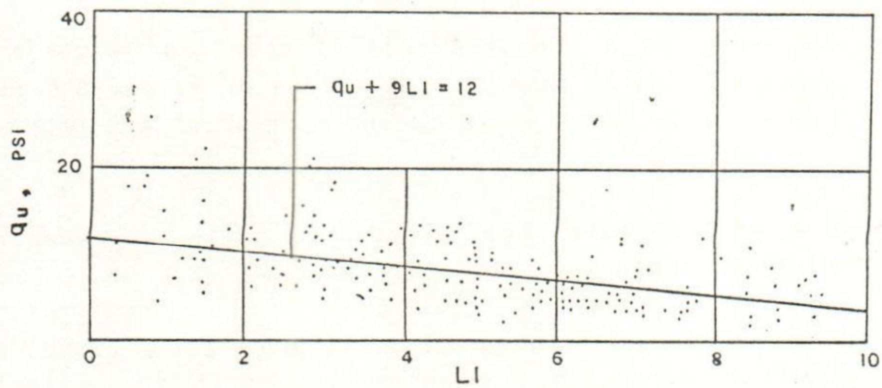
Range of values of qu, w, LL, PL & PI with other related information.

Location	Range of values							
	qu,PSI	W(%)	LL(%)	PL(%)	PI(%)	Depth,ft	eo	S(%)
Rangpur	15.38-30.30	27.78-36.40	39-64	25-46	14-18	10-34	.96-1.09	80-90
Bogra	9.00-17.25	27.30-32.60	43-76	15-30	13-50	10-22	.95-.98	85-92
Pabna	6.00-18.40	22.00-54.15	38-80	25-34	20-46	10-22	.73-1.45	92-100
Rajshahi	4.5-48.31	20.71-79.33	42-133	22-59	11-74	10-17	.77-2.00	92-100
Kustia	3.45-7.65	28.24-59.40	38-70	25-33	13-37	25-52	.75-1.35	100
Jessore	1.61-19.35	20.0-108.91	35-110	22-67	7-49	10-42	.78-2.00	93-100
Khulna	1.55-13.65	24.61-183.0	36-209	20-109	10-74	10-57	.81-2.70	80-100
Faridpur	1.28-27.29	24.61-48.49	35-115	20-72	8-47	10-37	.80-1.72	78-100
Dhaka	4.46-66.00	20.18-70.46	37-96	21-51	5-45	2-98	.70-2.35	90-100
Mymensingh	2.95-17.42	35.35-80.60	39-112	28-50	11-62	10-22	.97-1.24	97-99
Noakhali	9.82	35.75	46	30	16	20-22	.93	100
Chittgong	0.89-22.22	30.5-63.66	39-78	21-37	12-47	10-42	.90-1.65	83-100
Sylhet	2.60-45.82	22.20-182.20	30-169	20-98	8-50	10-47	.65-6.50	89-100

TABLE - 2

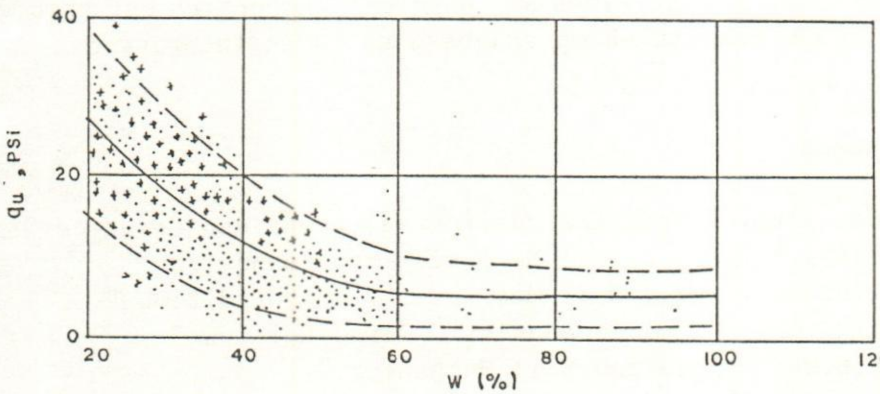
Qualitative and quantitative criteria of describing consistency.

Degree of consistency	Consistency Index,CL	N-values	Unit weight (Saturated) (kN/m^3)	Field Identification	Unconfined compressive strength q_u(kpa)
Very soft	0≤	0 - 2	15.71-18.85	Exudes between finger,when squeezed in hand	<20
Soft	0 - 0.25	2 - 4	17.29-20.43	Molded by light finger pressure	20 - 45
Firm of Medium	0.25 - 0.5	4 - 8	18.86-21.99	Can molded by strong hand pressure	40 - 45
Stiff	0.5 - 0.75	8 - 16	20.428	Cannot be molded by finger,can be indented buy thumbs.	75 - 150
Very stiff	0.75 - 1.0	6 - 32	20.428	Can be indented by thumb nails.	150 - 300
Hard	≥1	>32		Indented with difficulty by thumb nail.	>300



Unconfined compressive strength versus liquidity index

FIG·1



Unconfined compressive strength versus moisture content·

FIG·2

Discussion:
The values of q_u of a cohesive soil stratum can be estimated using equation - (4) by knowing the values of w, LL, and PL of that stratum. Hence an estimate can be made about the provable

value of q_u of a cohesive soil stratum bu collecting even only disturbed soil sample from that stratum.

Since the correlations between C_c and LL and w for regional soil gives approximate results, though we can use this correlations for the commencement of construction works.

From Atterberg limit test we can easily calculate consistency index, CI and using this CI-from Table -2 we can guess about the consistency, q_u , N-values and physical properties but this give approximate results which can be used is engineering design.

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A STUDY OF WATER QUALITY OF DIFFERENT RIVERS OF BANGLADESH

*Md. Abdus Samad**

Synopsis

Although Bangladesh is known as the land of rivers but a comprehensive data on water quality of different rivers are not available. The objective of this study is to store the data or water quality of different rivers of Bangladesh for different fields of uses. In this paper calcium-to-magnesium ratio; order of chloride, sulphate & bicarbonate concentration and relationship among calcium, magnesium, carbonate & p^H in natural waters and factors affecting the natural chemical composition of rivers have been discussed. In addition to these the sources and chemistry of some elements in natural waters have also been discussed in this paper.

1. Introduction

Bangladesh is known as the land of rivers. River water is the easily available source of water in Bangladesh. In our country river water and ground water generally used in industry, irrigation and domestic purposes. On the other hand,We also uses water for disposal of wastes ; for fishing, swimming, boating ; for navigation and for engineering construction works. But in different fields, different quality of water is required.

As we know, water is an universal solvent and practically all substances are soluble in it to some degree ; so natural water is not a chemically pure water. On the other hand river water is a mixture of rain water whose composition has been modified by

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contact with vegetation, soils and rocks. After leaving the area where the rain first contacts the ground, i.e., the source area, the water flows in a channel and is recognizable as a river.The chemical composition of river water might undergo further adjustment when the composition of river bed and banks changes or when a tributary transporting water or suspended materials with a different composition merges with the main flow or when the distribution of the biological organisms which live of the side or in the river changes. Beside these river water are generally polluted from industrial effluents, domestic sewage, fertilizers etc.

2.Nature of impurities

Generally the impurities present in natural water can be classified into four different groups as follows -

2.1 Inorganic impurities :

2.1.1. In suspension - Silt, Clay and colloids

2.1.2. In solution - Ca^{++}, Mg^{++}, Na^{+}, K^{+}, $CO_3^{=}$, HCO_3^{-}, CL^{-}, NO_3^{-}, etc.

2.1.3. In pseodo solution - Oxide of iron, alumina silica etc.

2.1.4. Dissolved gases- CO_2, O_2, N_2, H_2S, NH_3 etc.

2.2 Organic impurities :

2.2.1. In Suspension -Industrial wastes,decomposable organic matter in sewage plants, leaves etc.

2.2.2. In solution - Organic acid, organic matter etc.

2.2.3. In pseodu solution - Colloidal decomposable organic wastes such as animal secretion etc.

2.2.4. Dissolved gases - Methene.

2.3 Living impurities :

2.3.1. In suspension - Algae, bacteria, fungi and other living organisms.

2.4 Radioactive impurities:

2.4.1. In suspension - Cobalt, uranium etc.

3. Water sampling

In case of river water sampling the sampling stations should not be established at locations where mixing is incomplete, or where significant differences in water composition exist in the stream cross section.

The climate of Bangladesh is typical of the monsoon area in this part of Asia. There are two distinct seasons in Bangladesh - the rainy seasons from June to October and the dry seasons from November to May. For this kind of distinct seasons the river Research Institute (RRI) has taken observation twice during the year of study i.e. one observation in the period of Jan. -March and other in July-Aug. The water samples were collected with the co-operation of Surface Water Hydrology of BWDB.

4. Importance of study of natural water

The study of composition of natural water is of considerable importance for several reasons which may be summarized as follows :

- Water to be used in the home should be free from color and turbidity and should have no unpleasant odour or taste, harmful micro-organisms must be absent.

- Water used for irrigation should not contain any considerable quantity of soluble salts as these are injurious to growing crops as well as for the soils.

- water contains high percentage of soluble salts some times cause damage to bridge and other piers or abutment.

- Water used in boiler should not contain any substance in excess quantity which may cause scale, foaming or other troubles.

5. Sources and chemistry of some elements and gases in natural water

From the literature of geochemical investigation it appears that various ions such as Ca^{++}, Mg^{++}, Na^{+}, K^{+}, $CO^{=}_{3}$, HCO_3^{-}, $SO_4^{=}$, CL^{-}, SIO_2 etc. comes in natural water from different types of rocks such as igneous, resistates, hydrolyzates and precipitates by weathering and suspentinization process.

5.1. Calcium

Calcium is normally present in natural water in dissociated forms as the bivalent ion Ca^{++} . By the weathering process of calcium-bearing silicates minerals such as feldspars, pyroxenes and amphiboles may yield soluble calcium that is carried off in solution leaving behind a residual clay minerals . On the other hand in the presence of H^{+}, $CaCo_3$ is readily soluble in water by giving Ca++ in solution. The chemistry of the reaction may be shown as follows-

$$CaCo_3 + H^{+} \text{ ----> } Ca^{++} + HCO_3^{-}$$

5.2 Magnesium

By the surpentinization process the magnesium bearing silicate minerals such as forsterite gives Magnesium carbonate i.e.

$$5\ Mg_2SiO_4 + 4\ H_2O + 4\ CO_2 \text{ ---> } 2\ H_4Mg_3Si_2O_9 + 4MgCO_3 + SiO_2$$

Now, this $MgCO_3$ in the presence pf carbondioxide again gives

$Mg(HCO_3)$ which is more soluble. The chemistry is as follows -

$$MgCO_3 + CO_2 + H_2O \text{ ----> } Mg\ (HCO_3)_2$$

Once in solution magnesium has a stronger tendency to remain in that status than does calcium. This is indicated in a general way by the enrichment of magnesium in sea water where its concentration is second to that of sodium among the metalic ions. Magnesium present in natural water in dissociated forms as the bivalent ion Mg^{++}.

5.3. Chloride

The chloride - bearing igneous rock minerals are the feldspathoid sodalite, Na_8 [CL_2 $(ALSiO_4)_6$] and the phosphate mineral apatite. Sedimentary rocks, especially the evaporates is also the important sources of chloride. Chloride is present in all natural water, although in many areas the amounts are small. In most surface streams chloride is present in amounts considerable lower than sulphate or bicarbonate. Exceptions may occur where streams receive inflows of high-chloride ground water or industrial waste, or are affected by oceanic tides.

5.4 Dissolved gases

The commonest dissolved gases in natural water, include O_2, N_2, CO_2, H_2S, CH_4, CL_2, SO_2 and NH_3. Some of these gases are derived from the atmosphere and others come from decaying organic matter and gaseous pollution of the air. The source of chlorine gas is the water treatment plant. An important sources of O_2, & CO_2 is the metabolism of aquatic plants and animals.

The solubility of a gas in water varies inversely with temperature and directly with pressure. Solution of NH_3 has a basic reaction and those of CO_2, SO_2 are acidic in reaction.

6. Laboratory studies

To study the water quality of different rivers of Bangladesh the RRI with the co-operation of Surface Water Hydrology, BWDB collected some water samples from different locations of different rivers. The sampling location have been shown in the attachment-I. Analyses were dome for p^H, Free Carbondioxide, Calcium, Magnesium, Carbonate, Bicorbonate, Sulphate, Chloride, Salinity and total evaporated residue. Simple gravimetric method & volumetric method and improved volumetric method such as complexometric titration method were followed. The p^H was determined by a p^H meter. The reagents used in the analyses were of analytical reagent grade. The analytical results have been presented in Table-I.

7. Results and discussion

From the literatures of natural water it appears that the ratio of Ca to Mg commonly ranges from about 5 to 1 to about 1 to 1. High values for the ratio suggest that the water obtained Ca from relatively pure limestone or other calcium carbonate precipitate or that gypsum was available for solution. Low values of the ratio may indicate that magnesium silicate minerals are being dissolved or that dolomitic rocks are being attacked. Now from the Table-I it appears that the sources of Ca and Mg are the dolomitic rocks as the Ca to Mg ratio is about 2 to 1 to about 1 to 1.

The order of abundance of anions such as CL^- , $SO_4^=$, & HCO_3^- in sea water is $CL^- > SO_4^= > HCO_3^-$ but in case of natural water is the reverse i.e. $HCO_3^- > SO_4^= > CL^-$. In this respect, it can be observed from Table-I that in our sampling area the order of concentration of the anions follows the natural water systems. It may be mentioned here that the above order in case of natural water is only applicable when there is no tidal effects is observed.

From the literature of natural water it also appears that in chloride concentration water the dissolving tendency of Ca & Mg bearing rocks are increased. Again from Table-I it is seen that in our sampling area when chloride concentration is decreasing then simultaneously the concentration of Ca & Mg are also decreasing.

In the natural water there is a good relation between P^H & CO_3 i.e. if p^H is high then the CO_3 concentration is also high. From Table-I it is seen that when p^H is decreasing then the concentration of CO_3 is also decreasing.

As the data available on the various parameters for this paper relates to only one season of a particular year and as such it is not possible to make any specific comments about the seasonal variation of the analysed parameters.

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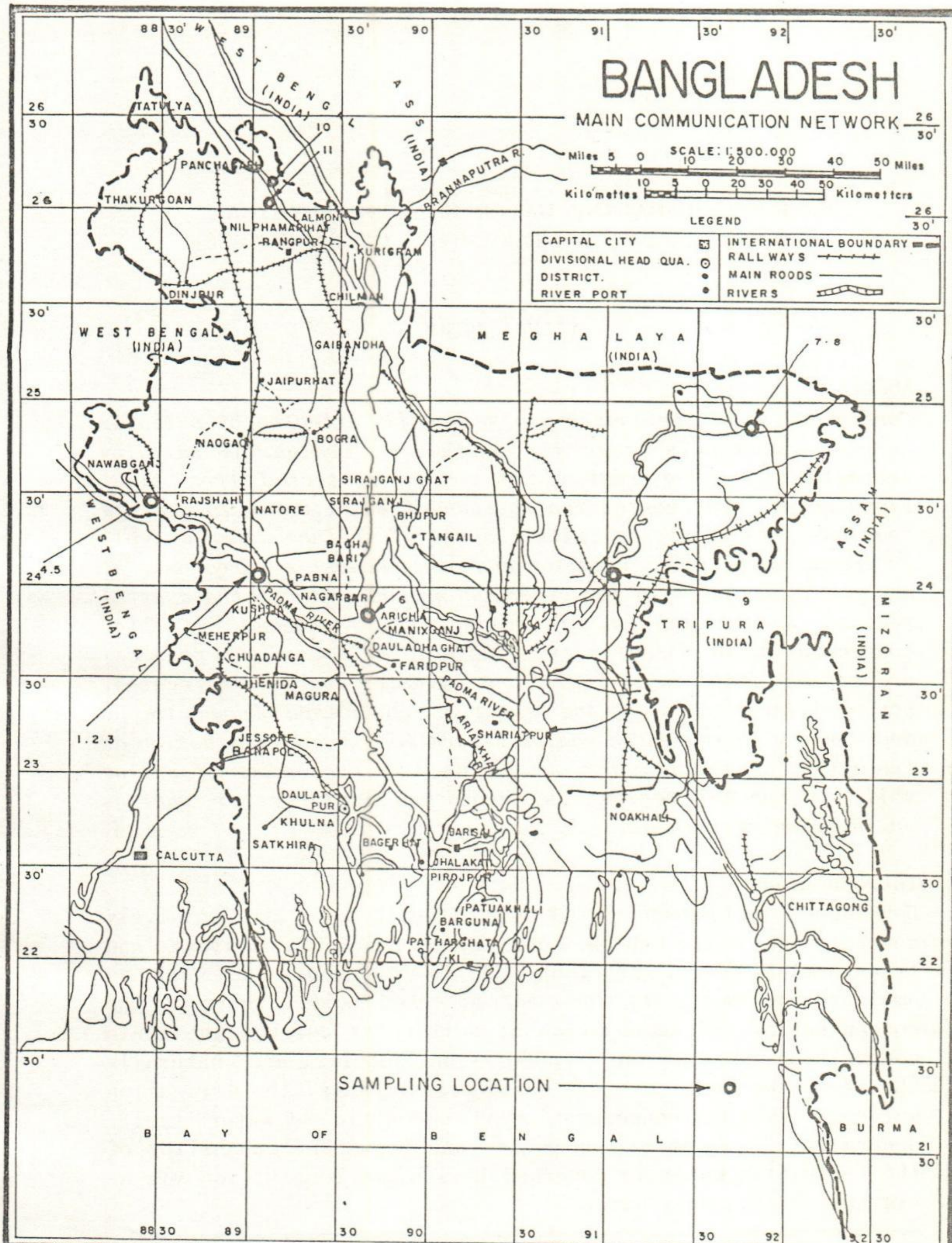
TABLE -1

RESULTS OF CHEMICAL ANALYSES OF SOME REPRESENTATIVE WATER SAMPLES COLLECTED FROM DIFFERENT RIVERS OF BANGLADESH.

