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# TECHNICAL PAPERS

# FIELD MODEL TESTS TO DRAIN UPPER BASIN OF SAGARKHALI KHAL OF G. K. PROJECT

BY
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## INTRODUCTION

Sagarkhali area comprising about 140 Sq Km. was a part of G. K. Project, Jessore Unit Phase- III. This area has been suffering from drainage congestion due to closure of Sagarkhali khal since 1960. Although some drainage improvement measures have been taken, but flood has been occuring frequently due to syncronization of the intensive local rainfall with high stage of the Mathabhanga. Most of the years drainage congestion occurs due to high stage of the river Mathabanga and syncronised local rainfall and the crops of the area suffer from inundation. In order to find out a rational solution of the problem, Bangladesh Water Development Board (BWDB), constituted a committee to investigate the cause and recommend remedial measures for the drainage problem of the Sagarkhali area.

The Terms of Reference (TCR) of the Committee as follows:-

To examine & evaluate the necessity of construction syphone and other structures for removal of drainage problem of Sagarkhali area,

To prepare the design & drawing of the structures, and

To submit the report with specific recommendation of the problem.

In course of discharging its duties, members of the committee visited the affected area, inspected existing drainage system, discussed with the local people & project officials, and studied the reports prepared by different agencies in past and also examined the opinions of different officials, experts and local people.

The committee conducted a field model test in order to examine the efficiency of the drainage system in the project area and possibility of passing excess water of the affected area without causing harm in the project area.

## DRAINAGE PROBLEM OF SAGARKHALI AREA

### Problem and its Salient Feature

A flood control and irrigation project named as G. K. Project was taken up in 1952, with its main component, pump house located at Bheramara. In order to develop Kushtia Unit Phaset - I, along with other activities, it was necessary to close Sagarkhali Khal near Amla in 1960. This crossdam obstructed the natural flow of flood water of upper basin of Sagarkhali to pass through the natural drainage system in the Kushtia Unit, Phase- I, and created drainage problem in the upper basin of Sagarkhali Khal, comprising about 90,000 acres of land.

Although several items of works were done the problem of drainage congestion of the area could not be removed. As and when high water stage in the river Mathabhanga syncronises with the local intense rainfall, the drainage is not effected in the river Mathabhnaga and the crops and dewelling housed of the area suffer from inundation. People of the area attempt to cut open the cross dam to drain out the area.

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## Location of the area

The Sagarkhali area comprising about 140 sq. km (about 90,000 acres) is bounded by the river Ganges in the north-east and Mathabhanga river in the south-west and Ganges canal of G. K. Project Kushtia unit phase- II in the east. The area which was originally a part of G. K. Project Jessore unit, phase- III falls in the upazilla of Daulatpur, part of Bheramara and Mirpur.

## Sagarkhali river and its influence on the problem

The Sagarkhali river originated from the river Ganges near Mohishkundi in Kushtia district flowing towards south-east to drain into the Chapai beel in the same district. At the mouth the river was known as Hishna, it was than called Chasnidoa in the flood problem area and thereafter upto Chapai beel it was named as Sagarkhali khal. A channel named Barisal khal took off from Chapai beel to join Kumar river at Vishnudia in Jessoe district. It flows further downstream as Kumar to meet the Nabaganga river at Magura and ultimately drains into the sea. After constraction of closure dam at Amla natural draiage of Sagarkhali area has been obstructed.

## Background of G. K. Project and creation of the problem

A project for agricultural development in Kushtia, Jessore, and Khulna, named as G. K. Project. providing flood control/drainage, and irrigation was taken up in 1952 with the help of F. A. O, which recommended immediate implementation of the Kushtia Unit of the Project by pumping water from the Ganges. The Kushtia Unit Phase- I with a gross area of 200000 acres and a net cultivable area of 120000 acres was taken up for execution in 1954. A pumping plant was constructed at Bheramara for pumping water from the river Ganges through an intake channel. A system of irrigation canals was constructed for carrying and destributing the water into the entire commanded area. A flood embankment along the bank of the Ganges and the Gorai river (left bank of main canal) for a length of about 65 miles was constructed to protect the project area from annual flooding. A net work of drainage channels for quick surface drainage as well as for removing drainage congestion from the low pockets, swamps and beels in the project area were constructed to reclaim lands and to ensure effective drainage. Before the project development of the Kushtia unit, phse- I, the entire low lying area of the project used to be flooded almost every year from the spills of the Ganges and Gorai through the river and channels, namely, Hishna (Sagarkhali), Kaliganga, Dakoa, etc, and the crops of these beels and low lying areas were damaged frequently. With gradual development of the project, all these spill channels were closed by earthen dams. Kaliganga and Dakoa were closed at their offtakes whereas the Sagarkhali khal was closed near Amla in the year 1960. The cross dam over the Sagarkhali khal near Amla was so located that the Ganges canal for the Kushtia unit phase- I of the same project can cross the Sagarkhali khal through the cross dam, obstructing the natural flow of flood water of the Sagarkhali upper catchment area between the Ganges, Mathabhanga and the Ganges canal of G. K. Project Kushti a unit.

## Previous plans executed for removal of the problem

To solve the anticipated drainage congestion due to the obstruction of natural drainage of upper Sagarkhali basin a diversion channel named D5M was excavated in the year 1961 to divert the Sagarkhali water from its original course to Mathabhanga river. The diversion channel took off from the Sagarkhali khal at a point near Amla, upstream of the cross-dam and connected river the Mathabhanga at Khalishakundi. But it was observed that the spill from the Ganges through Hishna, Chasnidoa (Sagarkhali innundates the entire low lying area of the upper catchment of the Sagarkhali khal and damages the crops.

To prevent the spills from the Ganges a cross-dam was constructed in the year 1964 on the Hishna (Sagarkhali) at Mohishkundi. Subsequently it was also observed that during low stage of Mathabhanga drainage through D5M is effectively occured, but during high stage of Mathabhanga (which generally occurs in August-September), instead of drainage, back flow from Mathabanga entered in the area and caused damage to the crops. The maximum water level of Mathbhanga at the outfall of D5M and water level at U/S of D5M regulator for some periods of non-effective drainage are shown in Table 1

**Table- 1**

**MAX$^{m}$ WATER LEVEL AT U/S OF D5M REGULATOR AND WATER LEVEL OF MATHABHANGA AT OUTFALL OF D5M**

| Period | W. L. at U/S of D5M Regularter Ft. (PWD) | W.L.of Mathabhanga river at the outfall of D5M |
|---|---|---|
| 1.9.82 to 18.9.82 | 40.00 to 40.70 | 40.05 to 44.80 |
| 14.9.83 to 29.9.83 | 39.10 to 41.70 | 39.40 to 46.40 |
| 1.9.84 to 21.8.87 | 40.80 to 43.75 | 40.65 to 45.85 |
| 17.8.87 to 24.8.87 | 38.60 to 41.85 | 39.10 to 42.70 |

To stop backflow of Mathabhanga through D5M a regulator was also constructed at the outfall of D5M.

**Present problem**

Almost every year when high stage of Mathabhanga is syncronised with heavy rainfall large area of Sagarkhali basin including homesteads submerges. In 1987 due to heavy rainfall (about 11 inches in 3 days) the problem became acute, As a result crops of about 11000 acres of land and 300 homesteads were damaged causing immense suffering to the people of the area. Thousands of people from the effected area of Daulatpur upazilla gathered at the cross-dam site of Sagarkhali at Amla and tried to cut open the Sagarkhali dam.

Realizing the consequences of cutting of the cross-dam the people in the benefitted area of G. K. Project Kushtia unit resisted the cutting of the cross-dam. A clash between the two groups of people occured. The people of the affected area believe the only solution to mitigate their suffering is to cut the cross-dam and restore the preproject condition of the Sagarkhali khal. Whereas the people of the benefitted area will never allow to do so. Since almost every year this has become a chronic problem, both for law enforcing agency and WDB, this demands a permanent solution. This report is therfore, the output ot the damand.

**Hydrology and flood depth analysis of the basin**

A storm of 5 day duration with 10 year recurrence interval (4 month period from June to September) has been selected as the design rainfall to compute the rainfall runoff. The inflow hydrograph of the basin generated for the design rainfall using the unit hydrograph concept is presented in Figure 1.

From area elevation curve of Sagarkhali basin presented in Firgure 2 it can be seen that at elevation +40 total basin area is 3600 acres which is only 4% of the total area and considered to the lowest pockets in the beel and may not be requirea to drain. This was also confirmed during field visit and discussion with field officials and local people that inundation upto + 40 RL in the Sagarkhali basin does not affect the standing crops and is accepted as tolerable level of flooding. Hence the excess water above + 40 RL needs to be removed to keep the area free from flooding.

A number of flood routing computations were made by micro computer using LOTUS 123 package program and from the time stage relation curve for 12 vents (5' X 6') regulator with outflow of 1675 cfs it is observed that above + 40 Ft. PWD the depth of inundation for 48 hours is less than 7" and hence is considered acceptable Figure 3.

**INFLOW HYDROGRAPH**

SAGARKHALI BASIN

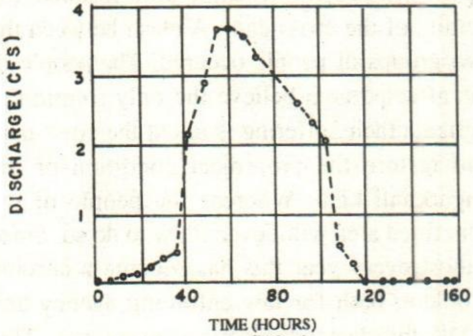



FIG -1

**AREA ELEVATION CURVE**

SAGARDHALI BASIN

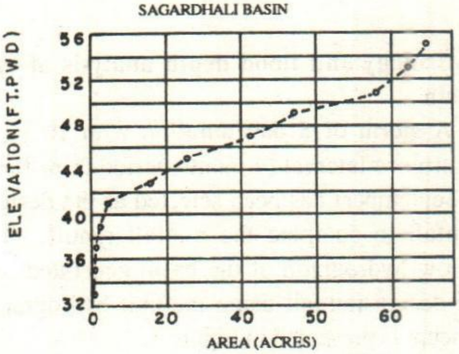



FIG- 2

**STAGETIME RELATION CURVE**

SAGARKHALI BASIN (12 VENTS 5'X6')

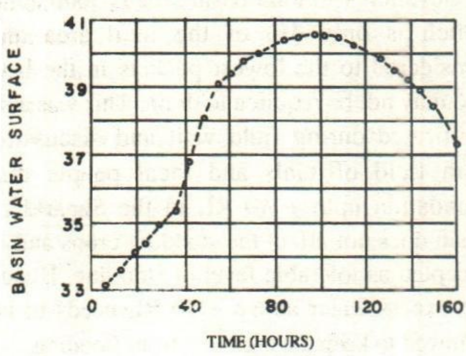



FIG -3

**Present agriculture in the Sagarkhali area:**

Present agricultural practice in the flood problem area of Sagarkhali besin showing present cropping pattern with percent of area covered are furnished in Table- 2 below.

**Table- 2**

**PRESENT AGRICULTURE IN THE FLOOD PROBLEM AREA OF SAGARKHALI BASIN**

| Crops | % of total area covered |
|---|---|
| Local Aus | 30 |
| IRRI Aus | 20 |
| Aman | 55 |
| Jute | 40 |
| Sugarcane | 20 |
| Wheat | 20 |
| Boro | 10 |
| Pulses/Tobacco | 50 |

**Proposed alternative plans.**

To solve the drainage problem in the Sagarkhali basin following alternative plans were studied :

-Construction of a syphone at Sagarkhali across Ganges canal to pass the entire flood excess water into G. K. project area.

-Construction of a pumping plant for drainage at the outfall of D5M.

-Construction of a regulator over Mathabhanga at a suitable distance U/S of the outfall of D5M. to keep the river stage lower permitting efficient drainage.

-Construction of an inlet structure to pass the flood water through Ganges canal, Alamadana canal. and excape channel.

**DISCUSSION ON THE PROPOSED PLANS:**

**Construction of syphone at Sagarkhali across Ganges canal :**

The people of the affected area have been demanding to restore the original flow condition of Sagarkhali khal. This can only be achieved by constructing a syphone across the canal and passing the flood water of upper Sagarkhali basin into G. K. Project area and drain into Chapai beel and then in to Kumar river. This will couse innundation is to the developed area of G. K. Project phase- I & phase- II & will damage the crops, specially in Chapai beel and adjoining area, This proposal will not be acceptable to the people in the developed and benefitted area of G. K. project Kushtia unit and will be very difficult to implement. Irrigation in the vast Ganges canal command area may have to be suspended for two seasons during construction of the propsed syphone.

**Pumping plant for drainage at the outfall of D5M :**

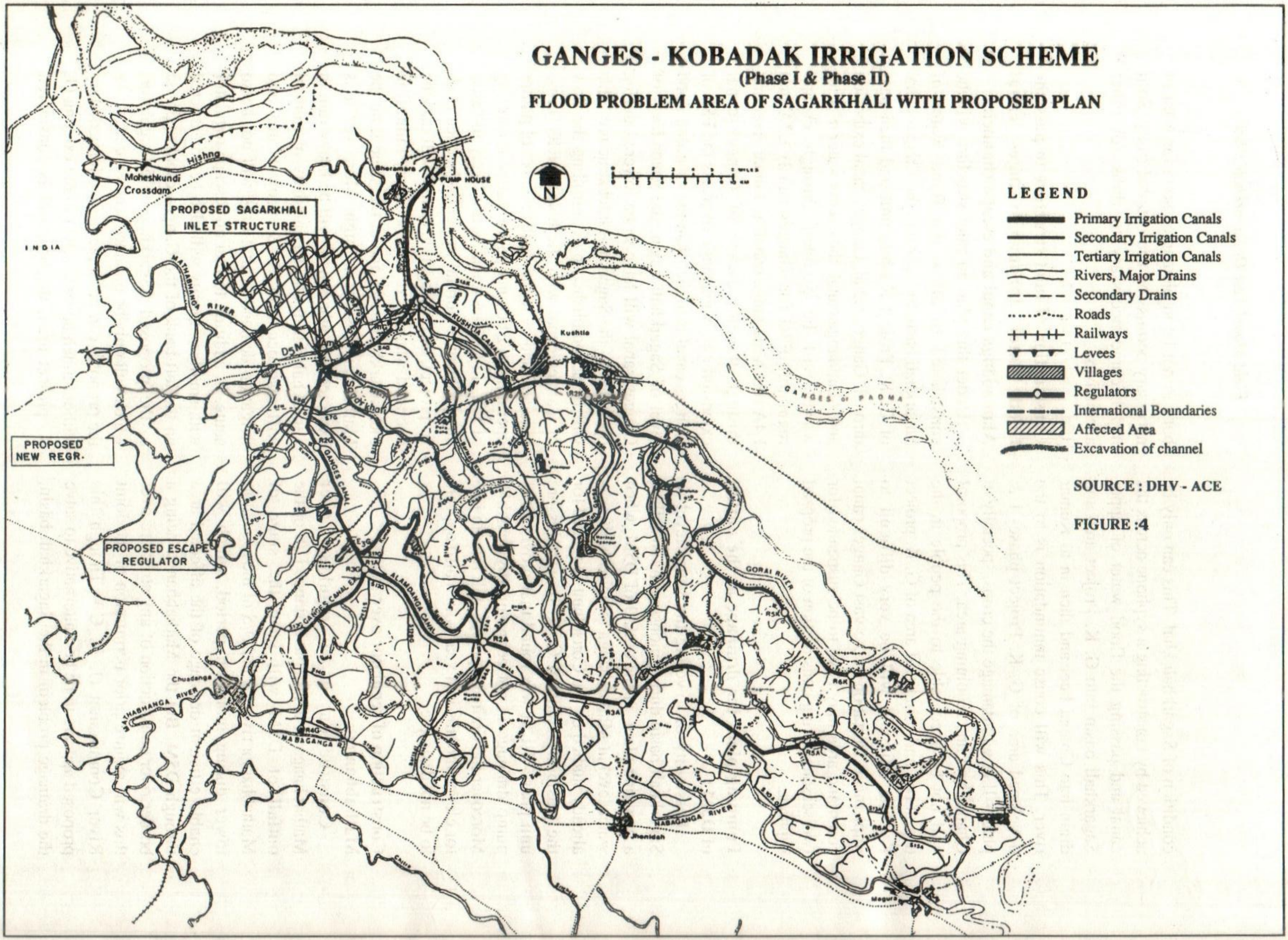
Possibility of lowering the water level of Sagarkhali basin during monsoon by installing a pumping plant at the outfall of D5M, when water level at Mathabhanga remains high, was already studied by the Consultants engaged for the feasibility study of G. K. project Jessore unit phase- III. It was found that provision of pump drainage is not economically feasible. Moreover the drainage pumps will remain idle for eleven months in a year which also appears to be not desirable.

**Construction of regulator over Mathabhanga :**

Construction of a regulator over Mathabhnaga at a suitable distance U/S of the outfall of D5M will keep the stage of Mathabhanga river, at the D/S of the regulator, lower during monsoon period, which will enable efficient drainage of the affected area through D5M. But the Mathabhanga being a border river construction of any structure over this will require prior permission of the Joint River Commission (J. R. C.). Though this proposed appears to be best alternative to salve the drainage problem in the Sagarkhali basin, but can not be in plemanted soon for want of necessary permission and detailed study. So it has become necessary to think for other alternatives.