(RESULTS EXPRESSED IN PPM EXCEPT pH)

Name of river with location and period	Sample No as mentioned in the attachment -I	Lab. pH	Free CO_2	Ca	Mg	CO_3	HCO_3	SO_4	CL	Salinity	TER
1	2	3	4	5	6	7	8	9	10	11	12
Ganges river at Hardinge bridge January, '83	1	11.72	19.13	63.08	30.55	14.70	188.30	13.72	16.89	60.48	240
	2	11.17	16.18	58.52	24.71	13.52	194.88	10.78	17.19	61.02	250
	3	11.37	17.66	57.00	25.08	10.58	200.26	12.25	17.59	61.74	238
Ganges river at Godagari january, '83	4	10.91	20.60	57.00	26.90	15.87	184.12	19.60	17.39	61.38	245
	5	11.01	16.68	56.24	27.81	13.52	196.07	21.07	17.57	61.71	240
Ganges & Jamuna at Aricha up, January,'83	6	10.97	13.73	37.24	15.50	9.40	89.07	9.80	8.45	45.25	190
Surma river at Sylhet town, March,'83	7	10.48	7.85	23.56	8.21	7.05	55.59	13.23	8.44	45.23	125
	8	10.61	6.87	22.80	9.57	6.46	57.98	16.17	7.95	44.34	120
Surma & Meghna at river at Bhairab March, '83	9	9.91	2.94	11.00	8.21	2.35	43.63	37.73	6.95	42.54	280
Teesta river at Doani, July,'83	10	9.66	13.73	7.60	7.29	2.64	31.08	26.46	7.45	43.44	1400
Teesta river at Dalia, July,'83	11	9.79	11.77	8.36	6.38	2.35	32.87	28.42	7.65	43.80	1700



Source: By the courtesy of Eng. A.K. Azad, Lekha prokashani Uttara. contd······P/8.

EFFECTS OF SALINE WATER ON SOME PHYSICAL PROPERTIES OF COASTAL SOILS OF BANGLADESH

*NILUFA ISLAM**

Abstract

Coastal soils of Bangladesh are seasonally affected by salinity intrusion, particularly in the southwest region. The salinity intrusion in these region are mostly protected from tidal flooding by embankments. The absence of sweet water in the dry seasons in these areas limits the use of surface water as well as ground water for irrigation and other domestic purposes. The major soluble sakts in the river and ground water in the coastal regions are chlorides of sodium. The presence of high concentration of Na-salts cause adverse effect on the physical properties of the soils. Bulk density, water stable aggregates, non-capillary porosity and shear strength are known to be decreased with increased sodium saturation. A full evaluation of the effect of saline water on the physical properties of saline soils of coastal regions is needed particularly for various engineering structures.

Introduction

Development and management of salt affected soils are decisively governed by water which acts as a reactant, solvent and transporting agent. In fact, it is the cause as well as ameliorating agent for the salt affected soils. Whereas, the concentration and composition of soil water governs the ionic composition of the soil adsorption complex and naturally therefore the properties of solid constituents, the Irrigation water controls the concentration nd composition of water itself. Generally the movement, retention and transport capability of the irrigation water is governed by its own composition vis-a-vis that of the soil water.

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Soils are said to be saline if they contain an excess of soluble salts. Salinazation does not mean simply an increase in the amount of salt in the soil. but a catastrophic changes in the structure due to salt, which renders the soil useless for any further agriculture (Bogenerider, 1982). One billion hectares of land around the world have been affected by salinity (FAO/UNESCO, 1973). In Bangladesh, the area of cultivable land is 13,317 million hectares (BBS,1991). Unfortunately about 1/5th of the total arable land is on the grip of salinity and the perilous consequence of salinity has been creeping towards mainlands which will create a devasting problem in near future.

Salt affected soils in Bangladesh (Figure 1) occur in the coastal districts of Khulna, Barisal, Patuakhali, Noakhali and Chittagong. Out of 2.85 million hectares of the coastal and off shore areas about 0.833 million hectares are affected by different categories of salinity. In the costal are as saline soils are classified into four categories on the basis of EC, namely slightly saline (S_1), moderately saline (S_2), saline (S_3) and highly saline (S_4). The highest area is under S_2 category (0.426) million hectares) followed by S_1 (0.281 million hectares) and S_3 categories (about 0.080 million hectares). About 0.04 million hectares are under S_4 category.

The situation of salinity in Bangladesh is rather peculiar. The salinity existing in the coastal regions of the country arises due to a regular inundation of the soil by saline tidal water. Besides , the saline ground water table approaches the surface as one comes towards the river. Moreover, tidal bore and flood, almost a regular feature of the country, invade a large area of cultivated lands of the coastal districts paryicularly those of the the Southwest region of Bangladesh.

The saline soils have developed on fine textured alluvial and tidal deposits and are mostly protected from tidal flooding by embankments. There is absence of sweet water in these areas and only one crop, the transplanted aman rice is grown with varying yield. Not enough work has been done to address the salinity

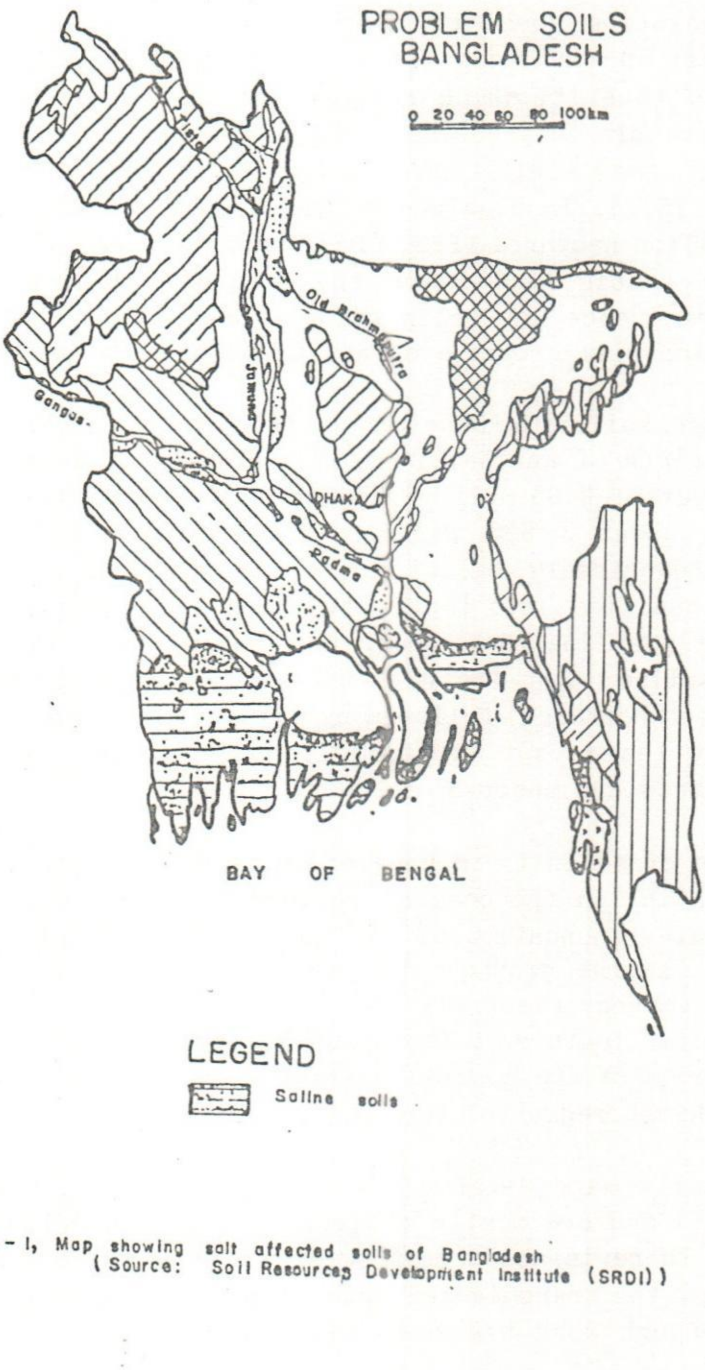

Figure – I, Map showing salt affected soils of Bangladesh
(Source: Soil Resources Development Institute (SRDI))

problems in these soils.The Bangladesh Agricultural Research Council initiated research projects on soil salinity and brackish water management (Anonymous, 1982 ; 1983;1992). However, studies on the effect of salinity on physical properties of the soil are scarce.

Causes of salinity

The salinity problems in Bangladesh are quite different than the saline areas of arid and semi-arid zones. The origin of salinity is mainly marine . Sea water surrounding the coastal zones of Bangladesh intrudes northward by :

a) Salt water inundation during cyclones or typhoons
b) heavy withdrawal of water upstream by other countries (e.g. India) and as a result saline water crept along main rivers and tributaries.

Besides,

c) withdrawal of ground water in the northwest for irrigation which encourages saline water movement in the fresh aquifer.

Chemical composition of surface water

One important consideration in the utilisation of water resources for various purposes (e.g agriculture, irrigation, drinking water) is its chemical quality, such that use over a long period of time does not result in deterioration of the soil resources. Water salinity has an extremely strong effect on the useability of river water for various purposes. Some of the coastal areas of Bangladesh are seriously affected by salinity intrusion, in particular the southwest region, where the saline water to a great extent hampers the use of riverine water for irrigation and domestic purposes more precisely during the drier months of the year.

The data on chemical quality of waters, particularly on salinity, of various river systems in Bangladesh have been collected sporadically by various sources during some decades. Among them,BWDB, that has carried out general monitoring of salinity levels, partly collected salinity data in connection with specific projects under the umbrella of BWDB,the Khulna Newsprint Mills and Delta Development Project (DDP). Extensive salinity data have also been collected by the Mathematical Model Study of Pussur-Sibsa River systems and Karnafuli River Entrance.

The historic salinity data, i.e. the data until 1990, have appeared to be rather scarce. The BWDB only carry out the regular monitoring on fortnightly basis. Furthermore, the monitoring stations are only located in populated inland areas, which means that salinity has not been monitored regularly in the Sundrbans or in the coastal waters. However, recently Surface Water Modelling Centre (SWMC) carried out extensive salinity data collection from coastal waters and a large number of monitoring stations have accordingly been set up by the centre.

Bangladesh Agricultural Research Council and Dhaka University jointly underlook co-ordinated research project on exploration of the production potentials of the coastal saline soils of Bangladesh. The objective of this project was to make a comprehensive study of the salinity regimes of the soils, the concentrations of soluble salts in surface and underground waters at different locations during the different time of the year (Tables 2 & 3).

Chemical composition of ground water

Chemical composition of ground water is a function of many factors and therefore there are wide variations from place to place. The composition of ground is water determined primarily by lithological formation and the circumstances of development. It makes a substantial difference whether the water-bearing layer was sedimented in fresh or in salt water. One of the most important factors leading to a change of the original composition of water is the solubility of rocks. Haloid rocks are very soluble and silicates are less. A large part of the Na and K content of water comes from the feldspars present in the weathering rocks.Al and Fe become soluble from silicates in waters having a p^H value of less than 3. Lime in waters originates from limestone, dissolved by solutions of carbonic acid. Waters in andesite-areas contain much Na and Ca, and those of granite-areas contain K.

A comparison of chemical composition of river, ocean and ground water shows that there is pick-up of salt when water moves through the soil profile during recharge (Table 1). Salts in the ground water are greater than the river water but less than the ocean, because of the excessive variability in ground water quality in space and time. It is therefore essential that the chemical composition of the ground water be examined in relation to the intended use before efforts are made for its exploitation.

TABLE 1 Analysis of river, ocean and ground water (Source : Gupta and Gupta, 1987)

Ions(meL^{-1})	River	Ground water	Ocean
Ca	0.85	9.38	12.29
Mg	1.89	18.57	52.86
Na	2.59	139.21	236.71
K	0.31	2.90	4.29
HCO_3	2.60	7.60	9.36
SO_4	0.85	28.18	30.43
Cl	2.76	133.25	263.74

Imamul Huq (1988) reported the chemical composition of ground water at different locations of the coastal areas of Bangladesh (Table 2). Land and Water use division of BWDB also reported the salinity of ground water by measuring EC at two locations of coastal areas (Table 3).

TABLE 2 Concentration of soluable salt in river and ground water at different locations during different time of the year (Source :Imamul Huq 1988)

Site	pH	EC	Soluable salts meq L-1								ppm
		dSm-1	Na	K	Ca	Mg	CO_3	HCO_3	Cl	SO_4	B
Payra River(Amtali)	7.3	0.60	3.1	0.16	0.90	2.8	1.30	1.9	21.5	11.0	0.10
Ground water(Amtali)	7.6	0.80	12.5	0.12	2.40	9.6	1.80	2.6	18.5	4.5	0.09
Bishkhali River (Barguna)	7.1	0.50	2.9	0.10	2.10	4.0	1.50	3..5	37.5	8.1	0.05
Ground water (Barguna)	7.5	0.60	3.6	0.19	2.50	3.5	0.93	1.9	30.1	5.6	0.09
Bishkhli River	7.3	0.60	2.1	0.11	2.30	3.3	0.59	2.2	62.5	3.5	0.10
Ground Water (Patharghata)	7.2	3.50	26.2	0.25	1.60	8.1	0.31	6.5	110.5	9.8	0.11
October - December '87											
Payra River (Amtali)	7.5	0.90	2.9	0.13	0.60	1.6	1.50	1.5	25.6	12.5	0.12
Ground water (Amtali)	7.1	2.10	21.6	0.43	1.90	8.2	1.60	2.3	69.2	9.6	0.25
Bishkhali(Barguna)	7.2	1.50	6.7	0.20	0.60	3.5	1.80	1.2	16.5	4.9	0.15
Ground water (Barguna)	7.3	2.10	12.5	0.31	1.10	1.9	1.10	3.5	66.0	10.5	0.18
Bishkhali River	7.1	4.6	26.7	0.62	1.50	3.2	1.20	4.9	79.5	12.5	0.32
Ground water (patharghata)	6.8	3.9	18.2	0.22	0.30	1.5	0.90	6.3	59.3	9.8	0.45
January - March '88											
Payra river (Amtali)	7.4	3.5	16.0	0.47	0.90	3.2	1.10	5.8	89.0	2.9	0.46
Ground water (Amtali)	6.9	1.8	12.0	0.32	4.20	3.2	1.60	6.9	51.5	3.5	0.28
Bishkhali River (Barguna)	7.0	3.2	22.5	0.15	1.60	2.5	1.50	7.5	91.5	3.8	0.49
Ground water (Barguna)	7.4	2.3	25.5	0.32	2.50	4.5	0.90	2.5	72.5	2.6	0.62
Bishkhali River	7.3	5.8	69.5	0.50	0.90	3.1	-	8.9	51.6	11.5	0.99
Ground water (Patharghata))	7.6	6.7	96.8	0.31	0.60	1.5	0.80	10.6	67.2	9.8	1.20

TABLE 3: Ground water table and salinity of ground water (Source BWDB, 1987-88).

Date	Tala U.Z. (Bonarpota)		Date	Assasuhuni U.Z. (Moheswarkati)	
	Depth of ground water from soil surface (cm)	Ground water EC (dSm^{-1} at 25^oC)		Depth of ground water	Ground water EC (dSm^{-1} at 25^oC)
15.11.87	59	4.10	12.11.87	16	4.72
28.11.87	57	4.20	28.11.87	22	4.32
14.12.87	55	4.30	13.11.87	19	4.57
30.12.87	62	4.80	29.11.87	27	4.62
8.1.88	104	5.30	8.1.88	39	4.70
23.1.88	116	6.70	26.1.88	94	4.80
8.2.88	142	7.52	6.2.88	115	4.65
23.2.88	131	7.62	20.2.88	105	4.90
1.3.88	128	9.30	1.3.88	132	4.97
13.3.88	122	10.20	23.3.88	135	5.20
12.4.88	108	10.40	12.4.88	160	5.50
28.4.88	163	11.48	27.4.88	161	6.00
X	101.41	7.24		77.89	4.72
CV%	40.38	38.67		75.52	10.79

Chemical effects

1) soil reaction (pH)

The pH values of salt affected and normal soils are shown in table 4. The most productive normal soils have pH in the range of 6 to 8. Whereas inland saline soils have pH, generally higher than 7, coastal saline soils may have pH (1:2 soil water ratio) as low as 5. Several factors influenced the pH values of the salt affected soils such as total concentration of soluble salts, nature of salts, composition of exchangeable cations, nature of cation exchange materials and the presence or absence of gypsum and alkaline earth carbonates (Gupta and Gupta, 1987).

TABLE 4 The pH values of soils (Source: Gupta and Gupta, 1987)

Nature of soil	pHs*	pH_2**
Normal	<7.5	<8.0
Saline	7.5-8.5	8.0-9.0
Alkali (Sodic)	8.2-10.0	9.0-10.8
Saline-alkali	8.5-10.0	9.0-10.5

*saturated soil paste

**1:2 soil water suspension

The excessive soluble salt concentration, particularly those of neutral salts, tends to depress pH values of the soils. The pH values of the normal soils, therefore, decreases when the soil is salinised particularly with NaCl. The highly saline soils having an EC of about 100 dSm-1, usually have pH value close to 7. It is for this reason that pH should not be the criterion for distinguishing saline soils from normal soils.

2) Salinity (soluble salts)

Salinity of the soil is measured through electrical conductivity (EC) of the soil solution extracted from the saturated soil paste. Conventionally, a soil having EC more than 4 dSm^{-1} has ben called saline and <4 dSm^{-1} as non saline. The most productive normal soils have EC less than 1 dSm-. The primary adverse effect of excessive salinity is due to rise in the concentration of soil solution, as a result of which the flow of water into the plant by osmosis is reduced or reversed and the plant is starved of water even though the soil is moist.

The soluble salts in saline mainly soil comprise the chlorides and sulphates of Na,Mg and Ca. Increasing from low to high salinity, the proportion of sodium salts over magnesium and calcium salts increase. Likewise at higher salinity levels, the proportion of Mg-salts over Ca salts increases. Thus higher concentration of Na and Mg salts cause adverse effect to the physical properties of the soils and plant growth.

3) Exchangeable sodium Percentage (ESP) and sodium Adsorption Ratio (SAR)

Exchangeable Sodium Percentage is a very important chemical property which defines the physical character of the affected soils. This parameters is usually a good indicator of the structural stability of soil and of the physical response that may be anticipated when water is applied.

The alkali soils containing 2:1 type expanding clay minerals (illite, vermiculite, montmorillonite) exhibit unfavurable physical properties at levels of ESP greater than 15.In general, the physical properties become increasingly unfavourble with increasing ESP but at a given level of ESP physical properties are usually poorer in soils with 2:1 type expanding clay minerals containing a low electrolyte concentration, more so of alkaline salts (sodium carbonate, sodium bicarbonate) than in soils with clay minerals of non-expanding 1:1 type (Kaolinite) or mixed types. containing a high electrolyte concentration, more so of neutral salts (NaCl or Na_2SO_4). It is for this reason that physical properties of saline-alkali soils are better than alkali soils.

Physical effects

1) Bulk density

The bulk density considerably depends upon the texture of the soil: usually coarse textured soils have higher bulk densities compared to medium or fine textured soils. Bulk densities of salt affected soils, whether saline or alkaline are generally higher. High bulk density of these soils results in low infiltration and hydraulic conductivity. However, instances of increasing sodium saturation of exchange complex leading to decreased bulk density are also reported (Varallayay,1977).

2) Water stable aggregates

Water stable aggregates is one of the important aspects of soil structure problems of salt affected soils. By and large CEC and exchangeable Ca^{2+} play important roles in enhancing the water stability of the aggregates. It is particularly true in alluvial and acidic soils. In alkali soils because of high Na on the exchange complex, water stable aggregates decrease with increasing ESP (Table 5).

TABLE 5 Cumulative percentage of water stable aggregates of various sizes as affected by ESP (Source: Gupta and Gupta 1987).

ESP	Size greater than (mm)					
	2.0	1.0	0.40	0.20	0.075	0.045
6.2	40.0	61.0	77.8	88.2	92.0	92.7
10.0	35.8	54.8	74.0	83.2	86.6	88.3
15.6	20.6	44.6	70.7	83.9	90.1	92.0
21.8	18.2	36.0	63.6	79.1	84.7	86.2
37.5	18.0	38.3	49.7	64.6	71.1	74.2
58.0	0.8	4.0	19.4	34.3	45.9	51.4
CD at 5%	6.58	4.51	6.23	5.44	10.20	7.85

3) Pore-size distribution

Geometry of the soil pores and pore-size distribution, rather than total pore volume govern both saturated and unsaturated hydraulic conductivity. Soil permeability of water is known to be a function of square of the mean pore radius and thus any process which results in reduction in the size of the pores due to swelling or clogging will have a significant *influence on* soil permeability. Clay-Na interaction causes internal swelling in soils, thereby resulting in an increase in volume of solids at the expense of volume of macropores. Moreover, dispersion of clay and its downward movement results in clogging of micropores reducing pore-size and macropore volume. Thus, the non-capillary porosity of alkali soils is reported to be as low as 2 % compared to normal soils where it is in the range of 10-30 %.

4) Crack and crust formation

Alluvial, heavy textures alkali soils may develop a hard surface crust on drying after rainfall or irrigation because of low stability of structure, high dispersion and slacking nature. This crust reduces the infiltration rate causing over saturation and aeration problem and retards the root penetration.Drought sensivity of alkali soils may as well arise from the crust formation in these soils. Swelling clay in alkali soils on the other hand results in cracking which possess another problem of water management in these soils.

The problem of crust formation and cracking is also common in saline soils with low organic matter content. The crust formation in these soils with saline irrigation water is traced to stagnation of water containing high sediments which accumulate in surface and sub-surface layers.

5) Shear strength

Shear strength of soil is important in determining its suitability for construction of embankments and its capacity to withstand rainfall erosion, apart from known uses in soil mechanics. The phenomenon of erosion occurs when fluid flow induced shearing stress on a surface increase to cause particle removal from the surface. Amount and type of clay, pH, organic matter, temperature, water content, thixotropy and type and concentrations of ion in the pore and eroding fluids affect the critical shear stress which is defined as the stress required to initiate erosion on a soil. Investigations have shown that the structure of the soil and osmatic influence set up at the surface of the clay, due to the differences in the concentration gradients between the pore and eroding fluids produce swelling of clay surface. This swelling thus caused largely reduces the bonding forces which ultimately result in increased erosion of cohesive soils. The critical shear stress decreases from 25 to 5 dyne cm^{-2} when SAR of the pore fluid increases from 5 to 15 for the same CEC of the soil. The critical shear stress further reduces and reaches 1 dyne cm^{-2} at a higher SAR of 60 and at that level it is independent of CEC (Arubhaanandan and Heinzen, 1977). Moisture content, pH and shear strength relationship for an alluvial alkali soil indicate that with increasing pH of the soil, shear strength decreased (Figure 2).

Conclusions

From the above discussion, the following conclusions may be drawn :

1) The causes of salinity of Bangladesh soils are mainly due to chlorides of Na.
2) The presence of higher proportion of Na-salts over Mg and Ca salts create adverse effect on the physical and chemical properties of soils.
3) Clay-Na interaction causes internal swelling or clogging of soils which have a significant influence on soil permeability.
4) The problem of crust formation in saline soils with saline irrigation water is traced to stagnation of water containing high sediments which accumulate in both surface and sub-surface layers.
5) The critical shear stress which is required to initiate erosion in a soil is influenced by type and concentration of ions in the soil fluids along with pH, amount and type of

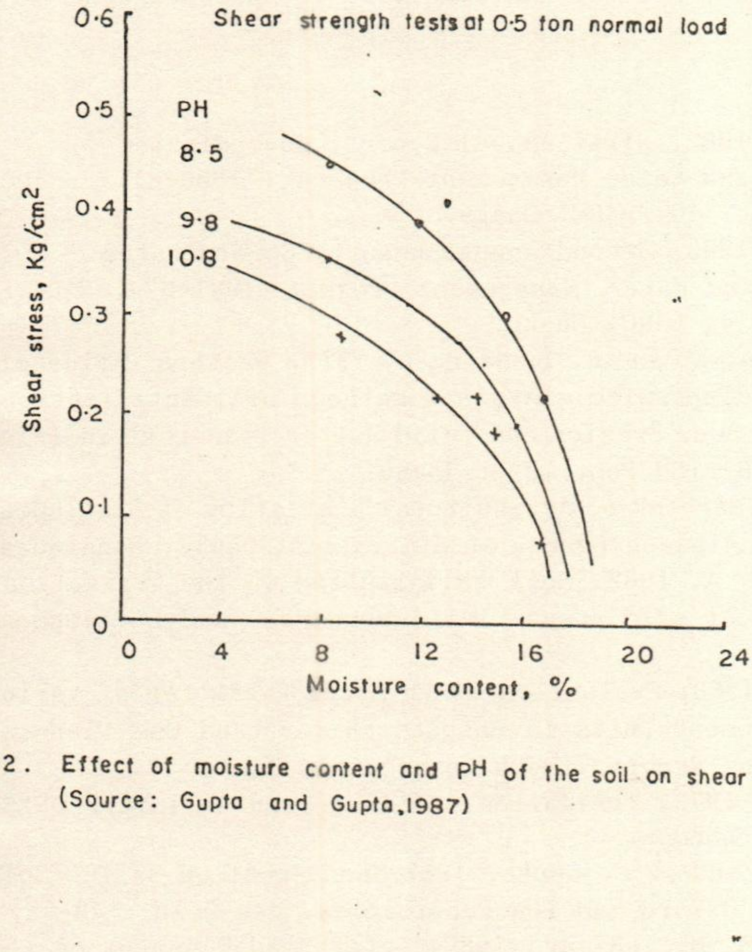

Figure-2. Effect of moisture content and PH of the soil on shear strength (Source: Gupta and Gupta,1987) clay.organic matter, temperature, water contentand thixotropy.

Therefore, it is important to initiate research projects on the effect of saline water on the physical properties of coastal saline soils to provide information about different physical parameters of soil required for construction or maintenance of embankments and other earthen structure on these region.

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IMPORTANCE OF MODEL STUDIES FOR BANK PROTECTION WORKS

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Abstract

Bangladesh is a riverain country. The Brahmaputra-Jamuna, the Padma, the Meghna are the three major rivers. The source of the tributaries of the Jamuna river is the Himalayas. The Jamuna river meets the Ganges near Aricha and from this confluence the river is called Padma up to the next confluence with the upper Meghna. Further downstream the river is called lower meghna and flows towards the Bay of Bengal. During the monsoon period a considerable part of the total discharge is spilling into extensive flood plain. Every year, they create a social problem by bank erosion. Physical model test and its implementation is the solution of said problems.

Basic principle

In a number of cases where theoretical calculations of the hydraulic performance of a structure is not possible, model tests are a tool to be applied. In general three types of similarities to be established for complete similarity to exist between the model and the prototype. These are geometric, kinematic and Dynamic similarity. It is the aim of the model investigation to reproduce the natural process such flow or wave pattern. The measurement of the model are then converted to the prototype (natural) by use of model law.

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River models are sometimes made in a distorted scale, so that the vertical scale is larger than the horizontal scale . Distorted models are those in which one or more terms of the model are not identical with their counterparts in the prototype.

Distorted model may have either geometrical distortion, or material distortion or distortion of hydraulic quantities or a combination of these. In the case of undistorted scale modelling, the vertical scale is identical to the horizontal scale. Undistorted scale is normally applied where structure are involved in the model testing.

Bank protection

In a broad sense, a bank can be defined as the sloping surface of land and coming into contact with a body of still or flowing water. Bank protection, therefore,includes any protective work that aims at maintaining the stability of land against the action of water.

Purpose of bank protection

The purpose of bank protection is to save the banks furious of water-soil interaction. The purpose that can be brought out in the following points.

(i) To check erosion of bank by current or wave.
(ii To check sliding of slope due to its gradual steeping because of erosion.
(iii) To prevent undermining of the toe of the lower bank by currents,waves or eddies.
(iv) To prevent sloughing or sliding of slope when saturated with water.
(v) To prevent piping due to various causes.
(vi) To check sliding due to drawdown of the flood.

Method of bank protection

The following methods of bank protections are in general practice

(1) Protection of banks by groynes or spurs.

(2) Protection of banks by vegetable cover either by turfing or sodding. Low growth of shrub and willows is by for the most effective cover. The method should be used if the slope is not subjected to strong current.

(3) Pavement of bank slopes by such materials which can resist quick erosion. Along with pavement the slope is given a temporary covering with brush wood or lumber mattress weighed down by stones for energy purposes. Where the current is very strong, protection is provided by stone revetment or various types of mattress such as willow, asphalt or articulated concrete.

Failure of bank protection

Some causes of failure of bank protective can be attributed to the failure of the toe, which is usually caused by

(a) In-sufficient depth of protection.

(b) In-sufficient weight of the protective material and therefore its being carried away by the current.

(c) Non-flexibility of the mattress. This failure may be experienced at heads of groynes or bank heads, or along a bank revetment work including retaining walls. Bank head and groynes may fail by out flushing due to in-sufficient protection at the wing. Long groynes may be breached at their middle portion owing to infringement by a sharp-turning current.

Paved slopes themselves may fail because of one or more of the following causes.

(a) In-sufficient thickness damaged by concentrated impact due to floating of debris or logs.

(b) In-adequate drainage at back of the bank.

(c) In-sufficient depth of paving.
(d) Permeability of mattress or paving.
(e) In-sufficient layer of crust stone under paving.
(f) In-sufficient bond-between upper bank paving and protection.
(g) Dislodging of mattress or paving blocks.
(h) Piping out of the lines from underlying soil due to inadequate filter blanket.

Importance of model study for this works

Model study is essential before taking up any protection work for the reason stated below :

(1) To determine the best location, position orientation, size and behaviour of groyne or spur.
(2) To determine the most effective and economical measure for this work.
(3) To find out permanent solution of the problem. If the protective work is done without any model study, the cost may be too high or the structure may collapse or any hazard may arise out of indiscriminate construction and to face this hazard a heavy recurring expenditure may be incurred.

Conclusion

Physical model study not only for the bank protection, also study for find out the solution of both erosion-siltation.

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HYDRAULIC MODELLING AND SCALING PROCEDURE

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Abstract
Hydraulic modelling is an important tool for the prediction of prototype behavior and for the finalization of design parameters for any hydraulic structure, river training and bank protection works. This paper represents the methodology for the selection of scale in hydraulic modelling in a comprehensive way. Some key points regarding the design and verification of hydraulic modelling are also presented.

1.0 Introduction

The behavior of river and river system is very erratic and complex. The solution of river mechanics problems are largely guided by certain boundary conditions, which are hardly possible to determine from the prototype conditions. But from the close observation to hydraulic phenomena in the model, it is possible to determine these boundary conditions with acceptable accuracy.