**Construction of inlet structure to pass the flood water through Ganges canal Alamadanga canal and escape channel:**

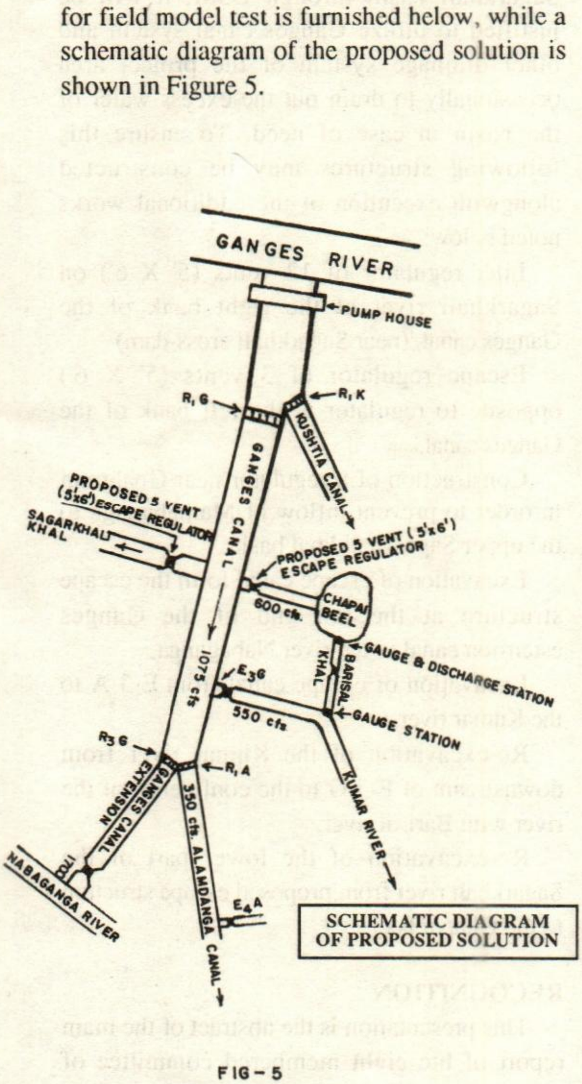
Under this plan an inlet structure will be constructed to allow the flood water of Sagarkhali basin to enter into the Ganges canal of G. K. Project Kushtia unit and then pass through Ganges canal, Ganges canal extension and Alamdanga canal, the excess water of after wards will be escaped through escape regulators E3G (on Ganges canal) E2A and E4A (on Alamdanga canal) to kumar river and through the tail escape of Ganges canal extension into Nabaganga river. As the FSL of Ganges canal is higher than the tolarable flood level of Sagarkhali basin, the water level of Ganges canal will have to be lowered during operation of the Sagarkhali inlet structure. This will be accomplished by controlling the flow of Ganges canal with the help of RIG. cross regulator and by reducing the number of pumps operating at Bherama pump station. Lowering of FSL in Ganges canal is not likely to cause any problem to normal irrigation practice in G. K. project, as the Sagarkhali inlet will only be operated when there will be heavy rainfall in the Sagarkhali basin and at the same time the Mathabhanga stage is high. Moreover it is expected that when there will be heavy rain in Sagarkhali basin similar intensity of rainfall will also occur in G. K. project area and irrigation demand at that time will not be so acute. In addition to that an escape regulator with smaller capacity will also be constructed on the left bank of the Ganges canal opposite to the proposed Sagarkhali inlet regulator, which will only be operated when required. As the proposal can be implemented immediately without much adverse affect to the existing G. K. project area, the proposal is considered


GANGES - KOBADAK IRRIGATION SCHEME
(Phase I & Phase II)
FLOOD PROBLEM AREA OF SAGARKHALI WITH PROPOSED PLAN
LEGEND
Primary Irrigation Canals
Secondary Irrigation Canals
Tertiary Irrigation Canals
Rivers, Major Drains
Secondary Drains
Roads
Railways
Levees
Villages
Regulators
International Boundaries
Affected Area
Excavation of channel
SOURCE : DHV - ACE
PROPOSED SAGARKHALI INLET STRUCTURE
PROPOSED NEW REGR.
PROPOSED ESCAPE REGULATOR
Moheshkundi Crossdam
Hishna
PUMP HOUSE
INDIA
MATHABHANGA RIVER
KUSHTIA CANAL
Kushtia
GANGES or PADMA
GANGES CANAL
ALAMDANGA CANAL
GORAI RIVER
NABAGANGA RIVER
Chuadanga
Jhenidah
Magura
FIGURE :4

attractive under existing condition. An index map of G. K. project Kushtia unit phase- I and phase- II showing the problem area with proposed plan is presented in Figure 4.

## FIELD MODEL TEST

To check the effectiveness of the last proposal, in the field, a field model test was performed from 24th to 26th September, 1988 by regulating the flow in the Ganges canal so as to attain the desired design water level at the D/S of the proposed inlet structure and find the actual discharge capacity of Ganges canal at that level. The details of the adopted procedure for field model test is furnished below, while a schematic diagram of the proposed solution is shown in Figure 5.




**SCHEMATIC DIAGRAM OF PROPOSED SOLUTION**

FIG – 5

Water level of Ganges canal at the Sagarkhali cross-dam was maintained at +39.5 by controlling the gates of RIG (regulator No. 1 across Ganges canal) at the U/S of the proposed inlet structure.

E3G regulator (Escape regulator No. 3 of Ganges canal at left bank) and R1A regulator (Regulator No 1 across Alamdanga canal) were kept open.

The model was run for more than 24 hours to stabilize the flow.

Discharges through Ganges canal, Alamdanga canal and through the escape channel of E3G regulator were respectively measured by current meter at Sagarkhali cross-dam. D/S of R1A and D/S of E3G regulators The results of the tests were as follows :

| Location | Discharge (cfs) |
|---|---|
| Ganges canal at Sagarkhali X-dam | 875 |
| Alamdanga canal at the D/S of R1A | 340 |
| Escape channel at the D/S of E3G | 550 |

## Discussion on the results of the field model test

During model test the tail escape of Ganges canal extension could not be opened as the escape channel has not yet been excavated. But from the opening of the escape regulator and gauge observation of Nabaganga river it is estimated that another 200 cfs of water can easily be passed through this tail escape to/Nabaganga/river. From the tield model test it is therefore, conservatigely estimated that out of 1675 cfs, 1075 cfs of water can safely be passed through the Ganges canal and for the rest 600 cfs. a 3 vents (5' X 6') escape regulator is proposed just opposite to the Sagarkhali inlet structure to escape into Chapai beel.

**Effect of the recommended proposal on Chapai beel and adjoining areas and comparison of the effect with the provision of syphone across Ganges canal instead of inlet structure.**

Two water level gauges were installed on 9.8.88 one at the off-take of Barisal khal rear Chapai been and other at the confluence of Barisal khal and Kumar river with a view to check the effect of model test in Chapai been and adjoining areas. After passing 550 cfs of water through E3G regulator during model test no rise of water level was observed on Chapai beel. From the water level records of the above gauge and corresponding discharges, a rating curve has been developed for Barisal khal and from the rating curve and area elevation curve of G. K. Project area (Chapai been and adjoining areas) it is found that about 400 acres of additional land may be submersed if 600 cfs of water is allowed to pass through escape into Chapai beel. Whereas about 2000 acres of additional land will be submersed if a syphone is constructed across Ganges canal instead of inlet structure and 1675 cfs of water is passed through the syphone into the Chapai beel. Moreover in average years it is expected that the Ganges canal will be capable enough to take care of the flood flow of Sagarkhali basin and the proposed escape at Amla might not be required to operate. On the other hand, if a syphone is provided, every year some additional area in and around Chapai beel will be subjected to flooding. Besides, construction of syphone will cause suspension of irrigation in the Ganges canal system at least for two years or diversion canal should be needed at the cost of huge lands and additional expenditure. Priority of providing a syphone of 600 cfs instead of the proposed escape of same capacity was also examined. Results of routing with or without syphone is indecate that Provision of syphone of 600 cfs capacity instead of escape of same discharge reduce the ultimate storage level in the basin by 0.25 ft. On the other hand dewatering expendinture, land requirement for diversion or suspensions of irrigation in Ganges canal system during construction is cost prohibative.

Moreover, syphone will always encourage drainage in the project area in all floods to incurs regular damage of additional land in Chapai beel area, of G. K. project area.

**RECOMMENDATIONS**

Until decision can be taken to construct off-take regulator on the river Mathabhnaga to ensure all time effective drainage of the upper Sagarkhali basin through D5M, it will be justified to utilize Ganges canal system and other drainage system of the project area occasionally to drain out the excess water of the basin in case of need. To ensure this following structures may be constructed alongwith execution of the additional works noted below:

Inlet regulator of 12 vents (5' X 6') on Sagarkhali river at the right bank of the Ganges canal. (near Sagarkhali cross-dam)

Escape regulator of 3 vents (5' X 6') opposite to regulator at the left bank of the Ganges canal.

Construction of a regulator near Goalgram in order to prevent inflow of Mathabgnaga to the upper Sagarkhali khal basin.

Excavation of escape canal form the escape structure at the tail end of the Ganges estension canal to the river Nabaganga.

Excavation of escape canal from E-3 A to the Kumar river.

Re-excavation of the Kumar river from downstream of E-3 G to the confluence of the river with Barisal river.

Re-excavation of the lower part of the Sagarkhali river from proposed escape structure to the Chapai beel.

**RECOGNITION**

This presentation is the abstract of the main report of hte eight membered committee of

WDB, The Author recognizes the contribution of the committee in presenting this case study report.

## REFERENCES

BWDB : Report on Drainage Problem in Sagarkhali Area in Kushtia District, 1987.

BWDB: Hydrologic and Hydraulic Design Procedure"

Report on: Investigation and recommendation of the drainage problem in the upper basin of Sagarkhali khal prepared by a committee of WDB.

# PHYSICAL MODELLING ON RIVER CHANNEL EROSION WITH CASE STUDIES

BY

**MD. TOZAMMEL HOSSAIN***

**ABSTRACT :**

*Bangladesh is a reverine country having most complex river system of the world. Great social problems are caused every year by bank erosion on three major rivers the Brahmaputra-jamuna, the Ganges, the Meghna and their tributaries and distributeries. Different river channel characteristics were discussed along with the cause of bank erosion with special emphasis on braided river. Physical modelling technique as used for solving the above river bank erosion were described briefly. Also reviewed the following problems, solutions obtained by physical modelling and its implementations :*

*Protection of Shumvuganj on old Brahmaputra River ;*

*Protection of Kustia and Kumarkhali Towns on Gorai River ;*

*Erosion of Sovapur Bridge on Feni River ;*

*Protection of Sirajgoni Town on Jamuna River;*

*Protection of Rajshahi Town on the Ganges.*

## INTRODUCTION

The aim of the paper is to present the main features of the Bangladesh river channel characteristics and the applied results of studies on hydraulics physical models in connection with engineering solutions for bank protection against erosion and local river training. Therefore 1st part summarises relevant existing data and 2nd part reviews some case studies.

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## TYPES OF CHANNEL AND EROSION IN PRINECIPAL RIVERS OF BANGLADESH & THEIR STUDY ON PHYSICAL HYDRAULIC MODELS.

### The river network.

The main rivers of Bangladesh are Brahmaputra-jamuna, the Ganges and Meghna. Jamuna River is a part of the Brahmaputra River, one of the largest of the world, flowing from the Himalyas in Tibet through Assam. Before crossing the border between India and Bangladesh, Brahmaputra collects it's Dharla River tributary and after the border another one, Teesta River. At the same place it changes own geographical name, being known as the Jamuna until its confluence with the Ganges near Aricha. After the confluence, the river is known as Padma, and after joining the Meghna River near Chandpur it flows into the Bay of Bengal.

The Brahmaputra-Jamuna-Padma River and the Ganges carry together at flood peak about 85% of the total river discharge of the country. The average flood curves of the two rivers reach quite high peak discharges, but at different times as shown in Table 1 and the grain size of bed meterials at different locations of Brahmaputra, Ganges and other Rivers have been shown in Table 2 The 10 years return period flow in the Jamuna is estimated 76, $400 m^3/s$ at Bahadurabad, based on flows recorded between 1950 and 1970. A flow of 92.300 $m^3/s$ was recorded there in 1974. By comparison, the maximum flow recorded in the

## Table- 1

## CONCOMITANT DISCHARGES ON JAMUNA AND GANGES RIVERS

| RIVER | Q | A | M | J | J | A | S | O | N | D | J | F | M |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| JAMUNA AT BAHADURABAD | MONTHLY MEAN DISCHARGES FROM MEAN HYDROGRAGES | 6100 | 11200 | 27500 | 42700 | 44100 | 38700 | 28700 | 46200 | 8400 | 5800 | 4500 | 4500 |
| GANGES AT HARDINGE BRIDGE | | 4800 | 4800 | 3100 | 44700 | 28500 | 37500 | 2z6500 | 42300 | 5600 | 3000 | 2400 | 2000 |
| RIVER | Q | A | M | J | J | A | S | O | N | D | J | F | M |
| JAMUNA AT BAHADURABAD | MEAN MONTHLY DSICHARGES 1980/81 | 9070 | 47500 | 28700 | 38200 | 48700 | 33300 | 23000 | 10700 | 6700 | 6860 | 4500 | 5340 |
| GANGES AT HARDINGE BRIDGE | | 843 | 1220 | 3290 | 25300 | 48000 | 44500 | 14500 | 5110 | 2730 | 1640 | 1330 | 994 |
| RIVER | Q | A | M | J | J | A | S | O | M | D | J | F | M |
| JAMUNA AT BAHADURABAD | MAXIMUM MONTHLY DSICHARGES 1980/81 | 18500 | 21700 | 44100 | 50200 | 61200 | 33800 | 38500 | 15500 | 8350 | 5350 | 4700 | 6540 |
| GANGES AT HARDINGE BRIDGE | | 959 | 1470 | 5350 | 43000 | 57800 | 57200 | 26700 | 7610 | 3510 | 2060 | 1500 | 1150 |

**Table- 2**

**GRAIN SIZE OF BED MATERIALS IN mm.**

| River | Place | $D_{65}$ | $D_{60}$ | $D_{Mean}$ | U.C. |
|---|---|---|---|---|---|
| Brahmaputra-Jamuna | Chilmari | 0.266 | 0.249 | 0.261 | 1.9 |
| | Bahadurabad | 0.270 | 0.259 | | 1.8 |
| | Sirajgonj | 0.171 | 0.149 | 0.172 | 1.9 |
| | Ballabbari | 0.165 | 0.161 | 0.162 | 2.0 |
| | Nakfater Char | 0.186 | 0.174 | 0.176 | 1.8 |
| | Nagarbari | 0.181 | 0.172 | 0.176 | 2.0 |
| Ganges | Hardinge Br. | 0.181 | 0.163 | | 4.6 |
| | Kalikapur | 0.134 | 0.126 | 0.131 | 2.1 |
| | Gorai | 0.193 | 0.187 | | 1.8 |
| Ganges- | Baruria | 0.164 | 0.153 | | 2.6 |
| | Goalundo | 0.153 | 0.144 | | 2.5 |
| | Taraghat | 0.139 | 0.129 | | 1.8 |
| | Bhagyakul | 0.187 | 0.175 | | 2.1 |
| | Chandpur | 0.180 | 0.129 | | 1.9 |
| Other river | Jagir | 0.165 | 0.162 | | 2.2 |

Ganges at Hardinge bridge was 73,000 $m^3/s$ in 1960.

Most of the difficult River Engineering problems are faced on Brahmaputra-Jamuna-Padma River, especially between Bahadurabad and Aricha, and on Meghna River. The Ganges Manifests nowadays a certain trend towards meandering and less randomly varying erosion, probably due to both a services of rather dry hydrological years and to flood discharge accumulation at Farakka dam plus discharge diversion.

Therefore, the comments on particular morphological and hydraulic river characteristics will be focused on Jamuna River.

## Geographical morphological and hydraulic characteristics.

The Brahmaputra-Jamuna-Padma River is a very young river, especially in the Jamuna sector. Two hundred years ago, the main Brahmaputra River flowed east of Dhaka and entered the Bay of Bengal through a mouth separate from that of the Ganges, Maps based on surveys made between 1764 and 1779 clearly show it. Before 1787 Teesta River, which comes from Sikkim and drains part of the southern himalayas, joined the Ganges upstream of its existing confluence with Jamuna. In that year it switched its course to join the Brahmaputra, taking with its large quantities of coarse sediment. By 1810 the combined river started to divert its flow into the present course of the Jamuna. By 1830 it had become the main watercourse and the channel passing to the east of Dhaka has decayed. The reasons for the change are complex and not yet fully understood but apart from the fact that the rivers all carry vast amount of sediments the area is subject to earthquakes. It is likely there has been progressive uplift and tilting of land which has caused changes in the course of rivers.

Immediately to the north of the river Ganges and upstream of its junction with the Jamuna is a low-lying belt associated with the courses of the rivers Atrai. This belt contains quantities of clay that have probably inhabited northward migration of the Ganges. A region further north and to the west of the Jamuna is raised above the general level of the Gangetic plain between 3 and 10 m. The raised part, known as the Barind Terrace, is similar geographically too, though much older than the area south of the river. The raised land consists of layers of silt with some clay overlaying and, at greater depths, gravel. Still further north, rivers draining the southern slopes of Himalayas bring down coarser sediments which form alluvial fans, the development of which has led to migration of river channels now feed the western side of the Jamuna.

It is interesting to examine the typical morphological trends of Jamuna River. It is known that three main types of flow pattern in river beds exists namely:

Straight streams, quite similar to straight canals;

Meandering streams, with more or less symmetrical loops or bends;

Braided streams, which consist of many intertwined channels (anabranches) separated by islands. Braided streams tend to be very wide and relatively shallow, with a random morphology of the bed. No formal statements about the geometry of braided streams are yet possible by difference of the first two types of flow.

For the main rivers of Bangladesh, the frequent case at relevant (high) discharges is just the braided type of flow.

Even for the most straight sector of Jamuna, before the junction with the Ganges, the river is close to the braiding flow pattern at high discharges and has large migratory shoals of sand known as Chars, which contribute to the development of braided channels in the wide river bed. At low discharges, some meandering in the individual channels becomes possible. More upstream on the river, the braiding becomes more manifest (Sirajgonj for example).

A special sector is the junction with the Ganges, with a trend to produce a secondary delta and at the same time it moves towards upstream for reasons which are not yet very clear.

The banks of Jamuna River are easily erodable in most cases due to the nature of the soil. The constituent sediments of the river bed show quite similar characteristics (Uniformity coefficient).

The hydraulic characteristics of the Jamuna vary with the discharge and increase towards upstream. The manning co-efficient varies from low to high stages and from downstream to upstream. It is interesting to compare the values with those of alluvial channels with sand bed and no vegetation, at various of bed types of bed forms:

| **Lower regime** | n |
|---|---|
| Ripples | 0.017-0.028 |
| Dunes | 0.018-0.035 |
| Washed-out dunes or transition | 0.014-0.024 |
| **Upper regime** | n |
| Plane bed | 0.011-0.015 |
| Standing waves | 0.012-0.016 |
| Anti dune | 0.012-0.020 |

The comparison suggests the river in most situations a lower bed form regime up to transition. Indeed, a detailed analysis of the flow conditions at Bahadurabad shows that during rising discharges up to the flood peak, the transition and plane bed are attained several times.