Mathematical solutions of vast natural river problems are normally done on the basis of some simplified assumptions and approximate boundary conditions. So, accurately determined boundary conditions from hydraulic modelling can be used to have

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reliable solution from the mathematical treatment of the problem. But some river related problems are still existing beyond the possibility of mathematical solution, such as, behavior of three-dimensional vortex in the vicinity of a local scour hole, but this problem can be understood and solved more accurately by hydraulic modelling.
The most important reason of hydraulic modelling of river problems is economy. Normally hydraulic structures such as bridges, culverts, barrages groynes , revetments etc. are constructed within the domain of rivers. These structures are constructed with specific goals and objectives. Most of the cases, these structures are very expensive. So, such an important and expensive structure does not permit an error in design or misinterpretation of actually prevailed condition in the field. As the actual and accurate mathematical formulation which happens in a river is hardly possible, so with the help of hydraulic modelling within limited constraints ,the possibility of misinterpretation of design parameters can greatly be reduced or even eliminated. Thus hydraulic modelling helps to avoid danger and financial losses which arise from the failure of structure due to improper design.

Sometimes hydraulic modelling helps to reduce the cost of a structure. Due to uncertainty of design parameters, the designer keeps a large safety factor making the structure more expensive. But once the better understanding of the process and certain behavior of the design parameter is attained by hydraulic modelling , the designer can change or reduce the size spacing , orientation etc. of his structure with confidence and consequently the cost will be reduced. So properly done hydraulic model investigation can act as a good tool for optimum design (from structural , hydraulic and economic points of view) of hydraulic structures constructed in the domain of a river.

2.0 Classification of model

Hydraulic model is generally classified as (a) Undistorted model (b) Distorted model (c) Fixed bed model (d) Movable bed model.

(a) **Undistorted model**

When all the similarity requirements are fulfilled in a model or if a model is perfectly similar to its prototype then the model is said to be undistorted or true scale model. In an undistorted model all the vertical lengths (including diameter of the bed material) and horizontal lengths are reduced with a constant scale factor and the slope ratio will be equal to unity. Results obtained from this type of model are quantitative and can be used successfully in the prototype.

(b) **Distorted model**

A model is said to be distorted if one or more similarity requirements are not fulfilled. A distorted model is in fact a compromise between a number of requirements and limitations. The rational approach will lead into the vicinity of the optimum point. But trial and error is only the way to find the best compromise.

In a distorted model vertical lengths and horizontal lengths of prototype are reduced to model with different scale factors and slope ratio deviates from unity. Results obtained from a distorted model are deviated from its prototype to some extend and does not represent to its prototype directly and that is why the results obtained from this type of model are qualitative. So profound knowledge , extensive experiences and fair judgements are essential to transfer data of distorted model to its prototype for use. To get precise result an undistorted model is expected but due to some physical and practical problems such as existing model facilities, precise and accuracy in measurements, fulfilment of hydraulic conditions, availability and characteristics of bed material etc. distortion (geometrical distortion, material distortion and the distortion of configuration) is allowed in some cases.

(c) **Fixed-bed model**

The fixed-bed model is designed with Froude's model law to hold strict dynamic similarity under the frame work of rough turbulent flow. In this type of model, the similarity of boundary resistance should also be ensured. The fixed-bed indicates that Sediment movement and bed forming processes are out of concern in designing this type of model.During the model construction , fixed bed action can be achieved with any suitable mechanism with the assurance that the required roughness in the model is properly reproduced.

(d) Movable-bed model

The name movable-bed implies that in this type of model due attention should be paid to sediment transport, erosion,deposition and bed-forming processes. The design criteria of this type of model is almost similar to that of fixed-bed model with a little exception. The exception is that in the movable-bed model extensive attention should be paid to attain similarity in incipient motion and bed forming processes and thereby ensuring the bed-load movement in the model. To fulfil the similarity requirement regarding bed-load movement, it is normally found to use light-weight material such as coal-dust, saw-dust etc.

3.0 Scaling procedure of hydraulic modelling

Scaling procedure in hydraulic modelling should be in such a way all the behavior and characteristics in the prototype could be reproduced in the model with a reduced scale. To achieve authentic results from the hydraulic modelling, it is necessary to reproduce the geometric, kinematic and dynamic similarity in the model and prototype.

3.1 Determination of vertical length scale

3.1.1 Froude's Model Law

Froude's number is defined as the square root of the ratio of inertia force to viscous force. Mathematically it can be expressed in the following way-

$$F= V/(gh)^{1/2} \qquad (1)$$

Where, F= Froude's number

V= Average velocity of the cross section
g= Acceleration due to gravity
h= Depth of flow

From similarity requirements ,the Froude's number in the model and in the prototype must be the same i.e.

$$F_p=F_m$$

$$\text{or} \quad V_p/(g_p x h_p)^{1/2}=V_m/(g_m x h_m)^{1/2} \qquad (2)$$

Where p and m indicates prototype and model respectively.
After rearrangements and considering $g_p=g_m$, from equation (2) ,one can write-

$$\eta_v=(\eta_h)^{1/2} \qquad (3)$$

Where $\eta_v=V_p/V_m$, velocity scale
$\eta_h=h_p/h_m$, vertical length scale

Equation (3) is the Froude's law for physical modelling in mathematical form.
Almost all the physical modelling of open channel flow are normally done on the basis of Froude's model law.

3.1.2 Reynold's model law

Reynold's law simply relates viscous force with inertia force through the following mathematical relation-

$$R= Vh/\nu \qquad (4)$$

Where R= Reynold's number

V= Average velocity in the cross section
ν= Kinematic viscosity (m^2/sec)

According to the similarity requirements,Reynold's number in the model and in the prototype must be the same i.e.

$$R_p=R_m$$

$$\text{or } V_p h_p/\nu_p=V_m h_m/\nu_m \qquad (5)$$

After rearrangements ,from equation (5) one can write-

$$\eta_v \eta_h \eta_\nu^{-1}=1 \qquad (6)$$

Where $\eta_\nu=\nu_p/\nu_m$,scale ratio for kinematic viscosity

Equation (6) is the Reynold's law for hydraulic modelling.

Here it is important to note that simultaneous fulfillment of Froude's model law i.e. equation (3) and Reynold's model law i.e. equation (6) in the model is quite impossible. So we are to move through a alternate way to avoid or to minimize the effect of viscous force in the model.
The effect of viscous force is no longer significant when the flow in the open channel is hydraulically rough and turbulent. So to avoid or to minimize the effect of viscous force, the flow in the model must be hydraulically rough and turbulent. Normally flow in open channel is turbulent when $R > 2000$ but for good minimization of viscous effect ,Reynold's number should not be less than 5000 in the model.

So, in designing a hydraulic model,one should strictly follow the Froude's model law to hold strict dynamic similarity with the assurance that the flow in the model is hydraulically rough and turbulent.

With the facts described above,the Reynold's model law need not to be used in the scaling procedure of hydraulic modelling and

another useful relation combining Reynold's number can be form in the following way-

$$R_p/R_m=V_p h_p \nu_m/V_m h_m \nu_p \qquad (7)$$

After rearrangements equation (7) can be written as-

$$\eta_R=\eta_h^{3/2}\eta_\nu^{-1} \qquad (8)$$

Where $\eta_R=R_p/R_m$, Scale for Reynold's number
Equation (8) is an important relation for the selection of model scale.

From equation (8) it is clear that one can not make vertical scale of the model large enough beyond a critical value after which the flow in the model will be no longer turbulent. A slight increment in the vertical scale causes a rapid reduction of model Reynold's number which in turn qualifies viscous force to affect the flow condition in the model.
Surface tension may have a significant effect on the fluid flow depending on the size of the model and flow conditions. Under turbulent flow condition ,the effect of surface tension is usually negligible. But if the model is very small , the effect of surface tension should also be investigated.
The effect of surface tension can be evaluated in term of Weber number which is the ratio of inertia force to surface tension.

Mathematically-

$$W= \rho V^2 h/\sigma \qquad (9)$$

Where W= Weber number
σ= Surface tension
From similarity requirements,equation (9) can be written as-

$$W_p=W_m$$

or $$\rho_p V_p^2 h_p/\sigma_p=\rho_m V_m^2 h_m/\sigma_m \qquad (10)$$

After rearrangements equation (10) can be written as-

$$\eta_\rho \eta_V^2 \eta_h \eta_\sigma^{-1}=1 \qquad (11)$$

Where $\eta_\rho = \rho_p / \rho_m$, Fluid density scale

$\eta_\sigma = \sigma_p / \sigma_m$, Surface tension scale

To avoid the effect of surface tension, the value of Weber number should have sufficiently large value in the model. The scale of Weber number is given by-

$$\eta_w = \eta_h^2 \qquad (12)$$

This equation simply states that Weber number ratio decreases too rapidly in the model with the slight increment in the vertical scale i.e. with little increment in vertical scale Weber number in the model reduces too largely and hence surface tension tends to be more dominating.

3.2 Determination of Friction and Slope Ratios

The average velocity in the cross section can be calculated from the following equation(Chezy's equation)-

$$V = (8ghS/f)^{1/2} \qquad (13)$$

Where V= Average velocity in the cross section

S= Average slope of the water surface

f= Darcy-Weisbach friction factor.

The friction factor may be calculated from the following relations, suggested by Keulegan and White Cole Brook respectively-

$$f = (2.21 + 2.03 \log(h/ks))^{-2} \qquad (14a)$$

$$f = (2 \text{ Log } (14.8\ h/Ks))^{-2} \qquad (14b)$$

Where, Ks stands for the height of the bed roughness. Equations (14a) and (14b) give comparable results.

Here , the value of Ks can be taken as 2 to $2.5D_{50}$. Where D_{50} is the grain diameter corresponding to 50 percents finer.

Here it can be mentioned that , the value of roughness projection,Ks ,cited above is quite approximate and no generally accepted method for estimating correct value of Ks in open channel flow has yet been put forward. So accuracy of the selected value of Ks in open channel flow largely depends on the engineering judgement and profound experience in the field of open channel hydraulics. But since Ks contributes to the friction factor (f) through logarithmic function, small error in estimating the value of Ks does not cause substantial error.

From equation (13),for the model and the prototype one can write-

$$\eta_v^2 \eta_f = \eta_h \eta_s \qquad (15)$$

From equation (3) and (15) the following relation can be obtained-

$$\eta_s = \eta_f \qquad (16)$$

This relation simply sates that the scale factor for the water surface slope and bed friction are equal.

3.3 Density of the model bed materials

From the similarity requirements ,it is essential that the Shield's parameter in the model and in the prototype must have the same value.In connection of Shield's parameter the incipient motion of the bed materials can be defined in term of bed mobility. The bed mobility in the model and in the prototype should be the same at the corresponding depth to achieve similarity in incipient motion of the bed materials and bed load transport in the model and in the prototype.

The bed mobility is defined as-

$$M = \Psi / \Psi_c \qquad (17)$$

Where M= River bed mobility
Ψ= Shield's parameter

The Shield's parameter can be defined as-

$$\Psi=\gamma hS/((\gamma_s-\gamma)D_{50} \quad (18)$$

Where γ= Specific weight of water

γ_s= Specific weight of bed materials

Ψ_c= Critical Shield's parameter whose value can be obtained from Shield's diagram corresponding to the particle Reynold's number.

Particle Reynold's number can be calculated from the following relation-

$$R_*=V_*Ks/\nu \quad (19)$$

Where R_*=Particle Reynold's number

$V_*=(ghS)^{1/2}$,Shear velocity

Particle Reynold's number can be expressed in term of Reynold's number by the following equation-

$$R_*=R(f/8)^{1/2}(Ks/h) \quad (20)$$

In the most of the cases ,flow in the natural river are hydraulically rough. So from the similarity requirements the flow in the model should also be hydraulically rough. Rough turbulent flow in open channel can be defined in the following two ways-

(1) The thickness of viscous sublayer can be defined by-

$$\delta= 11.6*Ks/R_* \quad (21)$$

If $\delta > Ks$, then the flow is said to be hydraulically smooth and in this condition the flow characteristics depends on the viscous force.

If $\delta < Ks$, then the flow is said to be hydraulically rough and in this condition the flow characteristics is independent of the viscous force and entirely depends on the roughness projection.

(2) Turbulent flow can be divided into three different types-

(a) Turbulent flow with smooth bed when $R_* < 5$

(b) Turbulent flow with transitional bed when $5 < R_* <$ RR

Where, RR=60 , for sand grain type of roughness
RR=100 , for any other type of roughness

(c) Turbulent flow with rough bed when $R_* > RR$

In this context it can be mentioned that the flow conditions may be hydraulically smooth even when the surface is rough provided that the roughness projections are small enough to be deeply buried within the viscous sublayer. As the Reynolds number increases and this sublayer shrinks, the roughness projections become a significant factor and the flow enters a transitional stage, culminating when the projections break through the viscous sublayer and dominate the flow behavior, which is then termed as fully developed rough flow.

From similarity requirements and from equation (17) one can write

$$M_p = M_m \quad \text{or}$$

$$\Psi_p/\Psi_{cp} = \Psi_m/\Psi_{cm} \quad \text{or}$$

$$\Psi_m = \Psi_p X \Psi_{cm}/\Psi_{cp} \tag{22}$$

Where Ψ_{cp} and Ψ_{cm} are known and Ψ_p can be calculated from equation (18) and hence from equation (22) Ψ_m can be found out.

Finally the scale factor for the Shield's parameter can be carried out as-

$$\eta_\Psi = \Psi_p/\Psi_m \tag{23}$$

It is clear that this value of η_Ψ should be equal to unity to fulfil the similarity requirements.But due to Reynold's number distortion, the value of η_Ψ may be deviated from unity.

From equation (18) the following important relation can be formulated-

$$\eta_\Psi = \eta_h \eta_S \eta^{-1}_{\gamma s-\gamma} \eta^{-1}_{D50} \tag{24}$$

Where $\eta_{\gamma s-\gamma}=(\gamma_s-\gamma)_p/(\gamma_s-\gamma)_m$,Scale factor for specific weight of bed materials.

In the equation (24); η_ψ,η_h,η_S and η_{D50} are known and hence $\eta_{\gamma s-\gamma}$ can be computed and once the value of $\eta_{\gamma s-\gamma}$ is computed ,the specific weight of the model bed materials can be found out.

3.4 Determination of horizontal length scale

Normally horizontal length scale is selected on the basis of available space for the model and from this point of view the horizontal length scale is given by-

$$\eta_L=L_p/L_m \tag{25}$$

If space for the model permits,the following relation can also be used to determine horizontal length scale-

$$\eta_S=\eta_h/\eta_L \tag{26}$$

Equation (26) gives the value of S_L for the similarity requirements and equation (25) gives the value of S_L for available space limitation.

3.5 Discharge and time scale

Scale ratio for area is given by-

$$\eta_A = \eta_L\eta_h \tag{27}$$

Discharge ratio can be evaluated by-

$$\eta_Q= \eta_A\eta_v \tag{28}$$

From equation (27) and (28) one can write-

$$\eta_Q=\eta_L\eta_h^{3/2} \tag{29}$$

Equation (29) gives discharge ratio for distorted model.

For an undistorted model , $\eta_L=\eta_h$ and equation (29) can be written as -

$$\eta_W = \eta_h^{5/2} \tag{30}$$

Reference time related to sedimentation process is defined by-

$$T_s = Lh/(Q_s/(\gamma_s-\gamma)) \tag{31}$$

Where , Q_s= Rate of bed load discharge and

$$Q_s = 8(\Psi-\Psi_c)^{3/2} \ (\gamma_s-\gamma)^{3/2} \ Ks^{3/2} \ (g/\gamma)^{1/2} \tag{32}$$

From equation (31) and (32) on can write-

$$T_s = (L.h)/(\ 8(\Psi-\Psi_c)^{3/2} \ (\gamma_s-\gamma)^{1/2} \ Ks^{3/2} \ (\ g/\gamma)^{1/2} \tag{33}$$

Converting this into time scale one can write-

$$\eta_{Ts} = \eta_L \ \eta_h \ \eta_{\Psi-\Psi_c}^{-3/2} \ \eta_{\gamma s-\gamma}^{-1/2} \ \eta_{ks}^{-3/2} \tag{34}$$

(assuming $\gamma_p=\gamma_m$ and $g_p=g_m$)

Equation (34) gives time scale related to sedimentation process. Reference time related to strict hydraulic aspects (also called reference time related to water) is defined by-

$$T_w = L \ h \ / \ q \tag{35}$$

where , q= discharge per unit width

= h V

So equation (35) can be written as-

$$T_w = L/V \tag{36}$$

Converting this into time scale one can write-

$$\eta_{Tw} = \eta_L/\eta_V \tag{37}$$

Equation (37) gives time scale related to strict hydraulic aspects.