For this reason the n values from field data should be carefully scrutinized.

### Typical engineering problems. The technique of physical modelling used for solving local river channel and bank erosion/protection problem.

The typical engineering problems are bank erosion and changes in river bed configuration with influence on navigation and local crossing. The bank erosion is caused by the geotechnical nature of the noncohesive bank material. At high stages the moisturing of the bank, accompanied by variations in water level, increased pore pressure and also local higher flow velocities produces bank sliding with serious implications for landing fronts and towns along the river. The braiding flow pattern, with random changes of channels and frequent modifications of the flow velocity incidence attacking the bank brings a very high factor of risk.

It is beyond the scope of the proceeding to analyse here the different approaches in physical modelling erosion. Sometimes, it is felt that a short review of the approach used in several studied cases where the prototype faced the conditions briefly mentioned above case studies could be useful.

The careful examination of every case history in much more important on braided river sectors, because of quick random changes from one year to the other one and from one season to the other one.

Therefore, for the study on hydraulic model of any bank protection of local river training works, two possibilities exist in principle :

A very large number of experiments on a mobile bed model which could (statistically) reproduce braiding flow pattern in the considered stretch.

A reasonable number of experiments on a fixed bed model (undistorted/distorted), covering dangerous relevant and extreme cases of morphological and flow incidence cases to which the bank/local training structures could become exposed. It is less expensive and probably safer to use the second approach for both general and sectional hydraulic models. It is advantageous to use fixed bed models made of granular material at its limit of equilibrium, which permits also to identify local scour trends. The most of the case studies of the present proceeding are based on such models.

The flow regime on the river model for calibrating and running it in relevant conditions is usually the steady flow one, although river discharge is rateable in time according to the locally known hydrograph.

The engineer and the scientist must therefore decide what constant discharges are of relevance for carried out the study. In the literature, a frequently met notion is the dominant discharge, which has a meaning if the river model has a mobile bed. If the model is a fixed-bed one, the highest relevant discharge is usually the bank full discharge, which has a return period of about 1.4 to 1.5 years calculated from annual maximum floods.

The calibration of fixed-bed models is a specific one and differs from that practised with mobile-bed river models, because it mainly implies a checking of the global kinematic behaviour of the model at discharges equivalent to those at which the field data on prototype were collected. The main steps are the following ones:

Verification of the similarity of flow velocity distribution in the reference cross-section, with possible interventions for improving the similarity.

Verification of the similarity of reference flow pattern, if such information from the prototype exists (at given discharges).

A possible adjustment of model's roughness to improve both the similarity of flow velocity distribution and to get a similar longitudinal profile of the free-level surface.

To get model a correct reporduction at relevant dischages (flood peak, bank-full discharge etc.) of the bank exposure to stream velocity concentrations, for making sure that the model gives proper information on bank scouring "trends" in natural (non-modified, viz, without the presence of such structures like dikes, spurs ect.) conditions.

This checking will make it possible to have confidence that the model will give later reliable information also it modifies local conditions, viz. after using groynes, spurs or other changes on the model.

**CASE STUDIES.**

Some example of model studies are described below by which one may get the idea of different types of problems, the solutions obtained from the model studies and the implementations in the field with its consequences.

**Erosion and protection of the left bank of the old Brahmaputra River opposit to Mymensingh Town.**

Shamvuganj Jute Mill was threatened by the erosion of left bank of the river old Brahmaputra at opposite of Mymensingh Town. Hydraulic Model study was conducted in 1977 with Horizontal scale of 1:200 and vertical 1:48 with distortion 1:4:2. It was recommended for construction of one closure dam of 500 ft length with guide bank of 1600 ft and 2 Nos, of impermeable spurs of length 600 ft of which the closure with guide bank and one impermealbe spur (600 ft) were constructed and the Shamvuganj Jute Mill was saved. But one spur is still to be constructed, the consequence of which is visible by bank erosion of the river u/s of the Mymensingh Rly. Bridge, which may be threatened ultimately.

**Bank erosion and protection on Gorai River at Kustia Town.**

Kustia Town was threatened by the right bank erosion of river Gorai. Model study was conducted with H=1:200 and V=1:30 and it was first recommended for construction of 2 Nos. of T-head groynes of size shank 500 ft and Head 400 ft. Out of the two, one T-head groynes (No. 2) was constructed in 1974, by which a big char was developed d/s of it but the problem of protection of Kustia Town deterionated day by day. Again model study was performed in 1979 and 1985 and it was recommended for construction of 3 Nos. of T-head groynes in place of the two. The second T-head entirely been changed. The char u/s of the groyne No. 1 and d/s of groyne No. 2 has completely been washed out during the last flood. Again model study is going to be conducted on the present condition of the river. If the execution of construction of two groynes was taken up at a time in 1978. the cost of additional groynes may be saved which are required for the changing condition of the river after passage of time.

**Bank erosion and protection on Gorai River at Kumarkhali Town:**

Kumarkhali Town was threatened on the left bank of river Gorai and two Nos of T-head groynes of shank length 850 ft. and Head 400 ft. and shank 600 ft and Head 350 ft. about 5000 ft apart were constructed in 1970 & 1972 respectively without any model study. Now, the thalweg of the river has been changed and the flow hits the bank directly at Kumarkhali by way of meandering pattern. Model investigation has been performed and one T-head groyne shank 500 ft. and Head 400 ft. and one Bell-mouth spur of 300 ft. length were recommended as a permanent solution.

**Bank erosion and protection of Suvapur Bridge on Feni River**

The Dhaka-Chittagong road crosses the river Feni through Suvapur Bridge. The bridge was initially 780 ft. long but in the year 1968 there was a high flood and the river bank was eroded heavily at right bank and out-flanked the bridge. The R & H department extended the bridge for about 448 ft. on the right bank of the river. In the year 1976 there was again a big flood and the right bank of the river was eroded heavily and threatened the bridge.

Model study was conducted with H=1:100 and V=1:30 and it was observed that four Nos. of T-head groynes of size head 200 ft. and shank length 100 ft. guide bank of 800 ft at both the banks and stone revetment would be necessary to protect the bridge from further erosion. When the site of that bridge was selected, no model investigation was conducted and bank erosion was not considered, other wise it my not be constructed in a river bend or if constructed necessary river training works were provided for its safety.

**Bank protection of Sirajgonj Town on Jamuna River :**

The right bank embankment of Brahmaputra River was constructed in 1967-68 keeping a distance of about half mile from non-eroding bank (firm bank) and about one mile from eroding bank. Since its construction, the right bank have seen continuing to shift towards the embankment in the process of bank erosion. As a result, within a few years of construction the embankment was attacked by erosion in several places e.g. Belka. Karaibari, Fulchari, Pakulla, Sariakandi, Kazipur, Sailabari, Sirajgonj, Someshpur, Enayetpur and Malipara. Retired embankment were constructed several times but the erosion advanced unabated. Sometimes closure of one channel at Fulchari and Kazipur, and revetment at some places were tried but without success. In 1985-86, model study was conducted to find out a permanent solution for the protection of Sirajgonj Town and it was recommended to construct a Hocky spur of 2800 ft. shank length with 250 ft, Hocky Head and two Nos. of 300 ft. and one No. of 250 ft. spur. Out of this 2700 ft. shank of Hocky spur has been constructed in 1985-86. As a result there is no erosion on the embankment adjacent to Sirajgonj Town. In this year the 250 ft. Hocky Head and the three Nos. of spur as recommended by model study should be constructed to make the protection permanent.

**Protection of Rajshahi Town from the bank erosion of the Ganges :**

Rajshahi Town is situated on the left bank of the river Ganges at about 40 miles u/s of Hardinge Bridge. During British Indian flood used to cause damages to the Rajshahi Town and it was protected by Earthen Embankment for a length of about 10.5 miles covered with brick mattressing. The main channel of Ganges at Rajshahi are constantly migrating channel swings around often tangentially and changes its course depending on the formation of shoals at the upstream. The uprecedented long duration of peak flood of 1970 damaged considerable portion of the existing mattressing and the following emergency works were doing against erosion without model study.

Construction of a T-head groyne No. 1 in front of General Post Office having 400 ft. earthen shank 165 ft. Head protected up to the toe with stone/Boulders.

A spur with brick cases projecting 90 ft deep into the river near the Dargah.

Strengthening and reconstructing Kajla Embankment.

Constructing short spurs at the strategic points at Kajla and Talaimari for deflecting the hitting current.

Later on, erosion started upstream of Post Office. Different alternative proposals for the protection of Rajshai Town were thought and Design Directorate, BWDB suggested to construct a heavy armoured T-head groynes at 12,500 ft.u/s. of the groyne No. 1 with 4300 ft. long shank and 2500 ft. head with protection up to the apron by hard materials. The proposal was studied in RRI in 1979 in a physical model of scale H=1:300 and V=1:72 and followings were the recommendations as a permanent and economic solution.

To construct three Nos. of Bell-mouth spur of 500 ft. length each at 12,500 ft,. 8000 ft. and 5000 ft, and one Bell-mouth spur of 250 ft. length at 2700 ft. from the groyne No.1. During 1979-80, the above works were completed except the construction of spur No. 4 which could not be constructed for the constraint of fund. Now, there is no immediate problem of bank protection but ultimately the erosion problem may be developed between the bank of 12,500 ft. to 5000 ft.

**CONCLUSIONS :**

Applied hydraulic research of design solutions concerning local river training and bank protection works is nowadays a recognized necessity in River Engineering and successful solutions are based on good collaboration between designers and researchers. This is even more true for braiding rivers like Brahmapurtra-Jamuna-Padma and Ganges.

Since the cost of river training and bank protection works are high/very high, applied hydraulic model research enables both a higher functional difficiency of the investment and saving of funds by possible amendment of the solution;

The design of local river training and bank protection works is at present time quite much based on general empirical or semiempirical formulae/equations. It is essential to improve and prototype, in order to get efficient guide lines for such specific design adapted to Bangladesh conditions (rivers with braiding pattern, high liquid and solid discharges wind waves at flood time etc);

When the river bank is subject to erosion and intended to be taken as early as possible rather than to tackle the problem when erosion become serious.

It has been observed that required fund is not make available to make the protection work as per design, so work could not be completed as per design and so the measure taken could not be come out successful.

For protection works against erosion, the type of materials should be decided considering its availability and cost of procurement.

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# SILT EXCLUDER SYSTEM OF TEESTA BARRAGE

BY

**A.K.M. NURUL ALAM***

**ABSTRACT :**

*Laboratory experiments were conducted to investigate the hydraulic performance of silt excluder tunnel system of Teesta Barrage having reference to the discharge in the head regulator Canal and for the exclusion of sand to the Canal. This paper presents a brief description about performance of keeping bed material out of the off-take channel, preventive measures of sediment control, advantages & disadvantages of silt excluder as well hydraulic model studies conducted in River Research Institute on silt excluder system of Teesta Barrage. The salient design features of excluder tunnel system of Teesta Barrage and some important excluder systems of this sub-continent are also devoted in this paper to the approach rather than to specific findings.*

## IMPORTANCE OF KEEPING BED MATERIAL OUT OF THE OFF-TAKING CHANNEL

Practically all the canals off-taking from the rivers at the foot of hills have a serious problem of sedimentation. The composition of bed material mainly depends upon the slope and stage of the river. Canals taking off from the rivers lower down in the alluvial stage draw sediment which is comparatively finer.

While designing an off-take from a river carrying sediment load, the control at entry of sand into the off-taking channel is extremely important and often decisive for the success or failure of a water conveyance project. Both preventive and curative methods have been used for controlling the sand entering channel. In the former method, the relatively coarser sediment is excluded before the water enters the channel, while in the later the material is excluded/ejected after it has entered it. Before this aspect of silt exclusion is discussed, it may be essential to give a brief idea of the sediment transport phenomena. This occurs in two ways, in suspension and as bed load. The relatively coarse material to be excluded is mainly confined to about one-third the depth above the bed. The finer particles of sand & clay are more or less uniformly distributed throughout the vertical. Fundamentally this occurs because of the tractive force, $\tau_0 = \gamma$ ds. Shield has given a formula for bed load movement of material having different density:

$$\frac{q_s(\rho_s - 1)}{q.S} = \frac{10(\tau_o - \tau_c)}{\Upsilon(\rho_s - 1)D_{50}}$$

where

$q_s$ = bed load per unit time per unit width

$q$ = intensity of discharge per unit width

$f_s$ = sp. gravity of bed material

$P_s$ = slope of energy line

$\tau_o$ = bed shear stress

$\tau_c$ = critical shear stress for bed material

$D_{50}$ = particle size

Variation in the suspended load concentration along the depth is given by Rouse,

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$$\frac{C_y}{C_a} = \left[ \frac{d - Y}{Y} \cdot \frac{a}{d - a} \right]^{\omega/\beta K u_*}$$

where

$C_y$ = sediment conc. at any elevation 'y' in the vertical above bed.

$C_a$ = reference conc. at a known height 'a' above bed

d = total depth of flow

w = particle fall velocity

$u_*$ = shear velocity

k = Von-Karman's constant

$\beta$ = ratio of sediment diffusion coeff. to momentum coeff.

For a still pond condition, the rate of silting should be much more than the rate of lifting and diffusion by turbulence. According to regime theory, the velocity in a channel for a given silt charge and channel dimension can be calculated by the following equation of Lacey,

$$v = 0.552 \times 1.15 \times f^{1/2} R^{1/2}.$$

where

v = velocity in m/sec.

R = hyd. depth in meter

f = Lacey's silt factor = $1.76\sqrt{D_{50}}$

where

$D_{50}$ = mean dia. in mm.

So, whenever the velocity is reduced below the regime velocity, sediment charge would start settling and to reduce bed movement. These principles are used in the "still pond" system where the water is drawn from the top layers.

## PREVENTIVE METHODS OF SEDIMENT CONTROL:

**Proper location of an off-take :** Prevention of excess load getting into the canal is achieved either through utilization of existing favourable curvature or by imposing suitable curvature such that off-take of a canal lies on the concave curve where comparatively sediment free water is available.

**Sill level of an off-take :** If sill levels of head regulators are kept higher than those of the barrage (under sluice), they would help to arrest an excess draw off sand into canal. Field experience and model tests have shown the limitation of this device. Raised sills results in a greater width of the head regulator and longer divide wall, both of which make the design less economical.

**Length of divide walls and width of Pocket for effective sediment control :-** A divide wall of optimum length plays an effective role in controlling the entry of sediment in the canal, efficient functioning of barrage.

A divide wall in conjunction with favorable curvature of flow are found to be successful in checking the sediment entry into the canals and suitable ratio of $V_R/V_P$ more than 1 can be obtained ($V_R$ = velocity of flow on river side and $V_P$ = velocity in pocket). This factor helps in better flushing of the shoal deposited in under-sluice pocket. The design of the nose of the divide wall also influences the sand exclusion to some extent. A compromise is to give the nose a flat slope on the pocket side and steep slope on the riverside. the deep scour hole on the river side will materially improve sand exclusion.

**Barrage regulation :** Two methods of operating the canals taking off upstream of a barrage are in practice :

(i) The "Still pond" operation in which all the gates in the pocket are closed and the discharge drawn into the pocket is limited to the canal withdrawals.

(ii) The "Semi-open flow" operation in which the pocket gates kept partially open.

So far as the exclusion of sand is considered, the "still pond" operation is decidedly more advantageous.

**Guidebanks :** The alignment of the guide bank upstream directs the flow pattern and helps to control the sediment from entering the canals. The exact shape is governed by the river approach upstream of the head works. Various types used are :

(i) Parallel guide banks,

(ii) Converging guide banks,
(iii) Diverging guide banks,
(iv) Bottle neck guide banks,
(v) Concave banks, and
(vi) Concave-convex banks,

The overriding consideration in fixing the alignment of the guide banks is to make the best use of the river energy to develop favourable conditions such as deep channel along the guide bank and a proper curvature of flow.

**Cantilever and skimmer platforms** : The construction of cantilever and skimmer platforms infront of the head regulator in the canal pocket is one of measure used for preventing sediment from entering the canal.

**Splay to the Head Regulator** : Splay to the head regulator gives an upstream orientation to the canal, enabling the water to flow smoothly into the canal. Generally a splay of $10^0$ to $12^0$ induces a suitable curvature of flow in the pocket.

**Sediment Excluder Tunnels** : Inspite of a good location of a head regulator, a large quantity of coarser material may still finds its way into the pocket. In these circumstances excluding or ejecting devices have to be employed. In 1932, F. V. Elsden propounded the idea of tunnel type silt excluder to prevent the sediment entering the canal. The basic principle is to exclude bottom layers of heavily silt laden water through the tunnel and diverting comparativelyless silted water into the canal. The idea took a practical shape in 1934 when H. W. Nicholas constructed an excluder in the pocket of lower Chenab canal at Khanki Headworks. Later on reviewing the success of this excluder, it was observed that the side openings provided are unnecessary and closure of these further improved the efficiency. Since then a number of excluders have been constructed in Pakistan and India. With the success of these excluders, it has now become an integral part of the headworks. Salient design features of some important sediment excluders are given in Table- 1.

**Description and design of a silt excluder** : A silt excluder consists of a no. of tunnels generally rectangular in cross section and have a bell mouth entry running parallel to the axis of the head regulator and terminating near the undersluices. The lengths of the tunnels are different but the head loss in each tunnel is kept same by suitably changing the cross sections of the tunnels. The tunnel nearest to the crest of the head regulator has to be of the same length as the head or regulator or longer. Other tunnels may be shorter in length. The height of the excluder tunnels is generally kept equal to the difference between the crest level of the canal head regulator and the upstream floor level of the undersluice bays minus the thickness of the top slab. The bottom layer of water which is highly charged with silt and sediment will pass down the tunnels and escaped through the undersluices. The clearer water over the top of the roof of the excluder tunnels. enters the canal trough the head regulator. The excluders tunnels covering only that much of the width of the approach channel which is carrying discharge almost equivalent to canal discharge should be provided.

**Design of tunnels** : A theoretical design of silt excluder is confined mainly to find out the area of the tunnel openings required to pass the designed discharge and to determine its structural requirements. On the basis of past experience and model studies, the design of an excluder system is finalised. Lahore Irrigation Research Institute and other similar Institutes in India carried out extensive research in the design of excluders. Based on their research findings, it has been found that if the excluder discharge is restricted to 20 to 30% of the canal discharge, satisfactory silt exclusion can be obtained. A minimum velocity of 2 to 4.5 m/sec. may be maintained through the tunnels in order to keep them free from sediment. After fixing the discharge and velocity, the cross-sectional area of the excluder tunnel openings, can be obtained. Knowing the height, the required width can be found and divided into a suitable number of bays.

**Approach & exit :** At the entrance, the tunnels are generally given a bell mouthed shape so as to increase the zone of suction. At the exit end, the tunnels are throttled for restricting the discharge to the desired value and to increase the velocity to prevent deposition of silt.

**Loses in channels :** The following head losses may be taken into account for calculating the tunnel sections :-

(i) Frictional loss, (ii) Loss due to bend, (iii) Transitional loss due to change of velocity in contraction and (iv) Transitional loss due to change of velocity in expansion.

The river approach condition, river regulation, sediment characteristics, alignment of head regulator, position of divide wall and staggering of excluder tunnels play a vital role in improving the efficiency of a particular excluder. Efficiency of such structure expressed as

$$\eta = \frac{Ir - Ic}{Ir} \times 100$$

Where

$I_1$ = sediment conc. in the river pocket and

$I_c$ = sediment conc. in the canal.

For model studies, the formula in the following form may be used :

$$\eta = V_{ex} - V_c \times \frac{Qex}{Qc} / V_{ex} - V_c$$

Where

$V_{ex}$ = Quantity of sediment excluded

$V_c$ = Quantity of sediment in canal

$Q_{ex}$ = Discharge through the excluder

$Q_c$ = Discharge in the canal.

## ADVANTAGES AND DISADVANTAGES OF EXCLUDER

### Advantages :

(i) The head across the barrage for operation is usually available.

(ii) Economy is secured by the use of barrage gates and

(iii) Large orifices, unlikely to be choked by rolling or submerged debris, can be provided easily.

### Disadvantages :

(i) The difficulty of securing good approach conditions :

(ii) The undersluice bays covered by the excluders cannot be used simultaneously for the passage of flood discharge, and

(iii) The structure being subject to the river action has to be robust.

## HYDRAULIC MODEL STUDIES IN RRI ON SILT EXCLUDER SYSTEM OF TEESTA BARRAGE PROJECT

The Teesta Barrage at Dalia now under construction, is located on the left bank of main course of river Teesta (Doani channel) at 6700 ft. from right bank starting pillar of C. S. No. 7. M/S. Binnie & Partners, Consulting Engineers, London prepared the hydraulic design of Teesta Barrage in the year 1969-70. It was then decided that RRI will conduct the model studies of Teesta Barrage Project based upon design as prepared by the consul. Engineers. According to this, a distorted three-dimensional model study on silt excluder tunnel system was first taken up in 1980. M/S. Binnie & Partners suggested silt, exclusion through 18 tunnels cover 6 out of total 8 bays of the undersluices together with an alternative proposal of silt exclusion through sluice/escape channel which prove unfavourable and so avoided. Tests on sediment excluder system were completed as per original plan of M/S. Binnie & partners. Plan was then modified further and study was conducted with silt excluder tunnels cover 3 out of total 7 bays of the undersluices. Axis of canal head regulator modified to $112^0$ and the upstream approach right guidebank infront of canal head regulator to $106^0$ in relation tothe axis of barrage.

# Table - 1.
## Salient design features of some important sediment excluders.

| Excluder for | Year of consturc-tion | Stage of river | Type of Exlcuder | Width of ex-excluder | Canal dis-Charge $m^3$/sec. | Discharge through excluder ($m^3$/sec) | Number of tun-nels | Staggerting of tunnel inlets (m) | Tunnel Dimension At inlet (mxm) | Shape at exit (mxm) | Avail-of intake | Efficiency able head in (m) | Length of Model per cen-tage | Proto-type percen tage. | regulator covered by exclu der. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 |
| Western Jamuna Canal at Tajewala | 1942-43 | Boulder sand and gravel) | khanki | 6.1 | 255.00 | 22.7 | 2 | 33.9 | 1.83x1.83 | . . . | Con-verging | . . . | 98.0 | 98 length | 2/3rd |
| Thal Canal at Kalabagh Headworks | 1944 | Ditto | Modified khanki | . . . | . . . | 56.5 | 6 | . . . | 2.10x4.0 | . . . | . . . | . . . | . . . | . . . length | Full |
| Remodelled Sedi-ment Excluder at Western Jamuna Canal at Tejewala | 1945 | Ditto | Khanki | 13.7 | . . . | . . . | 5 | 16.95 | . . . | . . . | . . . | | 95.0 | 93.98 | 5/6th |
| Nangal Hydel Channel at Nangal | 1954 | Ditto | Khanki | . . . | 339.8 | 421.93 | 6 | . . . | . . . | . . . | . . . | . . . | . . . | 80.100 length | Full |
| Lower Chenab Canal at kharki | 1933-34 | Alluvial | khanki | 17.07 | 340.0 | 113.0 | 6 | 17.70 | 2.13x3.05 | 1.52x3.05 | Con-verging | 3.5 | 65.0 | 60.70 length | Full |
| Haveli Main Line Canal at Emerson Barrage | 1937 | Ditto | Emerson | 42.98 | 141.6 | 24.0 | 4 | in one line | 9.75x2.44 | 9.752.44 | Straight | 4.0 | 70.0 | 72 length | Full |
| Rajasthan Canal at Harike | 1952-53 | Ditto | khanki | 40.8 | 509.7 | 141.6 | 12 | 10.96 | 3.38x1.98 to 1.79x1.98 | 2.48x1.98 | . . . | . . . | 70.0 | . . . length | Full |
| Sediment Excluder in the left pocket at Tilpara Barrage (West Bengla) | 1949-50 | Ditto | khanki | 38.7 | . . . | . . . | 2x4 | . . . | 4.15x2.18 | 4.15x1.88 | Straight | . . . | . . . | . . . length | Full |
| Lower Ganga Canal at Narora | 1967 | Ditto | khanki | 15.24 | 241.0 | 56.6 | 2x4 | 6 | 2.74x1.43 to 1.77x1.43 | 2.08x1.43 | Con-verging | 0.50 | 72.2 | 50.87 length | Full |
| Lower Sarda Canal at Barda Barrage | 1974 | Ditto | C.W.P.C | 39.88 | 780.0 | 170.0 | 14 | at a slope of 1;1 | 2.5x1.9 to 2.26x1.9 | 2.0x1.00 | Straight | 0.70 | 50.0 | . . . length | Full |
| Sediment Excluder in right pocket at Teesta Barrage (Bangladesh) | under construction | Ditto | khanki | 40.24 | 284 | 200.8 | 12 | . . . to | 3.05x2.29 to 2.13x2.29 | 3.05x1.68 to 2.13x1.68 | Straight | 0.90 | 77.0 | . . . length | Full |

## Table - 2

**Showing total river discharge, canal discharge, water levels, dynamical head diff & amount of silt deposition for different dischages.**

| Discharge in cfs. (Proto) River | Gauge in ft.P.W.D. Datum at D.S. in River | Piezometer readings (Height) in ft. 1st Tunnel U. S. | D. S. | Diff. (M) | Diff. (P) | 3rd Tunnel U. S. | D. S. | Diff. (M) | Diff. (P) | 5th Tunnel U. S. | D. S. | Diff. (M) | Diff (P) | Amount of Silt deposition (in lbs.) In canal | D/s of tunnels | D/S of under Sluice | $ of Silt entry in canal. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 |
| 20.000 | 164.34 | .540 | .476 | .064 | 3.07 | .627 | .557 | .070 | 3.36 | .431 | .368 | .063 | 3.02 | 1.88 | 4.00 | - | 32 % |
| 30.000 | 164.90 | .466 | .410 | .056 | 2.69 | .493 | .429 | .064 | 3.07 | .431 | .383 | .048 | 2.30 | 2.00 | 2.60 | 1.70 | 31.8 % |
| 40.000 | 165.40 | .474 | .422 | .052 | 2.50 | .444 | .384 | .060 | 2.88 | .515 | .470 | .045 | 2.16 | 4.50 | 8.00 | 2.37 | 30.26 % |
| 50.000 | 165.76 | .665 | .614 | .041 | 1.97 | .482 | .426 | .056 | 2.69 | .453 | .409 | .044 | 2.11 | 7.0 | 14.87 | 4.35 | 26.69 % |
| 60.000 | 166.10 | .539 | .504 | .034 | 1.68 | .516 | .463 | .053 | 2.54 | .494 | .462 | .032 | 1.54 | 10.12 | 15.0 | 6.50 | 32 % |
| 70.000 | 166.40 | .514 | .484 | .030 | 1.44 | .481 | .531 | .050 | 2.40 | .502 | .471 | .031 | 1.49 | 14.25 | 27:13 | 10.44 | 27.5 % |
| 80.000 | 166.70 | .521 | .499 | .022 | 1.06 | .503 | .458 | .045 | 2.16 | .476 | .449 | .027 | 1.30 | 22.0 | 64.5 | 12.30 | 22 % |

In all cases canal discharge 10,000 cfs. (p)

Inall cases upstream W.S. elevation in River 170. R. L.

In all cases canal W. S. elevation 168.0 R. L.

P = Prototype
M = Model

## Table - 3

## Showing the Gauge and Tunnel velocities for different discharges

| Discharge in cfs. (p) River | Gauge in ft.P.W.D Datum at D.S. in River | Head Diff. in ft. | Velocity in different tunnels (ft./sec.) No.1 | No. 2 | No. 3 | No. 4 | No. 5 | No. 6 | No. 7 | No. 8 | No. 9 | No. 10 | No.11 | No. 12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 |
| 20.000 | 164.35 | 5.65 | 9.55 | 10.96 | 11.89 | 13.21 | 13.26 | 13.32 | 15.52 | 15.98 | 13.15 | 12.66 | 12.57 | 15.83 |
| | 165.0 | 5.0 | 8.78 | 9.43 | 10.10 | 11.26 | 12.73 | 12.21 | 14.29 | 14.61 | 12.78 | 11.94 | 12.00 | 15.83 |
| | 166.0 | 4.0 | 7.26 | 8.50 | 8.28 | 9.47 | 10.52 | 10.65 | 10.65 | 12.47 | 10.20 | 9.72 | 10.77 | 13.26 |
| | 167.0 | 3.0 | 6.99 | 7.78 | 8.43 | 8.64 | 9.05 | 9.08 | 10.49 | 9.32 | 8.89 | 8.77 | 8.89 | 8.77 |
| | 168.0 | 2.0 | 5.07 | 5.68 | 6.99 | 6.80 | 7.86 | 7.76 | 8.31 | 7.03 | 6.68 | 6.92 | 8.14 | |
| | 168.20 | 1.80 | 5.40 | 5.36 | 5.71 | 6.19 | 5.94 | 6.23 | 6.87 | 7.20 | 6.43 | 5.88 | 6.18 | 7.31 |
| | 165.40 | 4.60 | 8.60 | 10.65 | 11.50 | 12.2 | 12.45 | 12.70 | 13.36 | 13.20 | 12.00 | 11.90 | 12.21 | 13.40 |
| 40.000 | 166.0 | 4.0 | 7.86 | 9.15 | 10.31 | 10.49 | 10.79 | 11.96 | 12.78 | 12.78 | 10.93 | 10.71 | 10.77 | 12.39 |
| | 167.0 | 3.0 | 7.02 | 8.24 | 8.43 | 9.09 | 8.43 | 9.09 | 9.30 | 10.22 | 9.29 | 8.70 | 9.08 | 10.55 |
| | 168.70 | 1.30 | 5.02 | 4.86 | 4.95 | 5.67 | 5.84 | 6.37 | 6.46 | 6.91 | 5.97 | 5.42 | 5.71 | 6.70 |
| | 166.10 | 3.90 | 9.22 | 9.43 | 10.03 | 10.57 | 11.68 | 11.38 | 13.41 | 12.94 | 11.63 | 10.58 | 11.78 | 12.39 |
| 60.000 | 167.0 | 3.0 | 8.13 | 8.10 | 8.78 | 8.80 | 9.57 | 10.10 | 10.75 | 10.99 | 9.86 | 9.60 | 9.92 | 9.50 |
| | 168.0 | 2.0 | 6.51 | 6.17 | 6.95 | 7.01 | 6.99 | 7.10 | 8.29 | 8.18 | 7.56 | 7.53 | 7.93 | 8.14 |
| | 169.20 | 0.80 | 4.47 | 4.16 | 4.48 | 5.03 | 5.30 | 5.63 | 6.00 | 6.01 | 5.67 | 5.06 | 5.89 | 6.47 |
| | 166.70 | 3.30 | 7.20 | 7.27 | 7.37 | 7.34 | 7.53 | 8.81 | 8.29 | 9.13 | 7.82 | 7.96 | 7.96 | 9.19 |
| 80.00 | 168.0 | 2.0 | 5.47 | 5.56 | 5.34 | 5.77 | 5.92 | 6.47 | 6.50 | 6.91 | 6.13 | 5.68 | 7.04 | 7.40 |
| | 169.0 | 1.0 | 4.25 | 4.31 | 4.40 | 4.87 | 4.63 | 5.16 | 5.60 | 5.84 | 4.80 | 5.46 | 5.75 | 6.13 |
| | 169.60 | 0.40 | 4.66 | 4.20 | 4.75 | 5.32 | 5.37 | 6.07 | 6.74 | 6.72 | 5.85 | 5.72 | 6.67 | 7.12 |

In all cases canal discharge 10,000 cfs. (p)

In all cases unstream W.S. elevation in River 170.0' R.L.

In all cases canal W.S. elevation 168.0' R.L.

P = Prototype

In order to verify the result of the above study, an undistorted 3-dimensional model study was conducted with excluder tunnels system as supplied by the Design Directorate of Teesta Head works. The salient test results are presented in Table- 2 & Table- 3. The physical features of excluder tunnel system of Teesta Barrage are also given in Table- 1.

**CONCLUSION :**

It is concluded that the hydraulic performance of sediment excluder tunnels system for Teesta Barrage will work satisfactorily and will satisfy the design consideration. There would be little chance of clogging of tunnels.

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# APPLICATION OF MIKE 11 IN DRAINAGE SLUICE DESIGN

BY
**RANAJIT KUMAR DAS***

**ABSTRACT :**

*A large number of drainage-Sluices/regulators are being constructed on river embankments in different parts of Bangladesh. The performance of these structures as well as the design of a proposed structure can be successfully studied applying MIKE 11 mathematical modelling system. In the present study an attempt was made to find the most economical and effective design of a drainage sluice in the tidal zone with respect to drainage capacity and stilling basin velocity applying MIKE. 11. However, the study should be treated as a preliminary study as the model setup was hypothetical.*

## MIKE 11 : A Short Description

MIKE 11 is a one dimensional, one-layer mathematical modelling system developed at the Danish Hydraulic Institute (DHI) and the Water Quality Institute, Denmark for the simulation of flows,sediment transport and water quality in rivers, estuaries, irrigation systems and similar water bodies. It offers a unique and friendly tool for design, management and operation of river basins and channel net works. MIKE 11 simulates unsteady one dimensional flows, transports and bio-chemical reactions in one layer (Vertically homogeneous) fluids. It has a wide range of applications, such as :

—Design of channel systems
—Studies of tides and storm surges in rivers and estuaries
—Design and operation of irrigation and surface drainage systems
—Simulation of flood control measures
—Flood forecasting
—Sedimentation Studies
—Study of salt-intrusion in estuaries

**Object of the present study :**

In the present study, MIKE 11 mathematical modelling system was applied to find the most economical and appropriate design of a drainage sulice-which is usually built into the embankment of a river for the purpose of draining excess rain water from a polder and at the same time to prevent salive water or flood-water from entering into the polder.

For the appropriate design of a drainage sluice, the following points were taken into consideration :

—Location of the drainage sluice
—Width and sill level
—Maximum and average daily precipitation into the polder (particularly in the rainy season)
—The area of the polder
—Hydraulic condition of down-stream
—Velocity on the stilling basin etc.

Location of the sluice should be such that the whole amount of water can be drained out into the nearest river or canal. The width and the sill level of the sluice should be such that it has the capacity of draining out a certain amount of water within a particular time. The amount of precipitation and area of the polder is needed to calculate the amount of water to be drained out. The hydraulic condition of down stream is important for the operation of the sluice (one way or two ways operation). Velocity on the stilling basin is necessary for the design of the stilling basin. MIKE 11 is

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able to study all of these conditions. For this study, the required data were collected from CERP II project, Khulna.

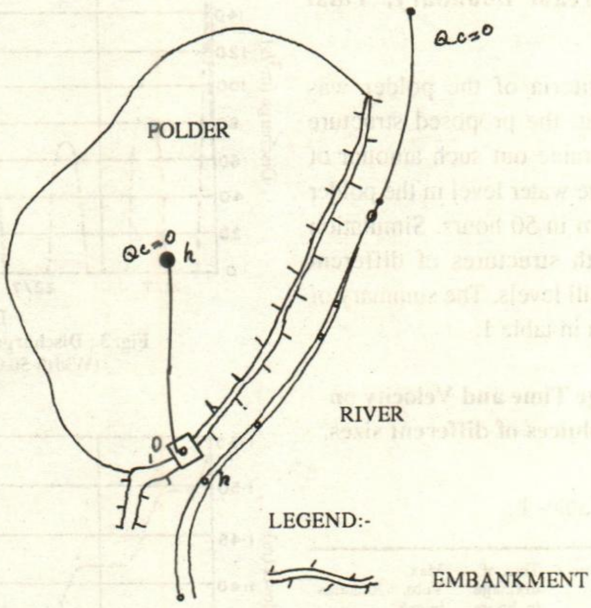
**The Model**

The set-up of the model in shown in fig. 1. It consists of a river branch, a polder of area 11. 0 sq. km. (Cultivable land) with hypohetical boundary, location of the drainage sluice and the river embankment.