3.6 Points to be remembered during selection of model scale

1. Froude's number in the model and in the prototype must be the same to hold strict dynamic similarity.
2. Flow in the model must be turbulent to avoid the effect of viscous force in the model.
3. The Weber number in the model should have sufficiently large value to overcome the effect of surface tension. In the river model, this is not too much important because in the turbulent flow condition, the effect of surface tension is always negligible. But in the case of very small model, the effect of surface tension should be investigated.
4. As the mcst of flows in the prototype occur in hydraulic rough condition , flow in the model should also be in hydraulic rough condition.
5. Utmost efforts should be provided to avoid distortion of the model, but when conditions compel to have it, care should be taken to keep distortion at the minimum level.
6. Similarity in the incipient motion of bed material and bed load transport between the model and the prototype should be ensured. In case of mobile bed model, care should be taken to ensure bed load movement.
7. Depth of water in the model should be sufficient for convenient measurements of velocity, scour etc.
8. During selection of model bed material, efforts should be provided to use available material.
9. Horizontal length scale should be selected on the basis of available space to accommodate the model.
10. Size of the model bed material should not be too small otherwise cohesion may develop.
11. Existing water pumping capacity should be considered.
12. Capacity of existing wave generator should also be considered.
13. Last of all, accuracy of the model test results is the fist priority. So one should not be deviated from the similarity requirements to fulfil all the constraints regarding available model bed materials, space available for the model, existing water pumping capacity and capacity of existing wave generator.

4.0 Model calibration

Even after designing a model with great care, it remains absolutely essential for distorted as well as undistorted models to be verified against some field data. Through verification of the model, one can understand the degree of representativeness of the model to its prototype i.e. how much closely the model is representing to its prototype.

A model can possibly be verified in the following ways-

1. Verification of stage-discharge relationship at least at two points.
2. The discharge at which incipient motion occurs should be verified.
3. At least verify some local scour aspects known to occur in the prototype under some flow conditions if verification of sediment load is not possible.
4. Verify flow pattern in the model without any hydraulic structure in the light of observed flow pattern in the prototype in the same condition as that in the model. If necessary , some adjustments in the model roughness need to be made to simulate the flow pattern.

5. Verify flow velocity in the model at some particular points (without hydraulic structure) against authentically measured prototype velocity at the same points.

If model verification disproves similarity with prototype, a trial and error approach will be required to improve the model by adjusting the model roughness or by adjusting any adjustable parameter.

5.0 Conclusions

Hydraulic modelling is an advanced technic to solve the complicated river problems. If the model is designed and verified properly , then it can produce excellent results ,which can be used successfully in the prototype. Also the accurate measurements and the accuracy of data interpretation technic are essential to get fruitful results.

The design of hydraulic model is a technic, which deals with a lot of limitations and possibilities to attain a good solution. Comprehensive thinking about the relative importance of the accuracy of the model test results and the cost involves in the modelling is also essential. As the design of hydraulic river model is completely based on the trial and error approach, through more trial and optimization it is possible to get more better solution.

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APPENDIX - A

List of symbols

Symbol	Meaning
ρ	Density of fluid
D_{50}	Size of bed material
F	Froude number
f	Darcy-Weisbach friction factor
g	Acceleration due to gravity
h	Depth of flow
S	Slope of water surface
ν	Kinematic viscosity
K_s	Roughness height
L	Horizontal length
M	Bed mobility number
m	Subscript indicates model
σ	Surface tension
p	Subscript indicates prototype
Q	Discharge
q	Discharge per unit width
Q_s	Bed load discharge
R	Reynold's number
η	Indicates scale factor
Ψ	Shield's parameter
T_s	Reference time related to sediment transport
T_w	Reference time related to hydraulic aspects
γ	Specific weight of water
γ_s	Specific weight of bed material
V	Average velocity
V_*	Shear velocity
W	Weber number

ALLUVIAL ROUGHNESS

*Mohammad Mohiuddin**

In open channel flow over movable bed ripples begin to grow as soon as particles are in motion and each one beginning as a result of the disturbance created by the ripples immediately upstream. Hence form resistances are developed. Many studies show that sum of the skin resistance and form resistance are close to the total resistance. Furthermore, the form resistance can be predicted from known bed form shapes. A relationship is derived to predict the form resistance as a function of ripples height for laboratory use based on limited data analysis.

Flow over an erodible bed forms different significant features which cause important but complex problem in hydraulic engineering. Resistant offered by that features are important for the channel design, river training, river improvement works, sediment transport rate calculations, discharge calculation etc. In fixed shape artificial channel, carrying little sediment friction factor can be estimated approximately. But for natural channel the friction factor, f, is most important parameter. The interaction of flow between fluid and bed materials results different bed forms. Bed forms are normally composed with Ripples patterns, Dunes with Ripples, Dunes etc. These bed configurations are the result of sediment movement. Bed configuration changes resistance to flow and sediment transport, bed elevation, flow velocity, stage of the river etc.

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The friction factor, f, depends on different quantities:

$$f=\phi(V,D',\mu,\rho,\epsilon,\epsilon',m) \quad (1)$$

Where,

V =flow velocity,
D'=Pipe Diameter,
ρ =Fluid Density,
μ =Fluid Viscosity,
ϵ, ϵ', m=Characteristics of Roughness,
ϵ =Size of the roughness projection,
ϵ'=Arrangement of Roughness Element,
m =Form Factor,

For smooth pipe flow, $\epsilon=\epsilon'=m=o$,them friction factor depends on four quantities, V,D',μ & ρ. These four quantities are expressed by Reynolds number, Re. For rough pipe, in general, the friction factor can be written as

$$f=\phi(VD'\rho/\mu,\epsilon/D,\ \epsilon'/D,\ m) \quad (2)$$

Where, ϵ/D=relative roughness.

Nikuradse proved the validity of relative roughness concept by his tests on sand roughened pipe. Mody had constructed one of the most convenient chart for determining friction factor, which are mostly using for pipe flow.

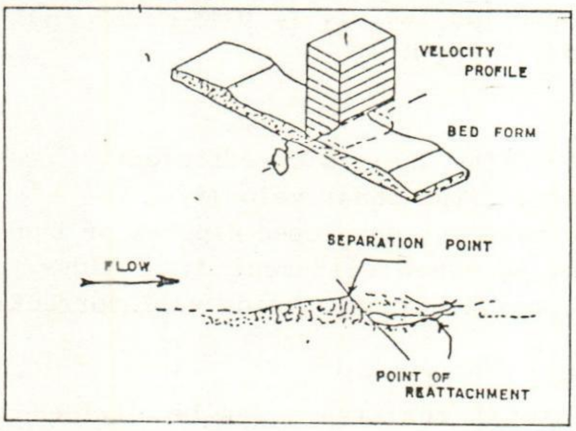

Fig. 1 : Flow Over Ripples formed bed(ASCE, J. Hy. Engg. 109)

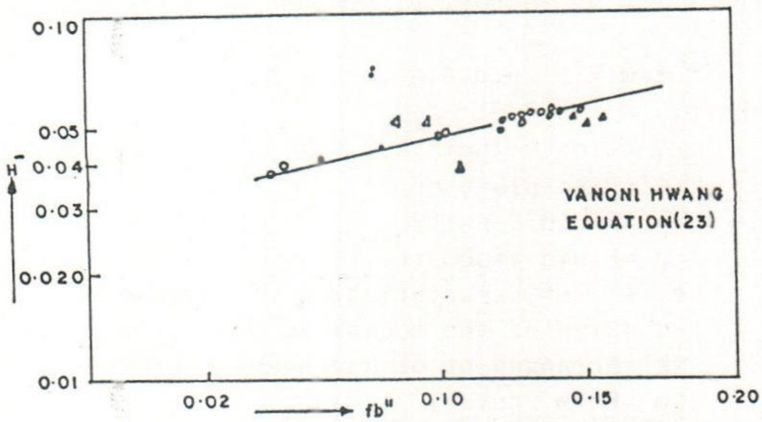

FIG.2 RELATION BETWEEN BED FRICTION fb" AND RIPPLES HEIGHT.

For alluvial stream, following variables determines resistance to flow :

$$\phi(V,D,S,\rho,\mu,g,d,\sigma,\ s,S_p,\ S_r,\ S_c,\ f_s)=0 \quad (3)$$

After simplification the resistance co-efficient can be written as

$$f=\phi(S,D,d,w,\sigma,g,\rho) \quad (4)$$

The mean flow velocity, U, can be expressed in terms of Darcy-Weisbach friction factor, f, by:-

$$U=(8/f)^{1/2}*(g*R*S)^{1/2} \quad (5)$$

Where

$(8/f)^{1/2}$=The Chezy's Co-efficient,

$(g*R*S)^{1/2}$=The Shear velocity,

Since sand movement developed Ripples or Dunes the roughness of the bed is somewhat different from sides. Hence Einestine & Jhonson proposed following side wall correction:-

$$U=(8/f_b)^{1/2}*(g*R*S)^{1/2} \quad (6)$$

Since the total resistance can be divided into two different individual resistance, similarly, bed shear stress can be divided into two different shear stress.

1) Bed shear stress developed from equivalent grain roughness, T_b'

2) Bed shear stress developed from form drag of Ripples or other bed forms, τ_b'

$$U=(8/f_b')^{1/2}*(g*R_b*S')^{1/2} \tag{7}$$

$$U=(8/f_b'')^{1/2}*(g*R_b.s'')^{1/2} \tag{8}$$

When bed forms occurs, a form drag must be added to the surface drag or to skin friction. Meyer-Peter & Mullar (1948) developed an idea that a certain friction of the energy is consumed to overcome the resistance due to surface drag and the remainder is used to overcome the form drag. If S' corresponds to the friction of the energy slope required to overcome surface drag and S" the friction factor required to overcome form drag, it is possible to write the total slope,

$$S=S'+S'' \tag{9}$$

The value f' is determined from a graph of Nikuradse date friction factor against Reynolds Number for several values of Relative Roughness. After solving equation (7) & (9), f" is calculated from equation (8).

Vanoni and Hwang developed an idea for the form friction factor, fb". Their calculation is based on in terms of hydraulic quantities and bed geometry. They used concentration of bed forms, A_s/A and characteristics length of bed form height, h.

$$f_b''=f(R_b/(A_s/A)*h) \tag{10}$$

Vanoni and Hwang introduced exposure parameter in the equation (10)

$$f_b''=f(R_b/e*\hat{H})$$

Where,

$R_b/e*H$=Relative roughness

H=Mean height of Dunes of Ripples

According to Vanoni and Hwang the relation between f_b'' and

Rb/e*H are

$$1/f_b''=3.5 \log (R_b/e*H) - 2.3 \qquad (11)$$

Co-efficient of drag has a significant role in form friction factor

$$C_d=F_p/o.5*p*v'^{5}*A \qquad (12)$$

The pressure drag F_p is integration of pressure distribution over the ripples or dunes, hence

$$F_p= \int_0^{S_c} p\sin\theta_u \, ds - \int_{S_c}^{S_t} \sin\theta_d \, ds \qquad (13)$$

The form shear stress τ" & form friction factor f" is known by

$$\tau''=F_p/L \qquad (14)$$

Where L= Dunes or Ripples length.

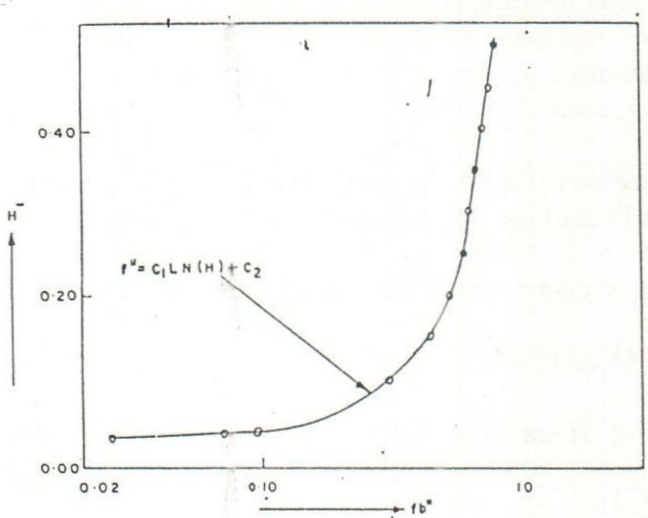

Fig. 3 : Relation between bed friction fb" and riples height

$$f''=8*\tau''/pv'^{5} \qquad (15)$$

Hence from the above equation,

$$f''=4*C_d*A/L \qquad (16)$$

Several investigator proposed relation for co-efficient drag, Cd with relative steepness (H/D)

$$C_d=m*(A/D)^{3/8} \qquad (17)$$

Where,

H= Ripples or Dunes height,
H/D=Relative Steepness,
D=Water depth,
m=A co-efficient dependent only on the bed form steepness ratio.

Shen et. al (1990) determined the relation between C_d and the relative roughness by using two different equations:

$$C_d=4/9*(H/D)^{3/8} \text{ for } H/D \leq 0.35 \qquad (18)$$
$$C_d=3/2*(H/D)^{3/2} \text{ for } H/D>0.35 \qquad (19)$$

But Muhammad and Mahmood used finite element analysing method to determine ϕ value and co-efficient m value.

$$f''=\phi(D/L)*(A/D)^m \text{ for } D/L > 0.5 \qquad (20)$$

By using $\phi=4.9*(D/L)^{1.477}$ (from graphical analysis) equation (20) can be written as

$$f''=5*(A/L)^{3/2}*(A/D)^{1/6} \text{ for } D/L > 0.5 \qquad (21)$$

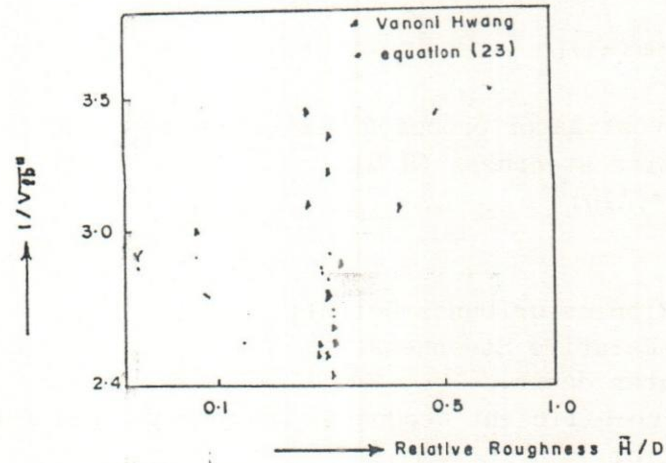

Fig. 4 : Relation between $1/(f_b'')$ and Relative Roughness.

Vanoni and Hwang showed that the roughness length eH for ripple bed is more appropriate than classical equivalent sand roughness, ks. The concentration of ripples which is expressed by e shows better co-relation with bed friction factor f" than ripples steepness factor H/L. But Muhammad and Mahmud expressed different idea about bed friction factors. For geometrically similar bed form ripples, the form friction factor depends on the steepness ratio, H/L and relative roughness H/D and the form friction factor does not depends on the absolute size of the bed form.

It is tedious and sometimes complicated measuring of ripples length and ripples surface projected area instead of height of the ripples. Simply, bed friction factor can be expressed as following logarithmic equation with some co-efficient

$$f''=C_1*Ln(\text{Ripples height})+C_2 \qquad (22)$$

With limited data analysis C_1 & C_2 are determined. The values of C_1 & C_2 are 0.28193 and 0.9712 respectively.

$$f''=0.28193*Ln(H)+0.9712 \qquad (23)$$

For the first approximation of bed form, ripples height, H in ft. can be calculated by following logarithmic equation.

$$H=K_1*Ln(U/(g*D)^{1/2} + K_2 \qquad (24)$$

Above equation is related with the mean velocity and with mean water depth. This equation can be used for the first approximation of the ripples height with the following limitation :

$$K_1=0.01026 \; ; \; K_2=0.0715 \text{ when } U/(g*D)^{1/2} \le 0.10$$
$$K_1=0.01026 \; ; \; K_2=0.085 \text{ when } 0.10\le U/(g*D)^{1/2} \le 0.30$$

Also equation (24) is derived from very limited number of data. It is observed that by increasing the value of F_r, an increasing in ripples height. But the increasing ratio is not as same as increasing of Fr value. If the Froud number increases two times but the ripples height are only 15% higher. It seems that increasing tendency of ripples height is less if velocity increases with constant depth. Hence for higher velocities, equation (24) shows lower ripples height than expected one. Both equation (23) & (24) can be used for primary idea of friction factor and ripples formation in laboratory study.