**FIG 1: The Model Set-up.**

The polder was connected with the river through a branch. The sluice was Situated at the middle of the branch. A zero discharge boundry ($a_e$=o) was selected for upstreaam boundaries, where as, a tidal variation of water level was set at its down-stream boundary (WT). The nature of the tide was as follows :

| | | |
|---|---|---|
| Time cycle | : | 12 hours |
| Mean level | : | 1.2 m |
| Amplitude | : | 0.8 m |
| HTL | : | 2.0 m |
| LTL | : | 0.4 m |




**BOUNDARY CONDITION:-**

**UP STREAM=QC=(Zero DISCHARGE)**

**DOWN STREAM WT-(TIDAL WATER LEVEL)**

**INITIAL CONDITION:-**

**Q=O WL=1.55m**

FIG. 1: The Model Set-up

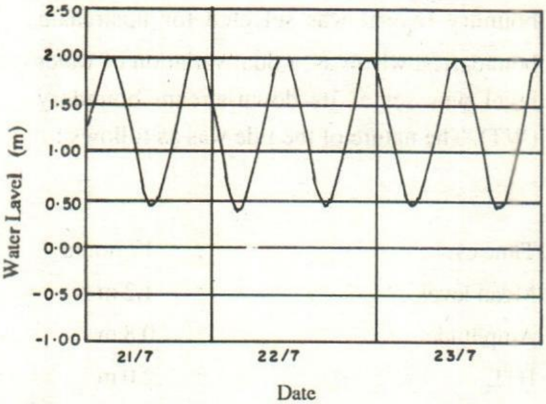



Fig. 2

**FIG. 2 : Downstream Boundary, Tidal Water Level.**

The drainage criteria of the polder was considered such that, the proposed structure should be able to draine out such amount of water as to reduce the water level in the polder from 1.55m to 1.25m in 50 hours. Simulation was carried out with structures of different width and different sill levels. The summary of the results are shown in table 1.

**Table- 1 : Drainage Time and Velocity on Stilling Basin for sluices of different sizes.**

Table- 1

| Sl. No. | Width of the Sluice in metre. | Sill level (m) | Time of drainage in hours | Max. Velo. in m/s | Remarks |
|---|---|---|---|---|---|
| 1. | 30.0 | 0.0 | 72 | 1.8 | |
| 2. | 30.0 | -1.0 | 60 | 1.5 | |
| 3. | 40.0 | -0.0 | 66 | 1.6 | |
| 4. | 40.0 | -1.0 | 55 | 1.4 | |
| 5. | 50.0 | 0.0 | 61 | 1.5 | |
| 6. | 50.0 | -1.0 | 50 | 1.4 | |
| 7. | 60.0 | 0.0 | 55 | 1.4 | |
| 8. | 60.0 | -1.0 | 50 | 1.3 | |
| 9. | 70.0 | -1.0 | 50 | 1.1 | |

## DISCUSSION OF RESULTS.

A sluice of 50m width with sill level - 1.0 m was found to be optimum for the present purpose.

A wider sluice with the same sill level (say 60m or 70m wide) can not reduce the drainage time. The increase in width only can reduce the velocity at the mounth of structure.

However, it was found from another test that instead of one sluice of 60m width, two sluices of 30 m each at different locations can reduce the drainage time fairly. It indicates that the drainage capacity of the adjacent river is also an-important factor for the performance of the sluice.

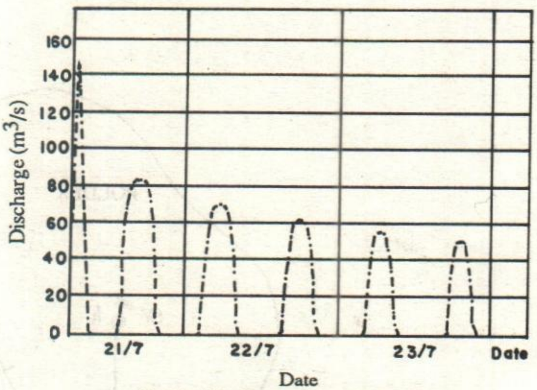



**Fig. 3 : Discharge through the Sluice (Width 50.0 m, S.L. - 1.0)**

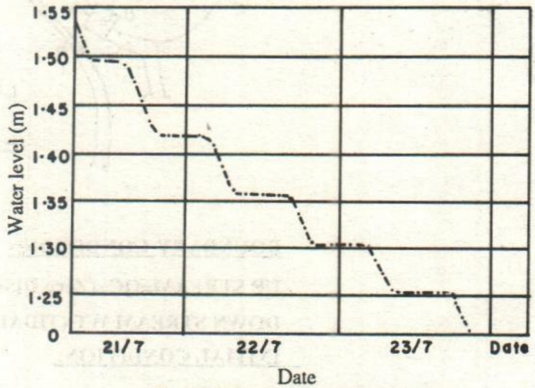



**Fig. 4 : Water level in the polder**

Fig. 3 shows the discharge through the sluice. The a lternate zero discharges are due to tidal variation of water level in the down stream of the sluice. When the tidal water level rises above the water level in the polder, then there will be no discharge through the sluice. In each tidal cycle, when the down stream water level falls upto the lowest Tide level (LTL), the discharge through the sluice is a maximum.

The fig. 4 indicates that drainage of water occurs only when the tidal water level in the downstream is less than that in the polder.

## CONCLUSION

MIKE 11 mathematical modelling system can be applied successfully for the design of a drainage sluice / regulator, even in the tidal zone.

A sluice of 50.0 m width with sill level 1.Om PWD was found most effective for the present case study with respect to drainage time and stilling basin velocity.

The stilling basin velocity for the above case was found 1.4 m/s.

In stead of one sluice of a particular width, two sluices of half the width at two different locations can reduce the drainage time faily. So the later is preferable.

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# EVALUATION OF QUALITY CONTROL OF EARTH STRUCTURES IN BANGLADESH

BY

**A. M. ZAHURAL ISLAM*, MD. ABDUL HAI**, MD. MUJIBUR RAHMAN*****

## ABSTRACT

*Earth structures like embankments, dams, retaining walls, high ways, rail roads and airfields require man placed soil or fill. Failure of embankments and dams may lead to catastrophic damage to land and people. The main reasons of their failure are bank-erosion by flood water, percolation of water through embankment, seepage flow and sliding of soil mass.*

*To put an effective halt to such failures and to satisfy the increasing demand, the required compaction of placement soils is desirable to decrease future settlement, increase shear strength and decrease permeability. The desired compaction is attained in the field by means of suitable compacting devices. Qualitatively speaking, the density of soils beneath a structure should be as high as possible.*

*The bearing capacity of an embankment is a function of its in situ density and increases with the increasing relative density and decreases with the decreasing density. For this reason quality control is necessary during the construction phase of an embankment. The field engineer or an experienced soil technician can conduct the quality control tests namely, the determination of field moisture content and the density of the compacted soils to ensure proper compaction.*

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## INTRODUCTION

The term quality control has become a familiar phrase in recent years and is equally applicable in different fields of industries, engineering and in other spheres. The application of statistical method to quality control for construction works is an old one and is a very vital tool in assessing the quality of the finished products. For this purpose some standard statistical procedures are used. For quality control of earth embankments emphasis is laid upon the following.

Selection of certain properties of the placed embankment.

Measurement of these properties of the placed embankment.

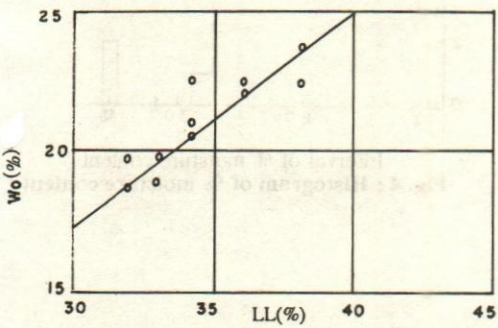
Analysis of the control test of these properties.

Practical limitation of these properties of the acceptable range compatible with design criteria.

In this paper an attempt has been made to evaluate the quality attained in some completed earth structures in Bangladesh.

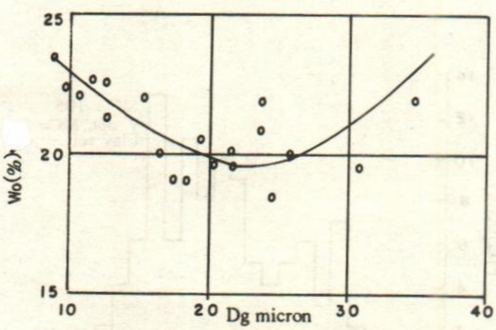
## LABORATORY TEST

In order to determine the maximum dry density and the optimum moisture content of the materials of the embankment, standard or modified Proctor compaction tests were run on the soils under consideration but as these tests are laborious and time consuming, for minor structures these are usually estimated by using the correlations arrived at by simple Identification tests.. The requisite curves have been shown in Figs. 1, 2, & 3. But for major works periodic checking of estimated values must be made with the laboratory compaction test results. The usual specification for field compaction is to attain certain percentage of the maximum dry density attainable by a compacting method like standard or modified Proctor compaction tests.



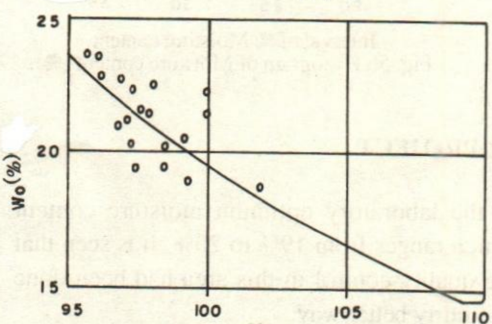

**Optimum moisfure content vs. liquid limit.**

FIG-1




**Optimum moisture content vs. geometric diameter.**

FIG-2




**Optimum moisture content vs. maximum dry density**

FIG-3

In this paper quality control histograms of some project works have been prepared and presented. From the study of quality control it appears that quality control done in the field was fairly good. The reason for deviation from the ideal compaction may be attributed to the fact that in nature there are innumerable number of soils occurring in the same place side by side and probably none of them has all the behaviours similar to these of others. It is also very difficult to keep the moisture in the field at the optimum moisture content as determined by standard or modified Proctor compaction test.

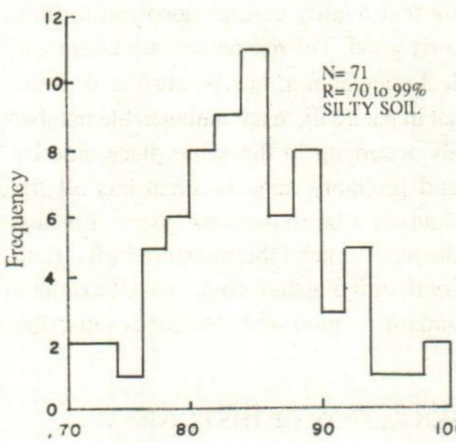
## PRESENTATION OF HISTOGRAM

In this paper several histograms of percent compaction an moisture contents (Figs. 4, 5, 6, 7,) have been presented by carying out site investigation in different areas of Bangladesh by the research personnel of RRI.

## DISCUSSIONS

It appears from Fig. 4 that the percent compaction varies from 70% to 99% and in many areas it did not attain the generally desired values. It is also seen that there has been a wide range of moisture content from 9% to 43% despite the fact that the optimum moisture content in all the areas was 14.4%. It is, however, estimated that if the moisure content in the field would have been rigidly controlled, the better attainment in density could be achieved.

Fig. 5 shows that the percent compaction varies from 84% to 97% and in many areas compaction did not attain the generally desired values and it is also seen that the percent moisture content varies from 18% to 41% whereas percent optimum moisture content was 23.86%. It is desired that if the moisture contents in the field had been brought closer to the laboratory optimum moisture value, the yeild would have been better and commendable.

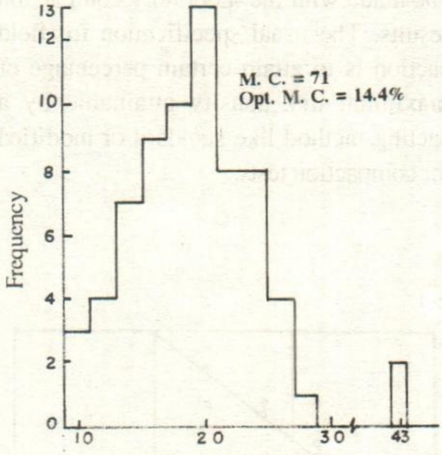



Interval of % compaction

**Fig. 4a : Histogram of % compaction (Moditied AASHO)**

N= No. of compaction test

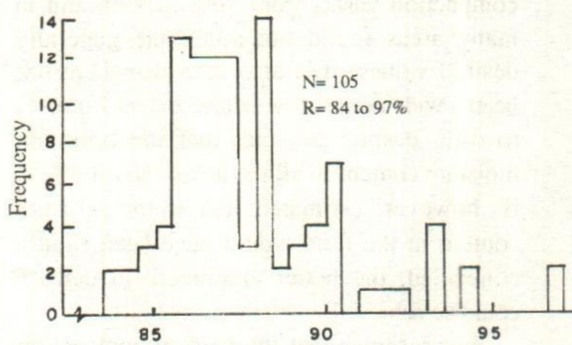
R= Range of % compaction




Interval of % moisture content

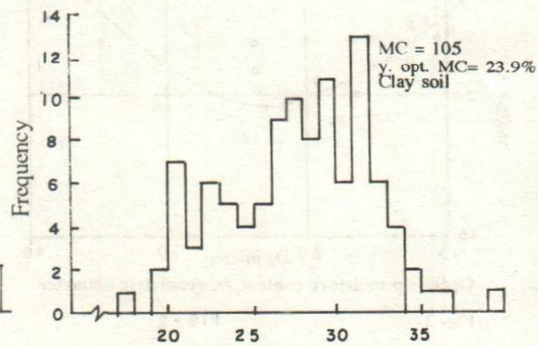
**Fig. 4 : Histogram of % moisture content**

## KARNAFULI IRRIGATION PROJECT




Interval of % compaction

Fig. 5a : Histograme of % compaction (Modified AASHO)




Interval of % Moisture content

Fig. 5b Histogram of Moisture content (%)

## MONU RIVER PROJECT .

From Fig. 6 it appears that percent compaction varies from 82% to 98% and this range of values is very close to the desired attainment of density. It is also seen that the field moisture attainment is more or less nearer to the laboratory optimum moisture content which ranges from 19% to 20%. It is seen that the quality control in this area had been done in a fairly better way.

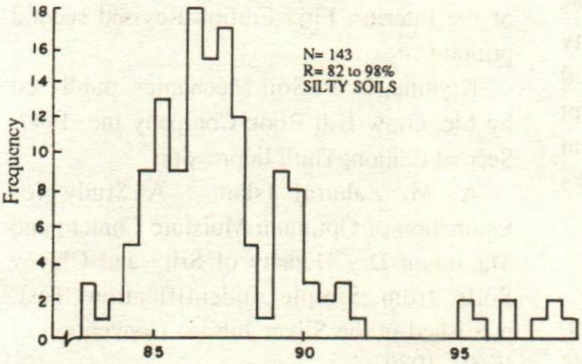



Fig. 6a : Histogram of % compaction (Modified AASHO)

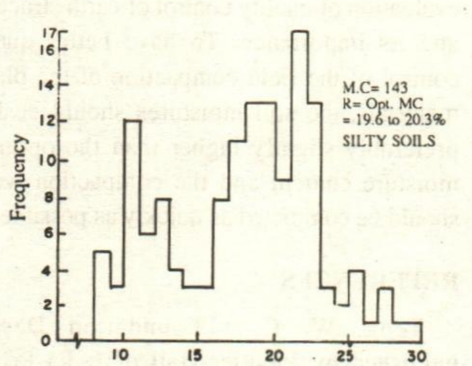



Fig. 6b : Histogram of % moisture content

**MEGHNA DHDNAGODA PROJECT**

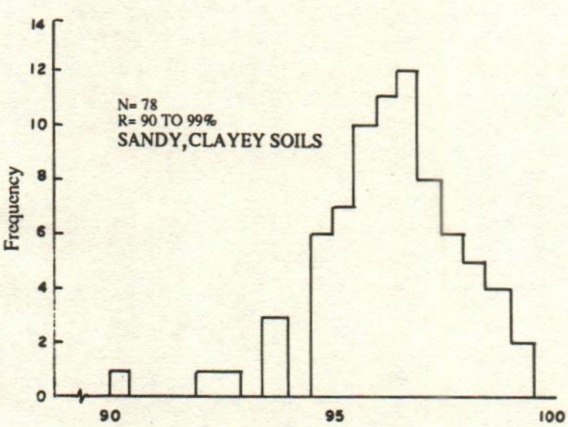



Fig. 7a : Histogram of % compaction (Standard AASHO)

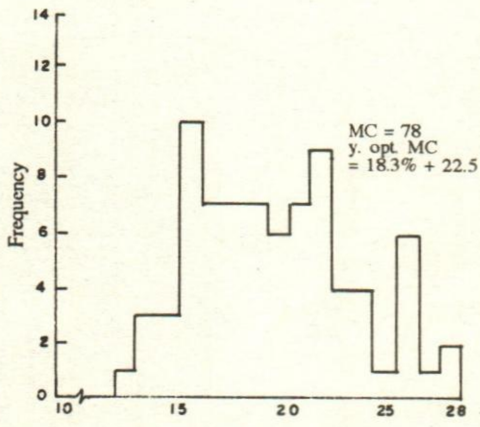



Fig. 7b : Histogram of % moisture content

**MUHURI IRRIGATION PROJECT**

It appears from Fig. 7 that the extent of variation of percent compaction is very small ranging from 90% to 99% and all these areas, so far the data are concerned, compacted to the range of maximum density as desired by the standard AASHO. The range of moisture content seems to be much closer to the percent optimum moisture content which range from 18.26% to 22.50%.

## CONCLUSION

This paper briefly gives a general idea of evaluation of quality control of earth structures and its importance. To have better quality control of the field compaction of the placed materials, the soil moistures should be kept preferably slightly higher than the optimum moisture content and the compaction works should be completed as quickly as possible.

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# SECONDARY COMPRESSION CHARACTERISTICS OF CLAYS FOR SOME AREAS OF BANGLADESH

BY

**A. M. ZAHURAL ISLAM*, PANKAJ KUMAR ROY****

## ABSTRACT

*The consolidation characteristics of foundation soils can be attributed to two distinct phenomena, one results from primary consolidation due to reduction of volume change by out flow of air and water from the soil pores and the other from secondary consolidation due to plastic deformation of the soil mass. Both the phenomena take place under applied constant load. In this paper the secondary consolidation characteristics of the some areas in western region of Jamuna-Padma-Meghna river system have been calculated and some relations developed with the index properties of these soils. The soils of the region consist mostly of fine grained materials and show a wide range of secondary compression caracteristics under a constant load. The value of the co-efficient of secondary consolidation, $C_{\propto}$, is less for sandy silt and stiff clays and relatively high for organic and soft clays.*

## INTRODUCTION

Every material undergoes a certain amount of strain when stress is applied. The compression strain of a foundation soil is attributable to two distinct phenomena within the soil. The first of these types of strain results from reduction in volume due to outflow of water from the soil pores under applied load and is generally referred to as primary consolidation. The 2nd type of strain is the result of plastic deformation of the mass under constant load and is called secondary consolidation. In this paper the secondary consolidation characteristics of the soils of some areas of Bangladesh, specially from some locations of the western belt of Jamuna-Padma-Mehgna river system have been calculated and some relations have been developed with the index properties of these soils.

## GEOLOGICAL BACKGROUND

The landform of Bangladesh is constituted by the alluvial deposit of sediment that has been transported from the Himalayan and sub-Himalayan region to its present location by water and deposited in a wide range throughout the areas of Bangladesh. The main streams or rivers which still carrying on this processes, are the Ganges, the Brahmaputra, Meghna, Teesta, Gumti etc. As the soils of Bangladesh comprised of alluvial deposit varying from gravel to fine grained soil material, the stratified arrangement of soil materials mainly depends on the fluctuation of the stream velocity. Thus in the upper region of Bangladesh the soil is a mixture of coarse and fine grained soils like boulders, gravels, sands and silt, and clay and in the lower region is obviously a deposit of fine grained soils like sand, silt and clay. In the western region of the country the soils mainly consists of sand, silt and clay deposits.

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## PRIMARY CONSOLIDATION

The portion of the total settlement that takes place in accordance with the theory of consolidation is called primary consolidation. The rate of primary consolidation was first derived by Terzaghi for the special case of one dimensional flow from a laterally confined soil. After application of a pressure increment $\Delta P$, the excess pore pressure becomes instanteneously equal to $\Delta P$ everywhere in the soil mass. The gradual out flow of water decreases $\Delta P$ and this process continues until the excess pore pressure is reduced to zero and the applied pressure is entirely supported by the intergranular structure. The void ratio at this point is $e_{100,}$ the void ratio at 100% consolidation.

## SECONDARY COMPRESSION

The additional settlement which occurs after 100% primary consolidation is known as secondary consolidation differs from the rate predicted by the theory of primary consolidation. This difference between observed consolidations and the theoretical primary consolidation is attributed to secondary effect and is defined as secondary compression. The secondary effects are illustrated in Fig. 1, 2. The most notable difference in the curves occurs when the theoretical primary curve approaches to its ultimate value. Then for the laboratory curve a linear relation is observed between void ratio-change and the logarithm of time. The slope of this line, in units of void ratio change per logarithmic cycle of time is called the co-efficient of secondary compression, $C_\propto$ and it is used to measure the magnitude of secondary compression effects. The co-efficient of secondary consolidation is found from the following formula:

$$C_\propto = \frac{\Delta e}{1+eo} \cdot \frac{1}{\log 10\ (t2/t1)} = \frac{Ct}{1+eo}$$

$$\text{where } C_t = \frac{\Delta e}{\log_{10}(t_2/t_1)}$$

the slope of the straight line portion of the e-log t curve and is known as the secondary compression index. Numerically $C_t$ is equal to the value $\Delta e$ for a single cycle of time on the curve. The additional settlement S due to secondary compression can be estimated as $S = 2H\ C_\propto$. Here 2H represents the total thickness of the soil layer for two phase drainage.

When the pore pressure falls to zero at the end of the primary consolidation, the secondary consolidation is governed by the rate of transfer of the absorbed water film to the solid bond. The secondary compression is assumed

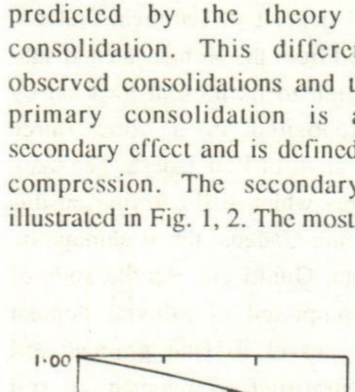


**Fig. 1**

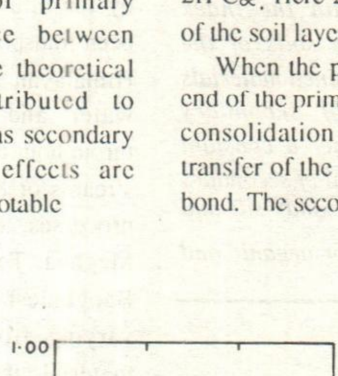
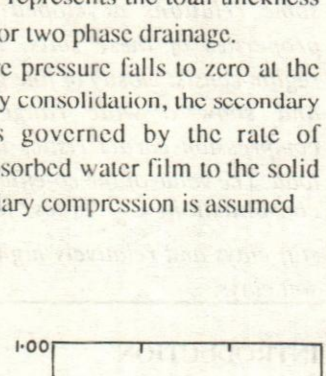


**Fig. 2**



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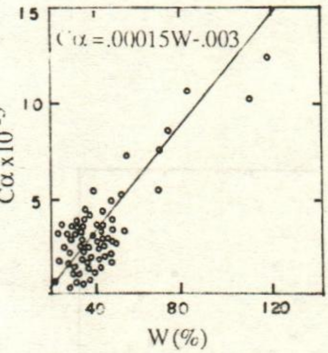


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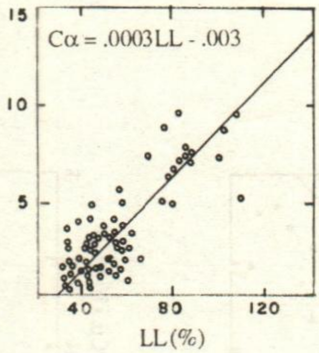


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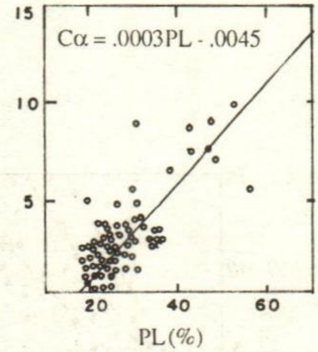
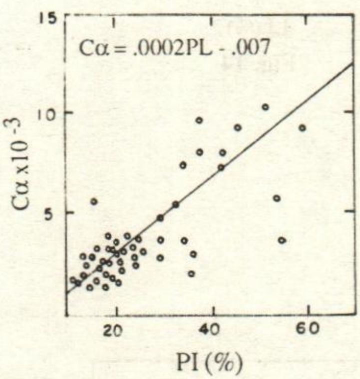



Fig. 6

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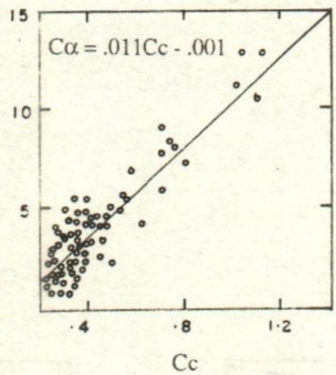


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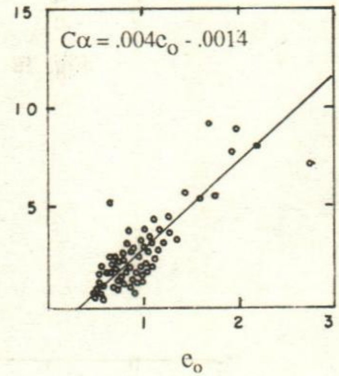


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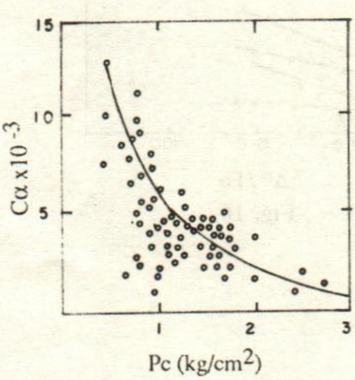


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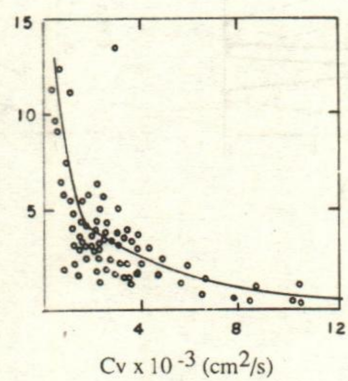


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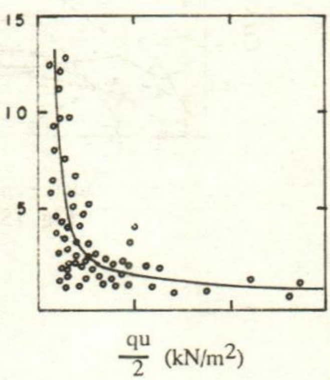


Fig. 12

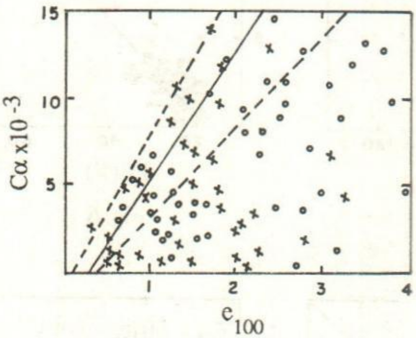


Fig. 13

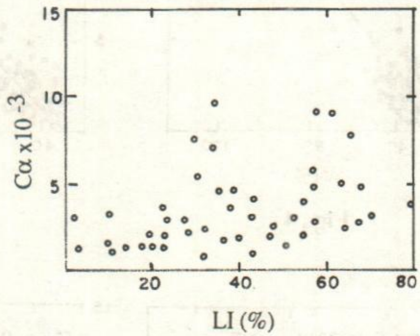


Fig. 14

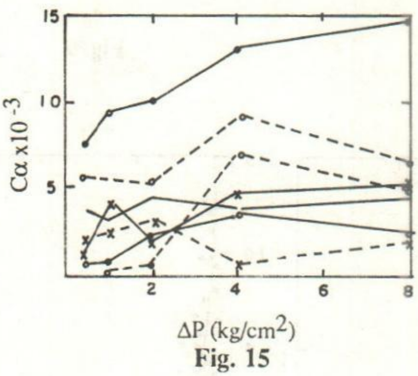


Fig. 15

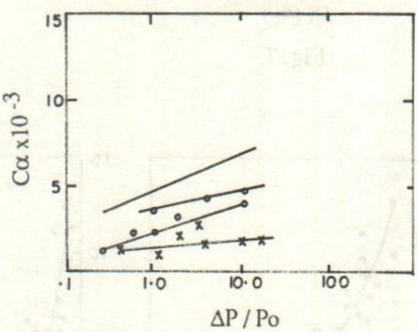


Fig. 16

to be the void ratio change that results from viscous yielding of the grain structure. The effective pressure that the intergranular structure can support depends on the particle orientation as well as the void ratio, Therefore, the viscous re-orientation of the grains gradually reduces the capacity of he intergranular pressure to be transferred to the pore water. This tendency is relieved if the void ratio is reduced slightly. It is assumed that the particle orientation occurs so slowly that the pore pressure developed by the process are negligible. Thus the secondary rate of void ratio change is governed entirely by the rate of viscous yielding and is independent of hydrodynamics.

## LABORATORY INVESTIGATION

In order to study the secondary compression behavior of the soils of some areas of Bangladesh, about one hundred consolidation tests data were reviewed and thereby the co-efficient of secondary compression, $C_{\alpha}$ was calculated. The soil samples had been collected from different location of Rangpur. Pabna, Khulna and Barisal. the samples were collected in 3" to 4" diameter shelby tubes.

**Soil Materials :** The soil materials used in the laboratory tests were classified as SILT, CLAY and ORGANIC CLAYS. The materials of these types were taken as to find out the general secondary compression characteristics of the soils in this region.

From laboratory tests the solid matters ranges by percentage as follows:

| | | |
|---|---|---|
| FINE SAND | - | 0 to 20% |
| SILT | - | 50 to 95% |
| CLAY | - | 5 to 40% |
| Organic matter | - | 0 to 20% |

Also the range of other parameters were as follows :

| | | |
|---|---|---|
| Unit weight (Dry) ($kN/m^3$) | - | 5.35 - 15.71 |
| Natural water content (%) | - | 20 - 141 |
| Liquid limit (%) | - | 33 - 184 |
| Preconsolidation loads ($Kg/cm^2$) | - | .25 - 3.0 |
| Relative Density | - | 2.03 - 2.69 |

**Apparatus and Procedure** : The sizes of the consolidometer rings were 2.54 cm high x 6.35 cm dia and 2 cm high x 5 cm dia. A double lever system and a single lever system consolidation frames were used for the application of loads.

The undisturbed samples were carefully trimmed to fit into the consolidometer ring. Six load increments were applied starting from 0.25 to 8.0 $Kg/cm^2$ of the saturated specimen and each load was allowed to remain in the machine for about 24 hours. Pressure increment varies a little for few samples.

For each pressure increment compression readings were taken at time intervals necessary for accurately plotting the complete compression logarithm of time curve.

Void ratio to 100% consolidation and co-efficient of secondary compression was determined.

## PRESENTATION OF DATA

The values of natural moisture content, liquid and plastic limits, wet and dry densities. relative density, compression index, co-efficient of comparability test results have been tabulated in Table- 1 and the results of pressure increment, compresion dial reaching void ratio at 100% consolidation and the values of Coefficient of secondary consolidation have been shown in Table- 2.

The graphical representation of C against some widely used parameters have been presented as below :

**Table- 1**

SUMMARY OF SOME TEST RESULTS

| Location | Hole No. Sample depth | N. W. C. No. (%) | LL (%) | PI (%) | Density Wet $kN/m^3$ | Density Dry $kN/m^3$ | eo | Relative Density | PC $kg/cm^2$ | Cc | Cv | $C_\infty$ $x10^{-3}$ $cm^2/sec$ | Soil Description |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 |
| Laxmipur | $I_2$ $B_5$ $U_1$ | | | | | | | | | | | | Sandy SILT |
| Pabna | 2o'—22' | 23.0 | 34 | 10 | 19.3 | 15.7 | .664 | 2.66 | 1.7 | .114 | 1.6 | .72 | |
| | 4—$U_1$ 15'—17' | 26.4 | 40 | 15 | 19.3 | 15.3 | .728 | 2.68 | 1.5 | .167 | .31 | 1.34 | Clay SILT |
| Ekbokta Rangpur | 3—$U_1$ 26'-22' | 23.6 | 39 | 20 | 19.1 | 15.5 | .70 | 2.67 | 2.65 | .155 | .32 | 1.6 | -do- |
| Kalatali Khunla | 1—$U_1$ 10'-12' | 39.1 | 44 | 19 | 17.9 | 12.8 | 1.062 | 2.67 | 1.35 | .277 | 3.5 | 2.5 | -do- |
| Jirbunia Khulna | 3—$U_1$ 15'.17' | 46.6 | 51 | .24 | 16.9 | 11.5 | 1.263 | 2.67 | 0.362 | 1.0 | 1.02 | .14 | Silty clay with organic matter |
| Rithein para | 4—$U_1$ 25'-27' | 37.3 | 13 | 17.6 | 12.8 | 1.0515 | 2.67 | 1.3 | 0.188 | 2.9 | 3.1 | | Silty CLAY |
| Gazipara Khulna | 1—$U_1$ 15'-17' | 56.8 | 110 | 54 | 15.7 | 10.0 | 1.575 | 2.64 | 0.88 | .505 | .91 | 5.5 | Organic Clay |
| Shniyer khal Khulna | 2—$U_1$ 20'-22' | 59.7 | 86 | 37 | 14.4 | 9.0 | 1.974 | 2.60 | 1.2 | 0.80 | 1.2 | 7.81 | -do- |
| Jirbunia Khulna | 2—$U_2$ 25'-27' | 141 | 150 | 70 | 12.9 | 5.4 | 3.614 | 2.5 | 0.65 | 1.405 | .37 | 11.3 | -do- |
| Khulna | 2—$U_1$ 16'-12' | 114.2 | 194 | 86 | 13.2 | 6.1 | 2.945 | 2.48 | 0.55 | 1.1325 | .67 | 13.8 | -do- |

**Table- 2**

**Experimental Results of Three Tests**

| Pre-ssure kg/$cm^2$ | Test No. 1: 24 hrs Dial Reading cm | Test No. 1: $e_{100}$ Void ratio | Test No. 1: $C_\alpha$ x$10^{-3}$ | Test No. 2: Dial ing cm | Test No. 2: $e_{100}$ read- | Test No. 2: $C_\alpha$ x$10^{-3}$ | Test No. 3: Dial Read- cm | Test No. 3: $e_{100}$ ing | Test No. 3: $C_\alpha$ x$10^{-3}$ |
|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.00 | - | - | 0.00 | - | - | 0.00 | - | - |
| .25 | .020 | .6485 | .24 | .052 | 1.0816 | 2.02 | .1445 | 3.305 | 4.6 |
| .50 | .035 | .6357 | .81 | .0905 | 1.0388 | 3.62 | .2329 | 3.085 | 7.70 |
| 1.0 | .051 | .6223 | .25 | 1.052 | .9901 | 3.05 | .3615 | 2.785 | 10.14 |
| 2.0 | .075 | .5764 | .51 | .2907 | .8180 | 3.11 | .7365 | 1.906 | 16.66 |
| 4.0 | .105 | .5764 | .51 | .2907 | .8180 | 3.11 | .7365 | 1.906 | 16.66 |
| 8.0 | .1452 | .5434 | 1.47 | .3840 | .7206 | 4.13 | .900 | 1.503 | 15.29 |

Fig. 1. Dial reading Vs. log time in minute for six loads circrements.

Fig . 2 & 3: Typical curves of dial readings $V_s$ log time in minute and square root of time minute for 90% consolidation respectively.

Fig. 4-12 : Co-efficient of secondary compression $C_\alpha$ vs. N.M.C., LL. PL, PI. $C_c$, $e_o$, $p_o$, $c_u$, $s_u$, $e_{100}$, LI, P,$\frac{\Delta P}{P_O}$ in order.

## DISCUSSION

From the observed behavior in Fig. 1,2 & 3 it concluded that the shape of the compression-logarithmic of time curve depends on the relative magnitude of the primary and secondary effects. As the results of the widely used parameters varies to a wide range, average lines were drawn in the curves and the following relations have been made.