Fig. 2 shows close relationship in logarithmic scale between ripples height and form friction factor. Except a few scattered point most of the point from Vanoni and Hwang data shows good agreement with equation (23). Equation (23) is the lower part of fig. 3 which is nearly straight one to $f_b''=0.15$ in fps unit beyond which it is assumed that equation (23) is not valid. That means, upper steepened part of the fig. 3 is not graphical

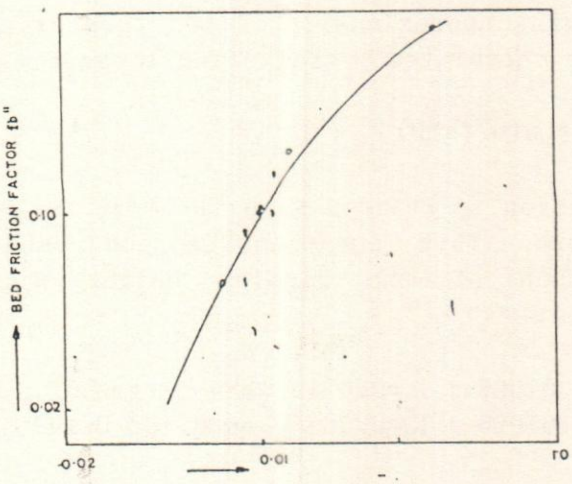

Fig. 5 : Relation between relative steepness $\bar{H}/L$ and bed friction factor, f_b''

clarification of equation (23) .It is assumed that dotted line may be idealized one for better explanation of equation (23). Equation for the dotted line will be determined from extensive study. Beyond f"=0.15 ripples height shows no significant influence on the form friction factor, which is a unrealistic behavior of this equation. So, solid line of fig. 3 which is beyond f"=0.15 has no significant explanation for describing bed friction factor. Within H=0.2 ft. ripples equation (23) shows good explanation in fig.3.

Conclusion

(1) If the ripples height is known bed friction factor can be determined from equation (23).
(2) Since most of data are used from different laboratory flume test, hence wide application is not possible . It needs further elaborate investigation.
(3) From equation (24) it is observed that for higher Froude condition the first term tends to zero and than the maximum value is only the right side co-efficient. Hence it is not possible to conclude that maximum ripples height will be

0.085 in ft. So, for higher Froude condition ripples height is quite unpredictable by this equation.

(4) Equation (23) is independent on equation (24). Thus, by measuring ripples height or dunes height from alluvial channel or from laboratory model bed, form friction factor can be determined from equation (23).

Appendix-I

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Appendix-II

Notation and its meaning

A= total bed area over which As is measured

A_s=horizontal projection of leefaces of the ripples or dunes

C_D=pressure drag co-efficient

e=exposure parameter

F_r=Froude number

f=Darcy-Weisbach friction factor

f_b''=form bed friction factor

f_b'=friction factor for sand grain roughness
g=gravitational acceleration
h=characteristics length dimension of the bed forms
H=mean dune or ripples height
d=mean diameter of sand
L=ripples or dune length
p=local surface pressure
S=channel slope
S'=slope due to grain roughness
S"=slope due to bed forms
V=mean velocity
D=depth of flow
τ_b'=shear stress resulting from sand grain roughness
τ_b"=shear stress resulting from form drag
R_b=hydraulic radius of the bed
τ"=bed shear stress due to form resistance
Q_d=down stream angle of bed form with respect to x direction
$Q_?$=upstream angle of bed form with respect to x direction
ρ=fluid density
μ=apparent dynamic viscosity of the water sediment mixture
σ=measure of size distribution of bed materials
S_p=shape factor of the particles
S_g=shape factor of the reach of the stream
S_c=shape factor of the reach of the stream
S_c=shape factor of the cross section of the stream
f_s=seepage force in the bed of the stream

PROBABLE EFFECTS AT DOWN STREAM OF A BARRAGE DUE TO IMPROPER GATE OPERATION

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Operation of a barrage is guided by certain rules which permits a tolerable effects at down stream. But enormous effects may be occurred due to any deviation from operational rule. During the dry season, quick gate closing creates a quick fall in the water level in the river (Stream). As a result the ground water tables may fall due to effluent characteristics of the river (stream) where an effluent river (stream) is one receiving ground water discharge (base flow). When the ground water depletes, the surface of the unconfined aquifer becomes dry more and more because of ground water flow from the ground water storage to the river.At the same time soil loses it's available moisture due to evapotranspiration. Then the shallow rooted trees and crops start to die due to lack of available moisture in the root zone. The surface temperature also becomes very high due to high reflection of incoming solar radiation by the dry soil. This phenomenon is known as drought which also called the desertification of the land. Then no irrigation water is available both from the ground water and surface water. As a result, production of food reduces tremendously which creates famine. Due to the groundwater movement from the groundwater storage to the river (Stream), the soil of the bank becomes very loose and the bank becomes unstable. As a result bank erosion takes place at the same time. The more movement of the ground

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water to the river, the more erosion takes place on the bank. Another effects during dry season due to sudden gate closing is rising of the bed level of the river and also formation of small island (Char). When the gate closes suddenly the velocity of the flow at downstream also falls abruptly which stimulates the sedimentation on the bed of the river because the settling velocity of the grain is become more than the flow velocity. The other effects due to sudden gate closing is saline intrusion if the river has a direct contact with the sea. Because when the water level falls in the river, the discharge of the river also falls at the same time and as a result more sea water (Saline water) begins to enter into the river due to much variation between the river water level and the tide water level of the sea which provides less obstruction & much space for tidal saline water. This saline water is harmful for fresh water fish. At the same time, saline water also starts to enter into the ground water storage which creates problems for irrigation and drinking water. The salts reduce the osmatic activity of the plants, thus preventing the absorption of nutrients from the soil, and reduce the permeability of the soil. Also with constant irrigation with saline water, the soil becomes plastic & sticky during wet and form crust & clods on drying.

During the monsoon season, the ground water table starts to rise due to percolation and influent nature of the river where influent river is one discharges water to the ground water storage. Although water flows from the river to the ground water storage, the river also receives water from the surface run off of the catchment. At the same time, if the gate of the barrage is opened suddenly the water level at down stream is increased abruptly which increase the discharge and velocity of flow. The direction also changes due to formation of small island (Char) along the course of the river. As a result the water heats the bank with high velocity which makes the erosion on the bank. An excess amount of water coming through the gates also creates sudden flood which damages lot of lives and crops.

Impact on environment starts due to all above effects which are directly related to improper gate operation of the barrage. So proper gate operation is a precondition of maintaining environmental balance at down stream of the barrage

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A STUDY FOR CORRELATIONS BETWEEN UNCONFINED COMPRESSIVE STRENGTH WITH SOME PHYSICAL PROPERTIES OF CLAYEY SOILS OF BANGLADESH

Uma Saha*

Abstract

The unconfined compressive strength (q_u) , wet density (γ_w), dry density (γ_d),liquid limit (LL),Plastic limit (PL) and moisture content (w) are used in compression characteristics of sub surface soils under structural load and these parameters are usually determined from the conventional laboratory test. In this paper the unconfined compressive strength (q_u) of the soils of different location of Bangladesh have been collected form laboratory test and some correlations have been developed with the physical properties of these soils .

Introduction

Knowledge of the shear strength of the soil is required for various purposes like construction of dams, embankments, retaining walls,bridges and buildings etc. There are many kind of tests usually used to determine shear strength like direct shear test,unconfined compression test,triaxial compression test and vane shear test. Unconfined compression test is one of the simplest, quickest and economical mens than any other shear test on cohesive soils. Portable unconfined compression tests are now used in this country and abroad for strength determinations of construction sites. Hence it will be advantageous if some correlations can be established on unconfined compressive strength (q_u) with physical properties of soils which can be easily approximated using this (q_u) value with less efforts and expenditures.

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Literature review

The unconfined compression test is a special case of triaxial compression test in which alround i.e. confining pressure is zero. Due to the absence of such confining pressure the uniaxial test is called the unconfined compression test. Here an axial load is applied to cause shear failure of the clay which id dependent upon the cohesion. The unconfined compression test, is a test at natural water content. The unconfined compressive strength is denoted by (q_u) (Punmia, 1987).

The unconfined compression test is generally applicable to saturated clays for which the apparent angle of shearing resistance(Φ_u) is zero. But from this test on c-ϕ soils cohesion and shearing angle may also be calculated which is briefly discussed later.

The unconfined compressive strength (q_u) is calculated on the basis of changed cross-sectional area at failure (Punmia,1987).

Presentation of results

In this paper clayey soils of the different locations of Bangladesh have been used which is tested in River Research Institute (Soil Testing Report, 1991-1992). The physical properties w,γ_w γ_d,LL,PI,and I_L of the clays have been presented in Table-1. Actual values of q_u ,w,γ_w ,γ_d, and I_L have been presented in Figure.1(a), 1(b), 1(c), 1(d). Unconfined compressive strength (q_u) have been plotted against w,γ_w ,γ_d and I_L in arithmetic scale & the best fit curves are drawn. The relations q_u with γ_w and γ_d are such that with increasing the values of q_u the values of γ_w andγ_d increases exponentially. On the other hand the nature of q_u vs. W curves such that q_u decreases exponentially with the increase of W. q_u vs. I_L gives a straight line with the negative slope giving the equation.

$$q_u = -44 I_L + 68 \quad (1)$$

Here, I_L = Liquidity index (which is obtained from Liquid Limit and Plastic limit)

Therefore, performing U.C.T. any location of Bangladesh this properties can be easily approximated using those correlations.

If this soil is c-ϕ, then performing U.C.T. c and ϕ easily can be calculated by knowing the angle of failure plane. The sketch of the failure pattern is shown in figure-2. From this following equation c and ϕ is determined :

$$q_u = 2c \tan\alpha \qquad (2)$$

where, $\alpha = 45^0 + \phi/2$ = Angle of failure plane (Punmia,1987).

Discussion & conclusion

This paper briefly gives a general idea of the correlations between unconfined compressive strength with some physical properties of the clayey soils of Bangladesh and c, ϕ values of the c-ϕ soils can also be approximated from the failure pattern of the sample . Thus after performing only U.C.T. in the field and using this q_u value we can approximate γ_w, γ_d, W from figure-1(a), 1(b), 1(c) and using the equation (1) I_L can be calculated. The cited correlations obtained from limited number of test results. So, extensive test results of study on similar papers is essential .

Reference

1. Punmia B. C. (1987), Soil Mechanics and Foundation, 10th edition, Published by Rajinder Kumar Jain, (422-495).

2. Soil Testing Report, River Research Institute, Faridpur (1991-1992).

TABLE-1: Unconfined compressive strength, wet unit weight, dry unit weight, moisture content, liquid limi, plasticity index and liquidity for clayey soils.

Sl. No.	q_u in kN/m^2	γ_w in kN/m^3	γ_d in kN/m^3	W in % (avg)	LL in %	PI in %	I_L	Classification
1	50.21	17.67	12.00	42.27	83.00	45.00	0.094	Medium stiff organic Clay, Trace fine sand, Medium to high plastic
2	48.30	17.21	11.94	44.08	67.00	34.00	0.325	Medium stiff Clay, trace organic, trace fine sand, high plastic
3	95.55	18.33	12.74	44.22	59.00	32.00	0.538	Stiff Clay, trace fine sand, high plastic
4	56.47	17.80	12.75	39.58	60.00	30.00	0.319	Medium stiff Clay, trace fine sand, High plastic
5	48.86	18.05	12.57	43.57	83.00	44.00	0.103	Medium stiff Clay (organic), trace fine sand, Medium to high plastic plastic.
6	121.30	18.98	17.34	32.32	53.00	26.00	0.204	Very stiff Clay, trace fine sand, Medium plastic
7	95.41	18.85	14.05	34.18	52.00	26.00	0.314	Medium stiff Clay, trace fine sand, Medium plastic
8	64.32	16.87	12.31	37.00	64.00	30.00	0.100	Stiff Clay, trace fine sand,. trace organic, High plastic
9	35.37	17.25	13.04	32.33	54.00	28.00	0.226	Soft Clay, trace fine sand, High plastic
10	172.60	19.46	15.25	27.56	51.00	25.00	0.062	Stiff Clay, trace fine sand, High plastic
11	34.12	16.77	11.50	45.89	58.00	30.3	0.596	Soft Clay, trace fine sand (wooden piece), High plastic
12	37.95	19.80	15.89	24.66	39.00	19.00	0.205	Soft Clay, trace fine sand, trace small bricks, Medium plastic
13	47.14	18.44	13.91	32.61	54.00	28.00	0.236	Soft Clay, trace fine sand, High plastic
14	191.59	20.63	16.46	25.34	42.00	20.00	0.167	Stiff Clay, some fine sand, Medium plastic
15	13.83	16.41	8.90	54.00	60.00	32.00	0.815	Soft Clay, trace fine sand, trace organic matter, High plastic
16	54.99	18.24	13.07	39.50	54.00	27.00	0.463	Medium stiff Clay, trace fine sand, High plastic
17	26.67	18.05	13.09	37.83	56.00	30.00	0.394	Very soft Clay, trace fine sand, High plastic
18	56.61	19.31	14.61	31.37	57.00	29.00	0.116	Medium stiff Clay, trace fine sand, High plastic
19	56.02	17.46	13.26	31.66	55.00	27.00	0.135	Medium stiff Clay, trace fine sand, High plastic

q_u VS γ_w

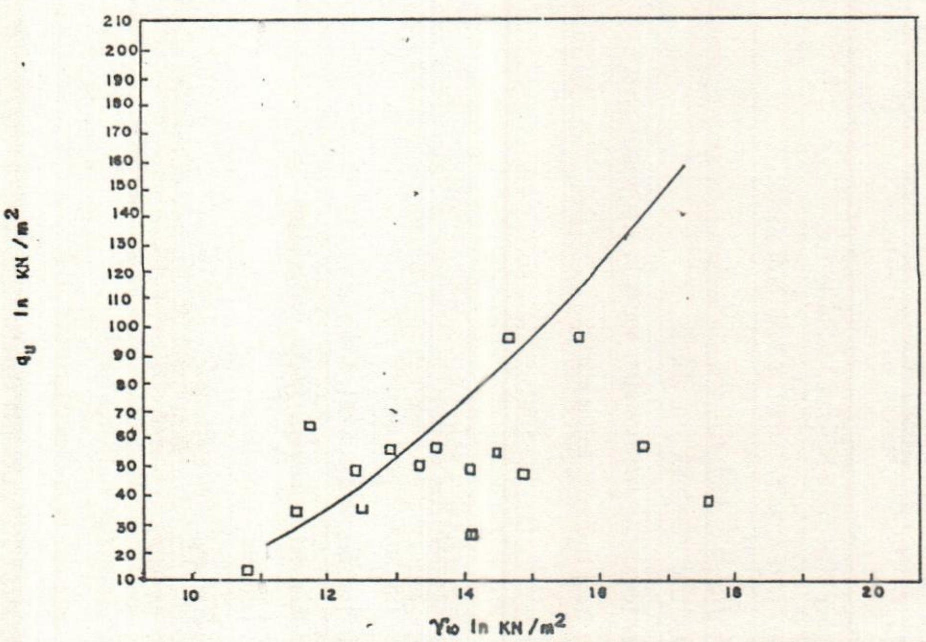

Fig.(Ia): Variation of unconfined compressive strength (q_u) with change in wet unit weight

q_u VS γ_d

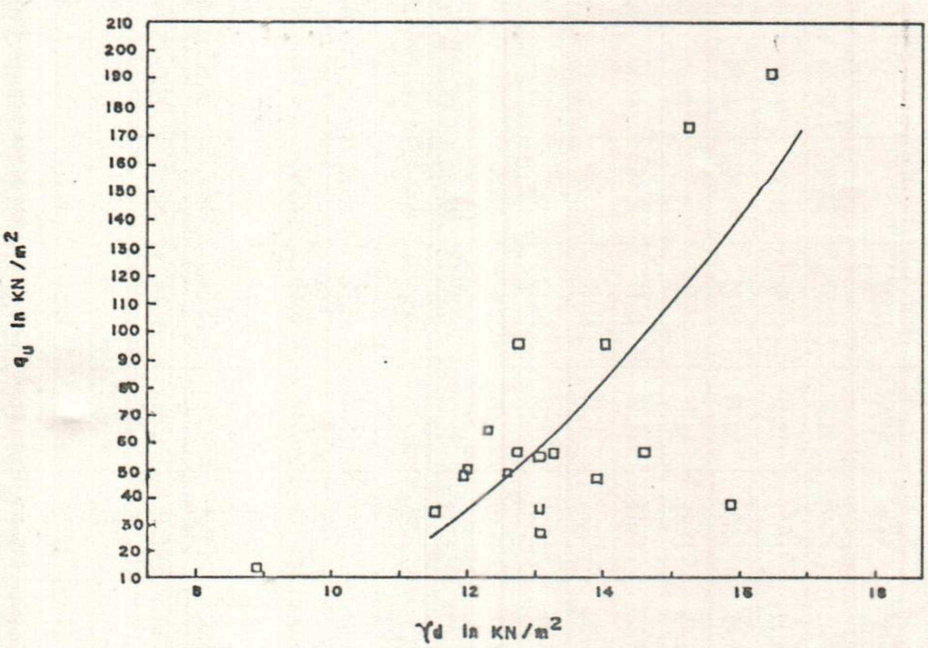

Fig.(Ib): Variation of unconfined compressive strength (q_u) with change in dry unit weight