**(i) $C_\alpha = 0.0015W - 0.003$**

**(ii) $C_\alpha = 0.0013$ LL - 0.003**

**(iii) $C_\alpha = 0.003$ PL - 0.0045**

**(iv) $C_\alpha = 0.002PI - 0.007$**

**(v) $C_\alpha = 0.011Cc - .001$**

**(vi) $C_\alpha = 0.004\, e_0 - 0.0014$**

**(vii) $C_\alpha = 0.0075\, e_{100} - 0.002$**

It is evident that with the increasing values of LL, PL. $C_c$, $e_o$, $e_{100}$ the corresponding values of $C_\alpha$ increases. Al though the values of these parameters varies widely for the soils of different regions of the western Belt of Jamuna-Meghna, the straight lines may be used to give general relation among the test parameters. The relations of $C_\alpha$ with Pc, Cv and Sü, $\frac{qu}{2}$ are such that with the increasing values of these parameters the values of $C_\alpha$ decreases exponentially. No notable relation has been observed with $C_\alpha$ & liquidity index. Load increment $\Delta P$ and pressure increment ratio $\frac{\Delta P}{P}$ have no significant effect on $C_\alpha$.

## CONCLUSION :

From the foregoing experimental results the following conclusions may be drawn :

The co-efficient of secondary compression $C_\alpha$ is dependent on the void ratio, total pressure, liquid limit, compression index, etc but independent of the pressure increment.

The time required for the rate of secondary compression $C_\alpha$ is directly related to the time required for completion of primary consolidation.

With higher value of $C_\alpha$ , the magnitude of Pc, Cv and decreases rapidly and then slowly.

As the value of $C_\alpha$ is very less for sand silt and stiff clay it may not be considered as a significant parameter for them but for organic and soft clays the value is found to be relatively high.

In the northern portion of the Ganges, as Pabna, Rajshahi, Rangpur the soils are mostly sand, silt and clays, and naturally the value of $C_\alpha$ is also found less. But for the southern region as in some locations of Khulna & Barisal the soils are soft silly and clayey and consisted considerble mount of organic matter and the settlement calculated for secondary compression is found as high as about 96% of the primary consolidation, which may cause a serious effect for the structure.

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# EFFECT OF PARTICLE SIZES OF CLAY AND SILT ON THE COMPRESSIBILTY AND CONSOLIDATION CHARACTERISTICS

BY

**ALIMUDDIN AHMED ***

## ABSTRACT

*The compression index (Cc), coefficient of compressibility ($a_V$) and coefficient of consolidation (Cv) are soil parameters which are used in computation of settlement characteristics of sub-surface soils under structural load and these parameters are usually determined from the conventional laboratory consolidation test. In this paper an attempt has been made to correlate the compression index, coefficient of compressibility and coefficient of consolidation of compressible soils with percent 5 of 2 micoron clay, percent of 5 micron clay and the ratio of percent 5 micron clay and percent silt (74 micron to 5 micron) which are obtained from simple hydrometer test. Some empirical relationships have been indicated mostly for inorganic silts and clays. In this paper soils of Khulna, Barisal, Patuakhali, Jessore, Faridpur and Kushtia districts have been considered.*

## INTRODUCTION

The magnitude and rate of settlement of silt-clay soils caused by consolidation under foundation load are measured by the consolidation test. For this purpose it is necessary to obtain undisturbed soil from various depths and various points of the site. The collection of such sample is costly and the performance of consolidation test on it needs about ten working days for a sample. In this paper a study has been made to estimate soil parameters necessary for computation of settlement from simple hydrometer test on disturbed samples which may be collected comparatively easily and at a small cost.

From laboratory consolidation test the settlement S can be computed from the equations.

$$S = H\frac{Cc}{1+eo}\log10\frac{Po+\Delta P}{Po} \text{ ......................(1)}$$

$$S = H.\Delta P.\frac{a_V}{1+eo} \text{ ..........................................(2)}$$

$$S = H.\Delta P.m_V \text{ ........................................... (3).}$$

where H is the thickness of the bed of silt or clay under a pressure Po and $\Delta P$ is the difference of pressure from Po to Po + $\Delta P$ due to structure load, $m_V$ is the coefficient of volume compressibility and $e_O$ is the initial void ratio. Degree saturation for soils considered in this study is found to about 95% to 100% and may be taken as 100% for all practical purposes. Knowing the values of natural moisture content (W) and specific gravity (G) of a soil, $e_O$ may be computed from the relation $e_O = WG$.

In this paper as attempt has been made to correlate $C_C$, $a_V$, $m_V$ and $C_V$ with % clay (finer than 5 micron), % clay (finer than 2 micron) and ratio of % 5 micron clay and % silt (74 micron- 5 micron) which are determined by simple hydrometer test on disturbed silt and clay samples, which can be collected

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comparatively easily from different cohesive soils strata and at a small cost.

From the knowledge of % 2 micron clay, % 5 micron clay and ratio of 5 micron clay and % silt, the values of compression index and coefficient of compressibility may be estimated from the empirical relations as indicated in this paper.

## ANALYSIS OFEXPERIMENTAL RESULTS

Test data of as many as eighty three undisturbed samples from different areas of Khulna, Barisal, Patuakhali, Jessore, Faridpur and Kushtia in connection with different projects of Water Development Board were selected and studied to correlate the grain sizes with the consolidation test parameters.

The range of values of different soil mechanics tests results of the samples as considered in this study have been shown in **Table- 1.**

The statistical analysis was performed to establish the relation between (i) compression index and percent 2 micron clay. (ii) compression index and percent 5 micron clay and (iii) compression index and ratio of percent 5 micron clay and percent silt. The equations of the regression lins for the problems were developed having calculated the correlation coefficient and the standard deviations of the variables from their combinations and squares and cross products of the difference between variables and their respective mean.

**Table- 1**

| Soil Parameters | Khulna | Barisal | Jessore | Faridpur | Kushtia |
|---|---|---|---|---|---|
| | Values in the range of | | | | |
| % Sand (200-74 micron) | 1-25 | 0-24 | 1-29 | 1-10 | 0-27 |
| % Silt (74-5 micron) | 52-83 | 51-83 | 31-79 | 41-84 | 33-85 |
| % Clay (Finer than 5 micron) | 14-43 | 9-35 | 7-67 | 15-56 | 7-66 |
| % Clay (Finer than 2 micron) | 4-21 | 4-22 | 4-37 | 4-32 | 4-37 |
| Liquid limit (%) | 37-103 | 37-83 | 34-109 | 34-82 | 35-69 |
| Plasticity index (%) | 11-15 | 6-27 | 10-29 | 8-48 | 5-40 |
| Compression index Cc | 0.179-1.00 | 0.14-0.91 | 0.14-0.457 | 0.145-0.532 | 0.12-0.51 |
| Coeff. of compressibility $a_v$ (X $10^{-2}$ $ft^2$/ton) | 1.84-34.00 | 2.03-15.08 | 1.72-7.86 | 2.16-8.37 | 0.39-8.0 |
| Coeff. of consolidation $C_v$(X $10^{-4}$ $cm^2$/sec.) | 5.0-172.0 | 8.0-663.0 | 11.0-90.0 | 2.66-158.0 | 2.46-566.00 |
| Initial void ratio, $e_o$ | 0.87-2.681 | 0.88-2.24 | 0.66-1.570 | 0.75-1.538 | 0.670-1.080 |
| Natural moisture content (%) | 27-66 | 33-84 | 22-119 | 27-57 | 24-104 |
| Wet density, (PCF) | 110-116 | 110-118 | 115-119 | 111-117 | 116-119 |
| Specific gravity | 2.625-2.70 | 2.583-2.730 | 2.55-2.70 | 2.635-2.750 | 2.66-2.684 |

**Table- 2** shows the relationships developed between variables by statistical analysis.

**Percent 2 Micron Clay vs. Compression Index.**

The values of compression index of different silt and clay soils increase consistently with increasing % clay (finer than 2 micron) as shown in Figure 1A. The abscissas of the points shown in the diagram represent % 2 micron clay and the ordinates the corresponding values of Cc for different silt-clays. All the points are located close to a straight line with the equation.

Cc = 0.0088 Cf + 0.128 ......................(3)

The correlation coefficient is found to be 0.52.

**Percent 5 Micron Clay vs. Compression Index.**

percent 5 micron clay of different silt and clay soils were plotted against corresponding values of compression index and the following statistic relation is obtained as shown in **Figure 1B.**

Cc = 0.0034 Cl + 0.156 ......................(4)

The correlation coefficient is about 0.62. It is seen that increasing values of compression index are associated with increasing values of 5 micron clay.

**Ratio of % 5 Micron Clay and % Silt vs. Compression Index**

In Figure- 1C is shown graphical presentation of ratio of % 5 micron clay and % silt against compression index. It is also observed that the values of compression index increase with increasing values of ratio of 5 micron clay and % silt. The straight line relationship is obtained by statistical analysis which is as follows :

Cc = 0.133 Cl/S + 0.19 ........................... (5)

where Cl/S indicates ratio of clay and silt fractions. The correlation coefficient is about 0.56.

From the knowledge of % 2 micron clay, % 5 micron clay and ration of % 5 micron clay and % silt the values of compression index can be obtained roughly from equations (3), (4)

**Table- 2**

| Soil Parameter | Statistical Relationships |
|---|---|
| Compression index ($C_c$) and % 2 micron clay, (Cf) | $C_c$ = 0.0088 Cf + 0.128; r = 0.52; $s(C_c)$ = 0.101 and s(Cf) = 5.95 |
| Compression index and % 5 micron clay (Cl) | $C_c$ = 0.0034 Cl + 0.156;r = 0.62; s(Cl) = 10.95 |
| Compression index and % clay / % silt (Cl/S) | $C_c$ = 0.133 Cl/S + 0.19; r = 0.56;s(Cl/S) = 0.413 |
| Coeff. of compressibility ($a_v$) and % clay (2micron) | $a_v$ = 0.039 Cf + 3.58; r = 0.25; $s(a_v)$ = 1.83 X 16 |

**Note of abbreviations :**

r = correlation coefficient
s = standard deviation.
s = % Silt
Cl = % 5 micron Clay
Cf = % 2 micron Clay
Cl/S = ratio of % 5 micron clay and % silt.

$s(C_c)$ = Standard deviation of $C_c$
$s(C_f)$ = " " of $C_f$
$s(C_l)$ = " " of $C_l$
$s(C_l/S)$ = " " of $C_l/S$
$s(a_v)$ = " " of $a_v$

and (5). From compression index so obtained, the estimation of settlement of the strata under structural load may be made without performing consolidation test.

clay, compression ration and % 5 micron clay and compression ration and ratio of % 5 micron clay and % silt by plotting the values of silt and clay soils and three straight line relationships have been obtained as shown in

Table- 3

| Composition of Soils | | | Compression index Cc | | | | |
|---|---|---|---|---|---|---|---|
| % Cf | % Cl | % Cl/% S values | Tested Values | Estimated Equation (3) | Values Equation (4) | Equation (5) | Average estimated values |
| 19.5, (Kushtia) | 35.5, | 0.53 | 0.253 | 0.299 | 0.296 | 0.260 | 0.276 |
| 17.0, (Khulna) | 31.0 | .45 | 0.270 | 0.277 | 0.261 | 0.249 | 0.262 |
| 16.0 (Barisal) | 35.0 | 0.57 | 0.367 | 0.268 | 0.275 | 0.265 | 0.269 |
| 11.0 (Jessore) | 26.0 | 0.37 | 0.190 | 0.224 | 0.244 | 0.239 | 0.235 |
| 9.0 (Faridpur) | 27.0 | 0.38 | 0.218 | 0.207 | 0.247 | 0.240 | 0.231 |

**Comparison of Tested values estimated values of compression index from equations (3), (4) znd (5)**

A comparison of the values of compression index obtained by actual consolidation test and estimated from the derived equations is shown in *Table- 3*

***Compression Ration vs % Clay (2 & 5 micron) and Ration % Clay & % Silt.***

Another attempt was made to correlate compression ration $\frac{Cc}{1+eo}$, and % 2 micron

Figures **2A, 2B, 2C**. From these relationships the values of compression ration may be estimated with the knowledge of % 2 micron clay, % 5 micron clay and ratio of % 5 micron clay and silt by conducting grain size analysis on disturbed samples without running consolidation test on undisturbed samples.

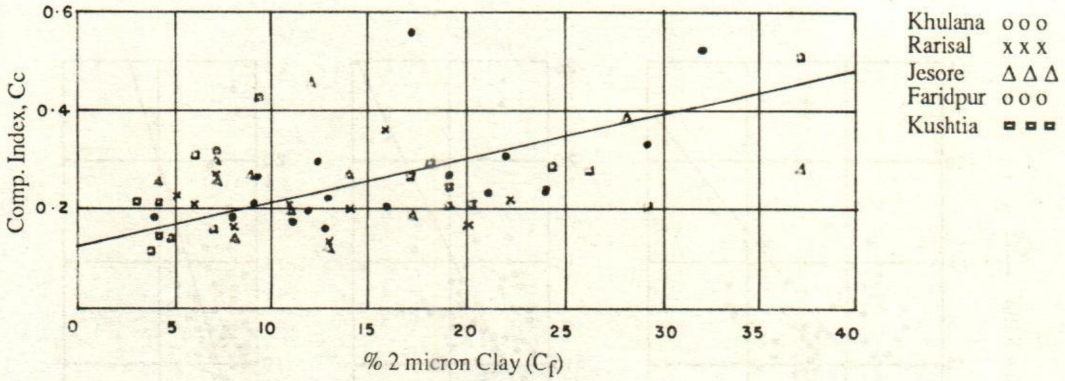


**Fig. 1a : Relation between compression index (Cc) and % 2 micron clay**

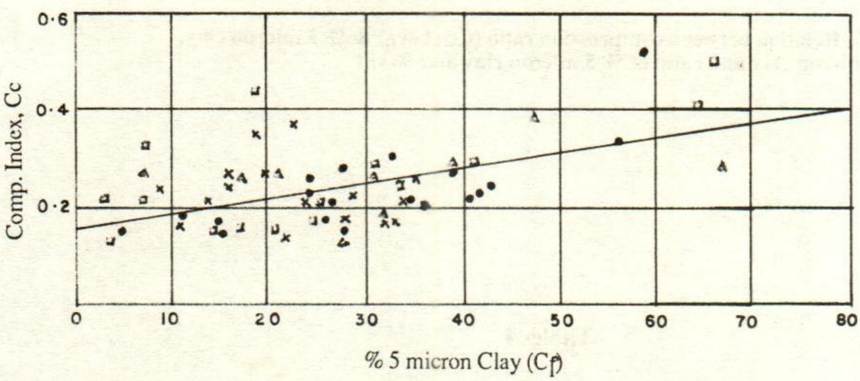


**Fig. 1b : Relation between compression index (Cc) and % 5 micron clay**

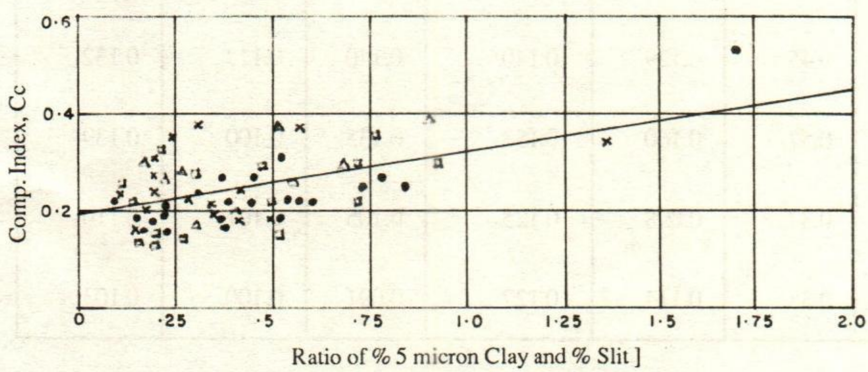


**Fig. 1c : Relation between compression index (Cc) and Ratio of % 5 micron clay & % silt (74 micron)**

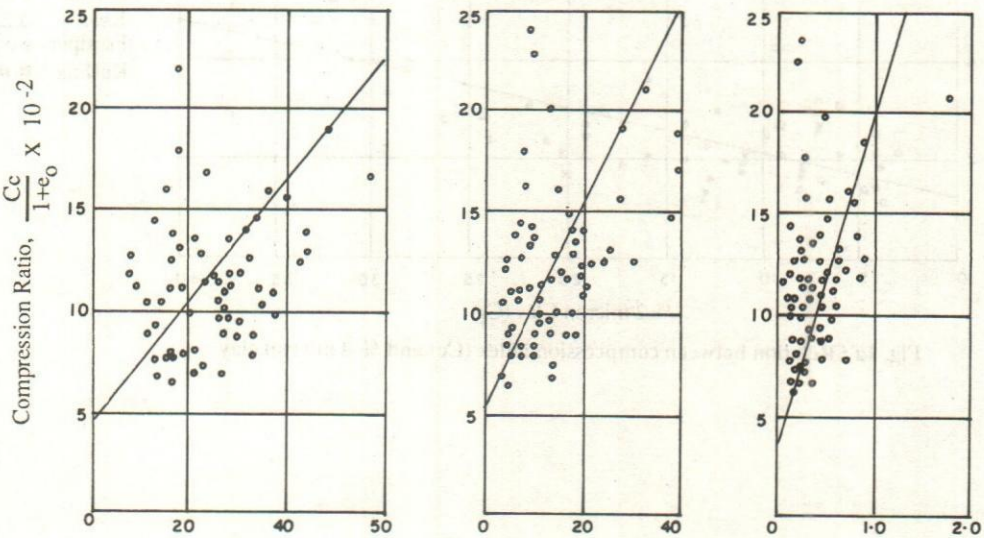


**Fig. 2 : Relation between compression ratio ($Cc/(1+e_0)$) & % 5 micron clay, % 2 micron clay and ratio of % 5 micron clay and % silt**

**Table- 4**

| Composition of Soils | | | Compression ratio = $\frac{Cc}{1+eo}$ | | | | |
|---|---|---|---|---|---|---|---|
| % Cf | % Cl | %Cl/% S | Tested values | Estimated values Graph A | Graph B | Graph C | Average estimated values |
| 19.5 (Kushtia) | 33.5 | 0.53 | 0.122 | 0.150 | 0.150 | 0.125 | 0.141 |
| 17.0 (Khulna) | 31.0 | 0.45 | 0.129 | 0.140 | 0.140 | 0.117 | 0.132 |
| 16.0 (Barisal) | 35.0 | 0.57 | 0.160 | 0.153 | 0.135 | 0.100 | 0.139 |
| 11.0 (Jessore) | 26.0 | 0.37 | 0.098 | 0.125 | 0.105 | 0.101 | 0.110 |
| 9.0 (Faridpur) | 27.0 | 0.35 | 0.114 | 0.127 | 0.091 | 0.100 | 0.103 |

**Comparison of tested values of compression ratio and that of estimated values from graphs as in Fig 2A, 2B & 2C**

A comparison of values of compression ratio determined from actual test results and estimated from the graphs of Fig. 2A, 2B and 2C has been shown in Table- 4 and the samples were selected at random.