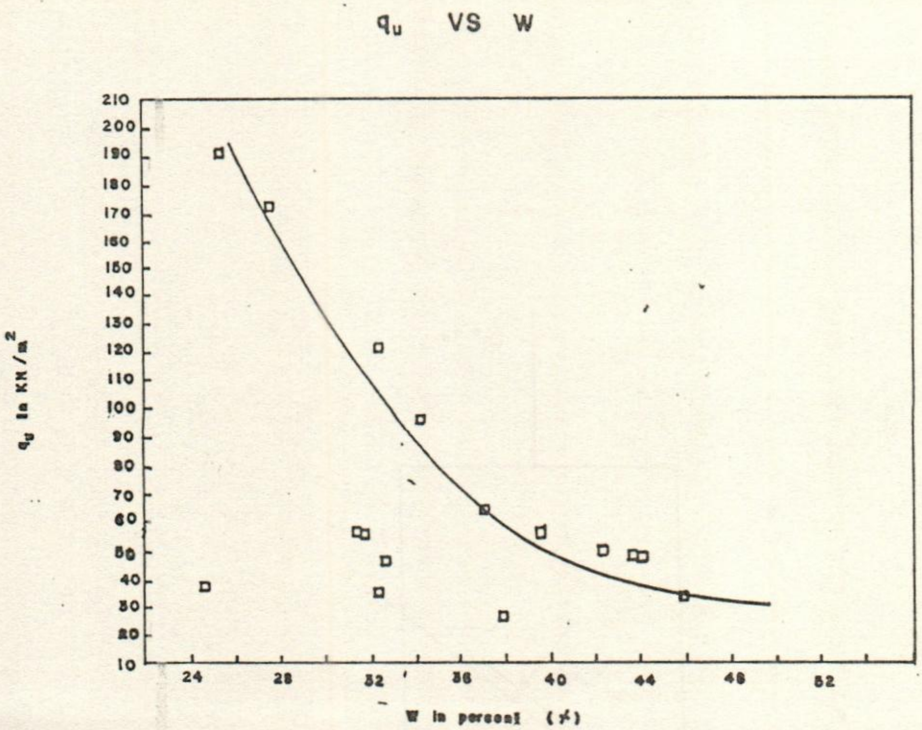

Fig. 1(c); Variation of unconfined compressive strength (q_u) with change in moisture content

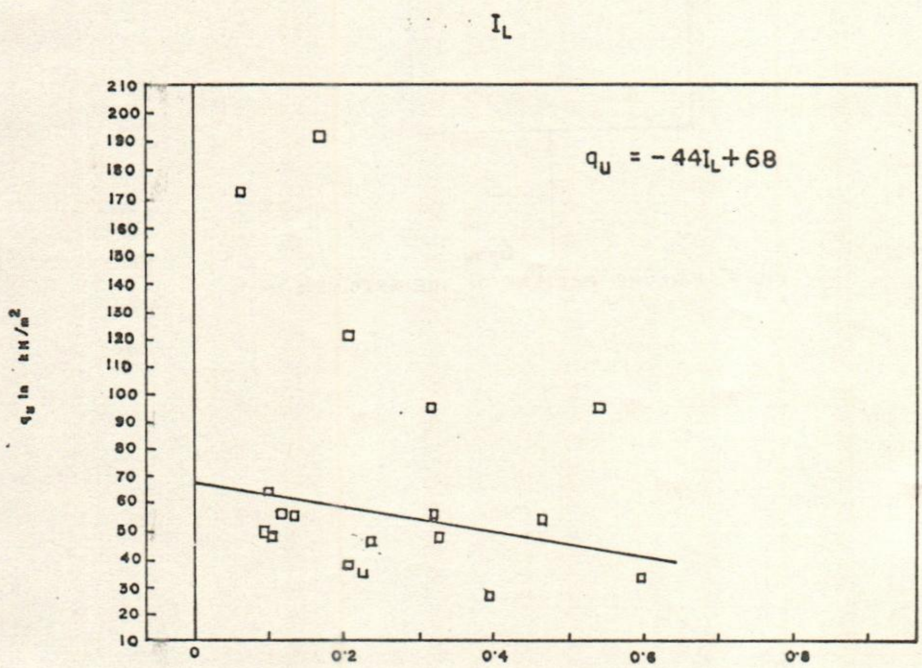

Fig. (1 d). Variation of unconfined compressive strength (q_u) with change in liquidity index

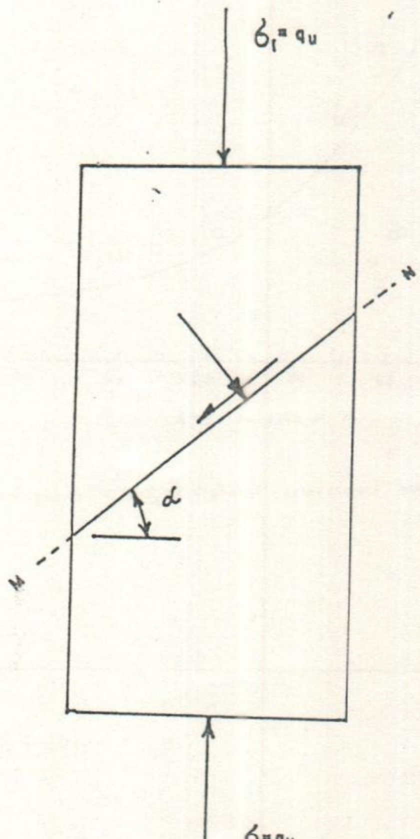

FIG-2. FAILURE PATTERN OF THE SPECIMEN

BANKLINE MOVEMENT OF ARIAL KHAN RIVER AND DEVELOPMENT OF NON-DIMENSIONAL CORRELATION

*A.K.M. Ashrafuzzaman**

Abstract

The Arial Khan is a meandering river. It flows through the district of Faridpur and Madaripur. It is worth mentioning that severe bank erosion and shifting of the Arial Khan endanger the Bhanga-Mawa highway and agricultural lands. The present technical paper includes determination of bankline shifting and establishing relationship between non-dimensional parameters. The shifting of the bankline of the Arial Khan between 1972 and 1991 determined by superimposing the landsat imageries of these years showed maximum lateral and maximum longitudinal shifting at Bagmara and at Chandra by 3.5 km left and 2.5 km downstream respectively. From the non-dimensional plots, it was observed that shape(W/D) decreases with the increase of flow Froude number($V/\sqrt{(gD)}$) and $Q/(\nu D)$ whereas flow Froude number($V/\sqrt{(gD)}$) increases with the increase of $Q/(\nu D)$.

Introduction

The Arial Khan has tremendous shifting tendency. It has two parts, one of them is called the Arial Khan Upper and the other is the Arial Khan Lower, both of them are distributaries of the Padma. Both the parts of the river are originated from the Padma near Chowdhury Char and Dubaldia respectively.

The length of the Arial Khan Upper and the Arial Khan Lower is about 70 km and 125 km respectively. The Arial Khan Upper meets the Arial Khan Lower at Shalmaback in the district of Madaripur. Then the river extends towards the downstream direction to meet various rivers such as the Naya Banga,

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the Babuganj, the Kirtonkhola etc and finally meets the Biskhali river which discharges into the Bay of Bengal through the Sundarban forest [3].

Bankline movement from 1972 to 1991

Using landsat imageries, bankline movement between 1972 to 1991 was determined at various places. Fig.1 shows the superimposed plan maps of the Arial Khan. It was observed from this figure that various amounts of shifting had occurred at various places. The major changes happened due to the bankline movement of the Arial Khan were as follows: (i) shifting of bends at Chandra and at Kamar Char by both laterally and longitudinally (ii) shifting of confluence of the Arial Khan Upper and the Arial Khan Lower from Husnabad to Shalmaback by both laterally and longitudinally (iii) lateral shifting of bankline from Bagmara to Kolokopa by forming a cut off and (iv)lateral shifting of the river which affects the Bhanga-Mawa highway. The bankline at Baparirkandi and at Char Chandra was shifted laterally by 0.5 km right and 0.6 km left respectively. The bend at Chandra was shifted laterally by 1 km left and longitudinally by 2.5 km downstream.

The lateral shifting of the Arial Khan endangers the Bhanga-Mawa highway. A lateral shifting of about 0.5 km left was observed during the period from 1972 to 1991 at this place. At the place near Bagmara, the bankline was shifted to Kolokopa laterally by 3.5 km left by forming a cut off.

The confluence of the Arial Khan Upper and the Arial Khan Lower at Husnabad was observed to be shifted from Husnabad to Shalmaback laterally by 5.0 km right and longitudinally by 3.0 km upstream. The bankline movement by 0.75 km left was observed for both at Kashikandi and Alipur. At the bend near Jafrabad a lateral shifting of about 0.875 km left had occurred. At the bends near Balahar Char, Char Kamarkandi, Gopalpur, Kajirhat, Zajirabazar, no appreciable shifting had taken place.The channel width was found to increase by about 0.5 km left for both the bends at Madrabazar and Mohuruddir Char. The bend near Kamar Char was shifted laterally by 2.5 km left and longitudinally by 0.25 km upstream [1].

It may be noticed from the foregoing discussion that the maximum lateral bankline shifting had taken place at Bagmara by about 3.5 km left and the maximum longitudinal bankline shifting had taken place at Chandra by about 2.5 km downstream.

Non-dimensional correlation between flow Froude number ($V/\sqrt{(gD)}$), shape(W/D) and $Q/(\nu D)$

The non-dimensional parameters such as shape(W/D),flow Froude number($V/\sqrt{(gD)}$) and $Q/(\nu D)$ were calculated for various years at Chowdhury Char(a discharge and water level station of the Arial Khan). The value of kinematic viscosity(ν) and acceleration due to gravity(g) was taken to be $1.116*10^{-6}$ m^2/s and 9.81 m/s^2 respectively.

Graphical relationships between flow Froude number($V/\sqrt{(gD)}$) and $Q/(\nu D)$ were developed for 1983 to 1988 at Chowdhury Char. The relationships are shown in Fig.2 to 7. These figures show a general trend of increased flow Froude number with the increase of $Q/(\nu D)$. Power type regression equations describing the trend of the variation of flow Froude number with the change of $Q/(\nu D)$ are summarized in Table-1. This table shows that the correlation existing between flow Froude number and $Q/(\nu D)$ at Chowdhury Char is quite good.

Non-dimensional relationships between shape(W/D) and $Q/(\nu D)$ were developed at Chowdhury Char for various years. The relationships are shown in Fig.8 to 12. From these plottings, it was observed that shape(W/D) decreases with the increase of $Q/(\nu D)$ i.e., the river is deepened as the discharge increased. Power type regression equations between W/D and $Q/(\nu D)$ are summarized in Table-2. This table reveals that these two parameters are well correlated.

Graphical relationships between shape(W/D) and flow Froude number($V/\sqrt{(gD)}$) were also developed using data at Chowdhury Char for the years 1983 to 1988 shown in Fig.13 to 18. From these plottings, it was revealed that shape increases with the decrease of flow Froude number. Power type regression equations were developed between these two parameters and are summarized in Table-3. This table shows that a good correlation exists between shape and flow Froude number [1]. Similar type of study was conducted by Hossain (1989) for the Ganges considering the data of all available cross-sections in the study reach and found a similar trend [4].

It may be worthwhile to point out at this stage that the non-dimensional parameters used in this study have a potential application in scale model study of river which is essential before implementing any river stabilization or flood control works.

Notations

- Q water discharge
- V average flow velocity
- W channel width
- g acceleration due to gravity
- D average flow depth
- v kinematic viscosity
- r co-efficient of correlation

References

1. Ashrafuzzaman,A.K.M.(1992),"A study on the geometric aspects of Arial Khan river," M.Engg.Thesis,Dept.of WRE,BUET,Dhaka.
2. Begum,Y.(1988),"Correlation of non-dimensional parameters for water courses in alluvium," M.Sc.Engg.Thesis,Dept.of WRE,BUET,Dhaka.
3. BWDB(1987-88),"Morphological survey data of river Mahananda and Arial Khan," Report No.3.
4. Hossain,M.M.(1989),"Geomorphic characteristics of the Padma up to Brahmaputra confluence," Final Report,R 02/89,IFCDR,BUET.

TABLE-1

Non-dimensional correlation between flow Froude number($V/\sqrt{(gD)}$) and $Q/(\nu D)$ for the Arial Khan Upper at Chowdhury Char.

Regression equation	Co-efficient of correlation(r)	Data used for the years
Y = 6.40*10-8X0.747	0.989	1983
Y = 1.16*10-7X0.719	0.982	1984
Y = 3.29*10-6X0.540	0.792	1985
Y = 4.05*10-7X0.652	0.992	1986
Y = 4.62*10-7X0.640	0.948	1987
Y = 3.88*10-7X0.637	0.987	1988

Where, $X = Q/(\nu D)$ and $Y = V/\sqrt{(gD)}$

TABLE-2

Non-dimensional correlation between shape(W/D) and $Q/(\nu D)$ for the Arial Khan Upper at Chowdhury Char.

Regression equation	Co-efficient of correlation(r)	Data used for the years
Y = 124064.979X-0.386	-0.932	1983
Y = 17361.880X-0.278	-0.936	1984
Y = 37753.260X-0.315	-0.855	1986
Y = 30971.445X-0.318	-0.888	1987
Y = 20431.240X-0.287	-0.910	1988

Where, $X = Q/(\nu D)$ and $Y = W/D$

TABLE-3

Non-dimensional correlation between shape(W/D) and flow Froude number($V/\sqrt{(gD)}$) for the Arial Khan Upper at Chowdhury Char.

Regression equation	Co-efficient of correlation(r)	Data used for the years
$Y = 98.753X^{-1.605}$	-0.941	1983
$Y = 787.500X^{-2.036}$	-0.894	1984
$Y = 50.000X^{-1.400}$	-0.722	1985
$Y = 124.300X^{-1.572}$	-0.807	1986
$Y = 316.984X^{-1.907}$	-0.839	1987
$Y = 463.094X^{-1.940}$	-0.781	1988

Where, $X = W/D$ and $Y = V/\sqrt{(gD)}$

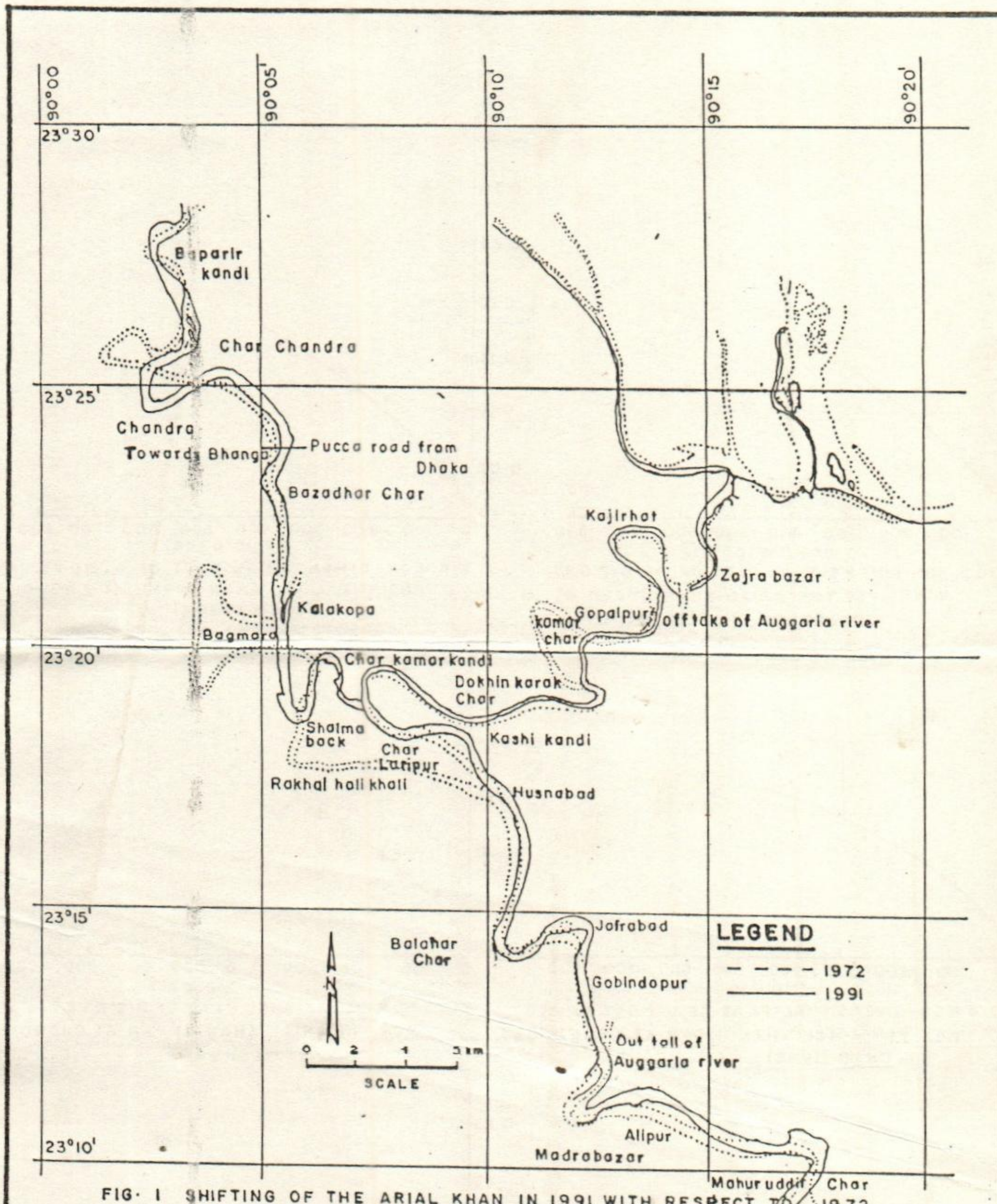

FIG. I SHIFTING OF THE ARIAL KHAN IN 1991 WITH RESPECT TO 1972

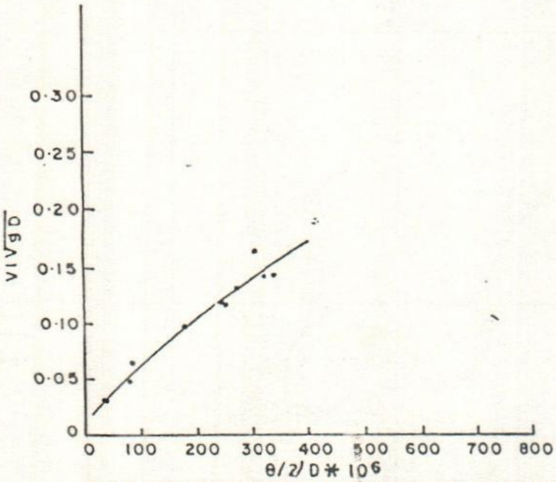

FIG.2. NON-DIMENSIONAL PLOT OF OF θ/2/D VS. V/√gD FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1983)

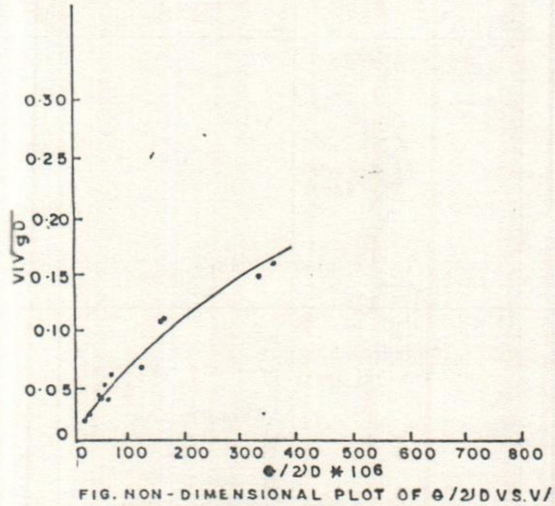

FIG. NON-DIMENSIONAL PLOT OF θ/2/D VS. V/√gD FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1984)

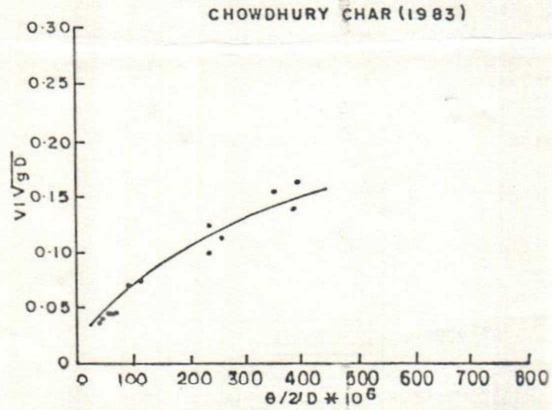

FIG. 4 NON-DIMENSIONAL PLAT OF θ/2/D VS. V/√gD FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1985)

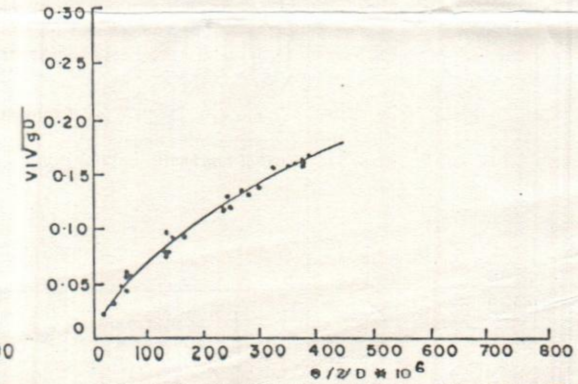

FIG.5 NON-DIMENSIONAL PLAT OF θ/2/D VS. V/√gD FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1986)

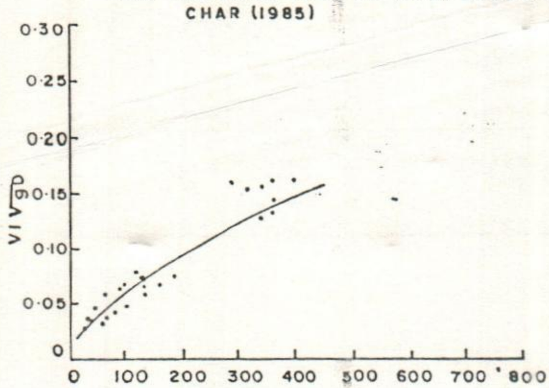

FIG. 6 NON DIMENSIONAL PLOT OF θ/2/D VS. V/√gD FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1987)

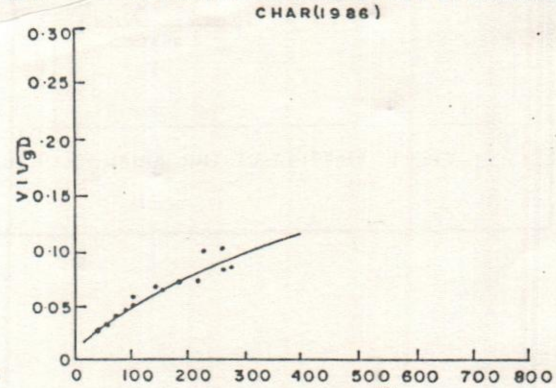

FIG. 7 NON DIMENSIONAL PLOT OF θ/2/D VS. V/√gD FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1988)

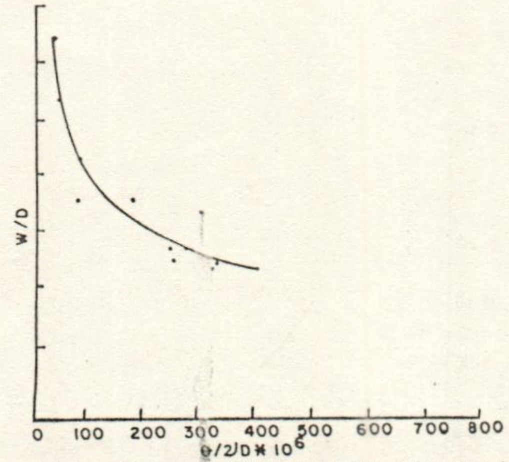

FIG. 8 NON-DIMENSIONAL PLOT OF W/D VS. θ/2D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1983)

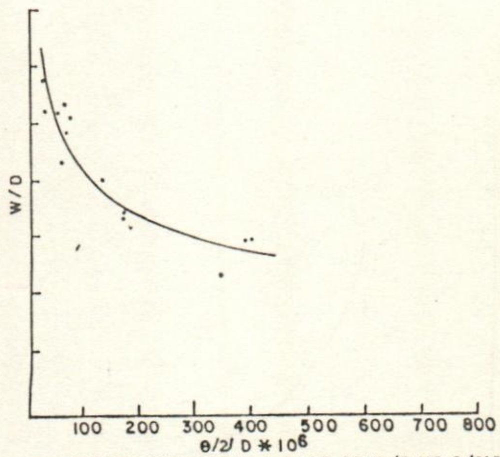

FIG.9 NON-DIMENSIONAL PLOT OF W/D VS. θ/2D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1984)

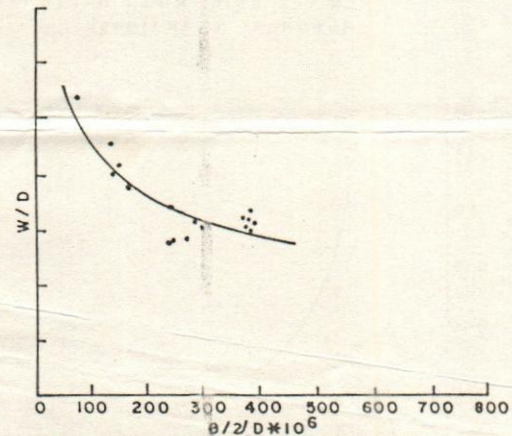

FIG.10 NON-DIMENSIONAL PLOT OF W/D VS. θ/2D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1986)

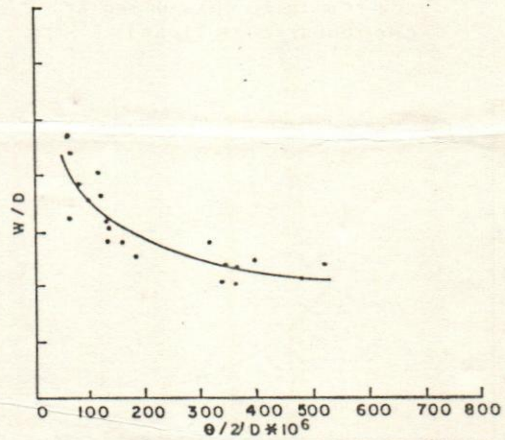

FIG. 11 NON-DIMENSIONAL PLOT OF W/D VS θ/2D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1987)

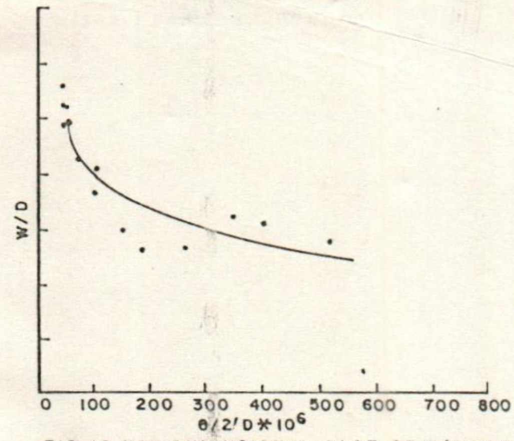

FIG. 12 NON-DIMENSIONAL PLOT OF W/D VS θ/2D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1988)

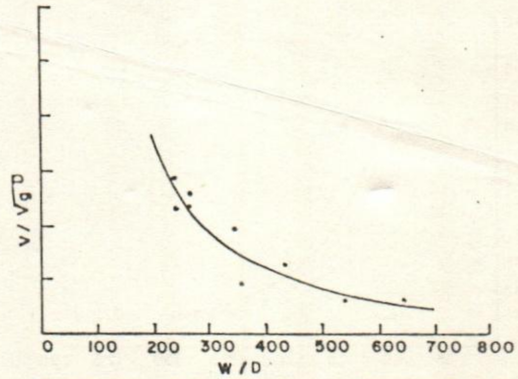

FIG.13 RELATION BETWEEN $V/\sqrt{gD}$ AND W/D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1983)

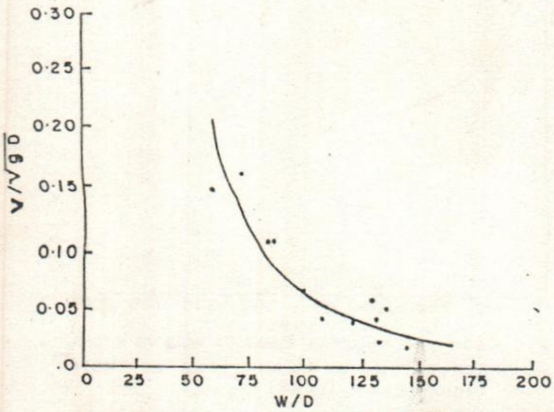

FIG.14 RELATION BETWEEN $V/\sqrt{gD}$ AND W/D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1984)

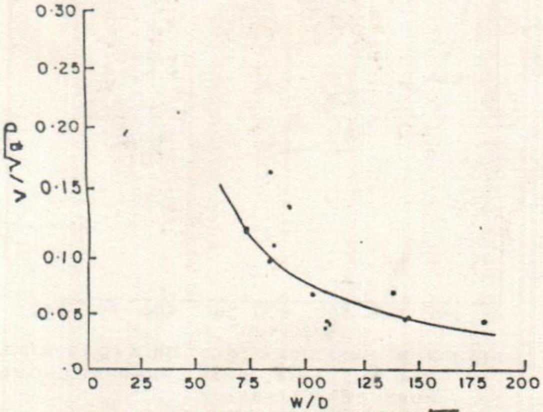

FIG.15 RELATION BETWEEN $V/\sqrt{gD}$ AND W/D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1985)

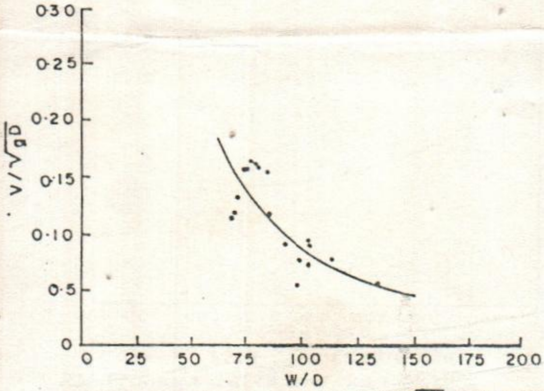

FIG.16 RELATION BETWEEN $V/\sqrt{gD}$ AND W/D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1986)

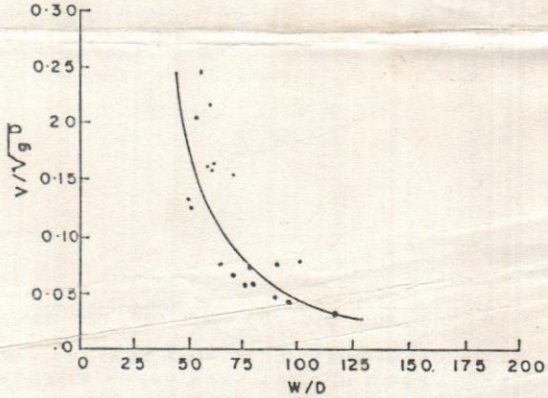

FIG.17 RELATION BETWEEN $V/\sqrt{gD}$ AND W/D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1987)

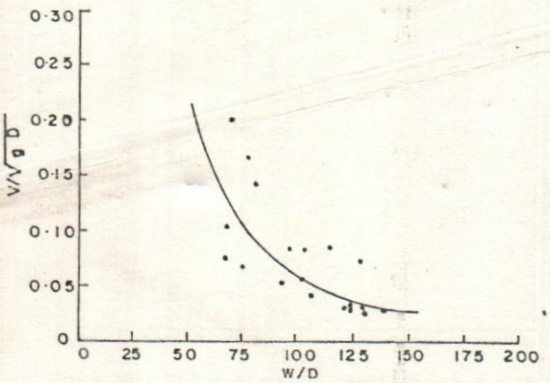

FIG. 18 RELATION BETWEEN $V/\sqrt{gD}$ AND D W/D FOR THE ARIAL KHAN UPPER AT CHOWDHURY CHAR (1988)