**Coefficient of Compressibility vs. % 2 & 5 micron Clay and % clay / % silts**

The values of % 2 micron clay, % 5 micron clay and ratio of % clay and % silt are plotted against the values of coefficient of compressibility and the average lines as shown in **Fig. 3A. 3B & 3C** have been drawn. It appears that the values of coefficient of compressibility tend to increase with increasing values of % clay (2 & 5 micron) and % clay / % silt. The estimation of coefficient of compressibility from physical compositions as discussed is this paper is not encouraging. However, the author intends to make further study on this point considering the results of soil samples from selected areas and calculating av from change in void ratio with respect to small pressure difference instead of 4 of 6 ton/ft$^2$ as considered in this study. In **Table- 5** is shown a comparison between tested results of coefficient compressibility and calculated results from the graphs as shown in **Fig. 3B and 3C.**

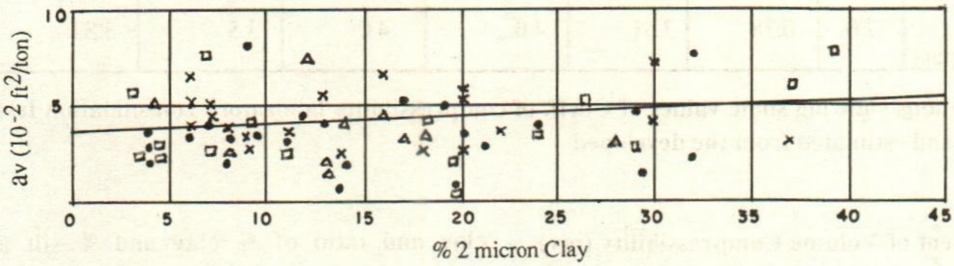


**Fig. 3a : Relation between co-efficient of compressibility ($a_v$) and % 2 micron clay**

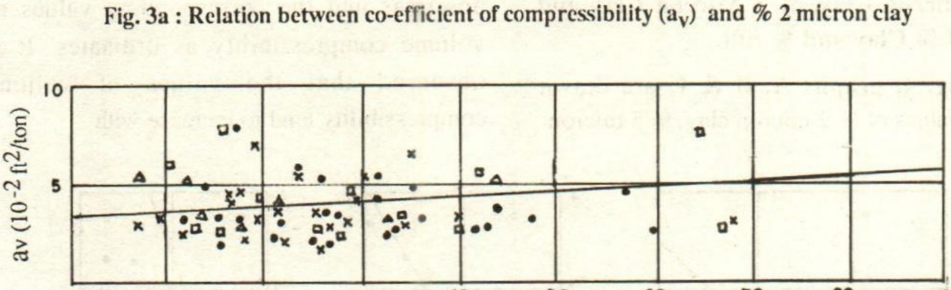


**Fig. 3b : Relation between co-efficient of compressibility ($a_v$) and % 5 micron clay**

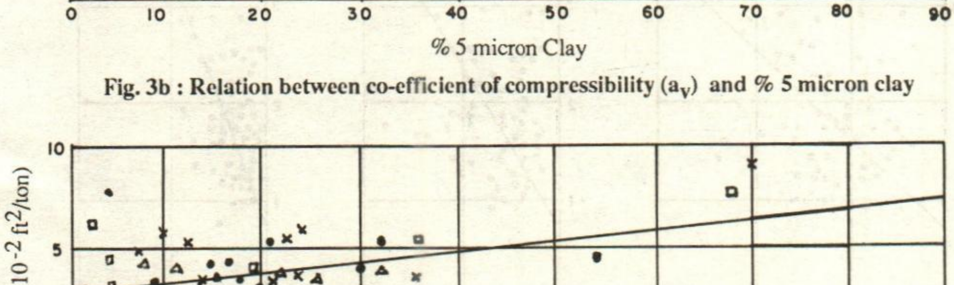


**Fig. 3c : Relation between co-efficient of compressibility ($a_v$) and % 5 micron clay and % silt**

**Table- 5**

| Soil Parameters | | | Coeff. of compressibility $a_v$ ($1X10^{-2}ft^2/ton$) | | | | |
|---|---|---|---|---|---|---|---|
| % Cf | % Cl | %Cl/%S | Tested values | Estimated Values Graph 3A | Graph 3B | Graph 3C | Average of estimated values. |
| 26.5 (Kushtia) | 39 | 0.73 | 5.14 | 4.6 | 4.3 | 4.2 | 4.36 |
| 17.0 (Khulna) | 31.0 | 0.45 | 5.36 | 4.3 | 4.2 | 3.6 | 4.03 |
| 16.0 (Barisal) | 35.0 | 0.57 | 6.74 | 4.2 | 4.2 | 4.0 | 4.13 |
| 11.0 (Jessore) | 26.0 | 0.37 | 2.93 | 4.0 | 4.0 | 3.5 | 3.83 |
| 9.0 (Faridpur) | 27.0 | 0.38 | 3.51 | 4.0 | 4.0 | 3.5 | 3.83 |

**Illustrations showing some values of Coeff. of compressibility laboratory consolidation tests results and estimated from the developed**

**Coefficient of Volume Compressibility (mv) vs % Micron Clay, % 5 Micron Clay and Ratio of % Clay and % Silt.**

In **Fig. 4. graphs A, B & C** are drawn having values of % 2 micron clay, % 5 micron clay and ratio of % clay and % silt as abscissas and the corresponding values of volume compressibility as ordinates. It is observed that the values of volume compressibility tend to increase with

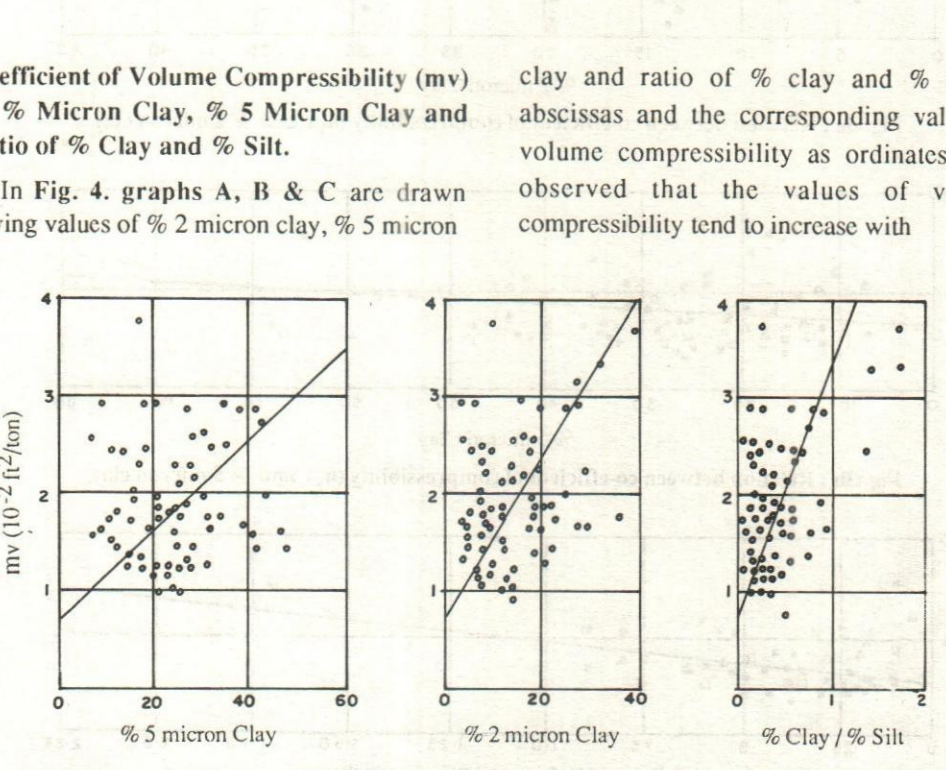


**Fig. 4 : Relation between co-efficient of volume compressibility ($m_v$) and % 5 micron clay, % 2 micron clay and % 5 micron clay/% silt**

increasing values of % 2 micron clay % 5 micron clay and ratio of % clay and % silt. From these graphs rough estimate of volume compressibility may be made by grain size analysis on disturbed soil samples.

In **Table 6** is shown the comparison of coefficient of volume compressibility from actual test and computed from graphs obtained. The soil parameters for comparison are selected at random.

**Table- 6**

| Soil parameter | | | Coeff. of volume compressibility ($1X10^{-2}$ $ft^2$/ton) | | | | |
|---|---|---|---|---|---|---|---|
| | | | Tested | Estimated Values | | | Average |
| % Cf | % Cl | % Cl/% S | values | Graph A | Graph B | Graph 3 | estimated values |
| 26.5 (Kushtia) | 39 | 0.73 | 2.84 | 2.55 | 2.95 | 2.65 | 2.71 |
| 17.0 (Khulna) | 31.0 | 0.45 | 2.57 | 2.10 | 2.15 | 1.95 | 2.06 |
| 16.0 (Barisal) | 35.0 | 0.57 | 2.90 | 2.35 | 2.05 | 2.25 | 2.21 |
| 11.0 (Jessore) | 26.0 | 0.37 | 1.52 | 1.90 | 1.65 | 1.65 | 1.73 |
| 9.0 (Faridpur) | 27.0 | 0.38 | 1.84 | 1.95 | 1.50 | 1.75 | 1.73 |

Illustrations showing some values of coeff. of volume compressibility from laboratory consolidation tests results and estimated from graphs developed.

**Coefficient of Consolidation versus % 5 Micron Clay % 2 Micron Clay and Ratio of % Clay and % Silt.**

An attempt has been made to correlate between coefficient of consolidation and % 5 micron and % 2 micron clay of % clay and % silt be plotting their values and all these curves A, B and C of **Fig. 5** appear to be concave upwards and there is a vide scattering of points at given % 5 micron and % 2 micron clay and % clay % silt. However it will provide a range of values. As there is a trend of relation between these parameters the author intends to make some further study taking into consideration of tests results of soil samples from limited areas.

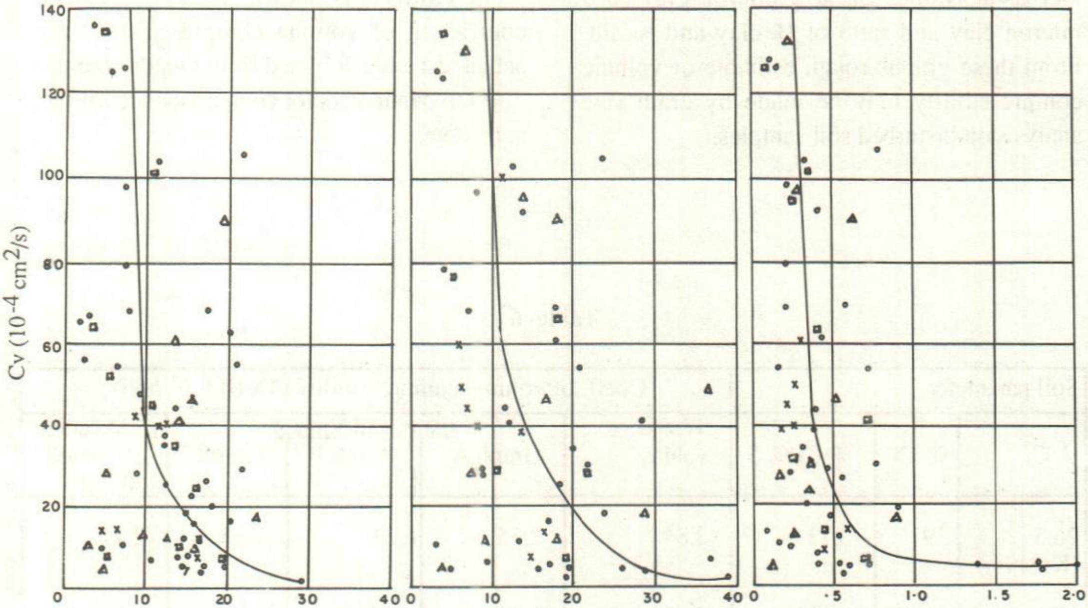


**Fig. 5 : Relation between co-efficient of consolidation ($C_V$) and % 5 micron clay % 2 micron clay and % 5 micron clay/% silt**

## CONCLUSIONS

The empirical equations (3), (4) and (5) for Cc as obtained for silt and clay soils of the south west region of Bangladesh may be conveniently used to estimate settlement from grain size analysis on disturbed samples without running the expensive and time consuming consolidation test for minor foundation and to help minimising the number of consolidation test.

The graphical relationships in Fig. 2A, 2B

A rough estimation of $a_V$ may be obtained using relations as in Fig. 3A, 3B and 3C in computation of settlement from grain size analysis of silt and clay soils. A direct estimation of my in computation of settlement may be done from the relation as shown in Fig. 4A, B and 4C simply running the grain size analysis.

The study reveals that the estimation of Cc is more reliable than that of $a_V$ from mechanical analysis.

The empirical relations so fro obtained during this study are applicable to inorganic silt and clays.

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*Technical Journal, RRI, July 1991*

# STUDY OF CORRELATION OF 28 DAYS COMPRESSIVE STRENGTH OF CONCRETE WITH THOSE OF 3 & 7 DAYS

BY

**A. M. ZAHURAL ISLAM *, MD. ASHRAFUL BARI ** & MD. AFTABUDDIN*****

## INTRODUCTION

Concrete which is one of the most important construction materials is a hardened mixture of cement, water and aggregates (sand, stone or brickchips). It gives strength with continued hydration. The rate of development of strength is quicker at earlier ages than at the later ages. Though concrete develops strength beyond 28 days, for design purposes it is customary to assume 28-day strength as the full strength of concrete. Both in the construction work and running of trial design mixes of concrete it is not only time consuming and uneconomic but also risky to wait for 28 days to confirm whether or not the design strength is achieved. To overcome the situation, many research works have been carried out in different countries to correlate 28-day strength with those of 3 and 7-days strength. Due to many factors such as nature and composition of the contents of concrete, mode of preparation, variation of climatic condition, etc, the suggested empirical relationships vary widely.

In Germany the relation between 18-day and 7-day compressive strengths is taken to lie between :

$$f_{28} = 1.4f_7 + 150$$
$$f_{28} = 1.7f_7 + 850$$

Another suggested relation is :

$$f_{28} = k_2 (f_7)k_1$$

where $k_1$ ranges from 0.3 to 0.8 and $k_2$ from 3 to 6.

The number within parenthesis indicates references.

Troxell and Devis stated that in America 3 and 7-days compressive strengths are assumed to be 38-40% and 62% respectively of the 28-day strength at $70^0$ F while as per USBR. it is 40% and 62% respectively.

Shetty opines that for ordinary portland cement with water-cement ratio of 0.40 to 0.80, the percentages of 3 and 7-days strengths relative to 28-day strength are respectively 40-56% and 68-80%.

In this paper an attempt has been made to derive some empirical relations between $f_{28}$ and $f_7$ & $f_3$ under both field and laboratory conditions using shingles and brickchips as coarse aggregates for conditions prevalent in Bangladesh.

## LABORATORY AND FIELD STUDIES

In connection with different project works of BWDB quite a good number of 6"X 12" concrete cylinders were prepared under both laboratory and field conditions by RRI personnel using Type- I Portland cement, sand of different FM and 1.5" down-graded shingles or brickchips and tested in the concrete laboratory of the Institute from time to time. The details about the studies are available in the internal reports of RRI

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## PRESENTATION OF TEST DATA

For the purpose of analysis the test results have been divided into of the following three classes :

6" X 12" concrete cylinders prepared and tested in the laboratory using shingles as coarse aggregates (Table- 1).

6" X 12" concrete cylinders prepared and tested in the laboratory using brickchips as coarse (Table- 2).

6" X 12" concrete cylinders prepared in the field and tested in the laboratory using shingles as coarse aggregates (Table- 3).

The frequency distributions of the test results for the specimen under (a), (b) & (c) have been shown in Figs. 1, 2 & 3 respectively along with corresponding normal distribution curves. The values of $f_3$ and $f_7$ for samples under (a) and those of $f_7$ for samples under (b) and (c) have been plotted against $f_{28}$ in Figs. 4 & 5 arithmetic paper and in Figs. 6, 7 & 8 in log-log paper. The linear and exponential relationships as obtained have also been shown against each of the respective curves.

The summary of the values of average strength, average relative strength in percent, standard deviation, co-efficient of correlation and standard error of estimate have been provided in Table- 4.

## DISCUSSION OF TEST RESULTS AND COMMENTS

It will appear from the values in Table- 4 that although the data analysed contained strength as low as about 800 psi to as high as over 5000 psi, except correlation between $f_3$ and $f_{28}$, all other relations obtained in the study are quite reasonable. A comparative statement of the values obtained in the study to that suggested by different authors have been shown in table- 5.

In some cases the values seem to be apparently a bit higher which may be attributed to the climatic conditions and other factor prevaiLing in the country. For rough estimate 28-day strength can be predicted from average strength of 3- & 7-days multiplying by the factors 2.10 and 1.46 respectively. It will also

**Table- 1**

| $f_3$ | $f_7$ | $f_{28}$ | $f_3/f_{28}$(%) | $f_7/f_{28}$(%) |
|---|---|---|---|---|
| 1343 | 2015 | 3005 | 44.7 | 67.1 |
| 1438 | 2086 | 3157 | 45.6 | 66.1 |
| 790 | 1260 | 2062 | 38.3 | 61.1 |
| 848 | 1379 | 2262 | 37.5 | 61.0 |
| 2144 | 2911 | 4030 | 53.2 | 72.2 |
| 2015 | 3028 | 4195 | 48.0 | 72.1 |
| 2180 | 3052 | 4112 | 53.0 | 74.2 |
| 2168 | 3299 | 4442 | 48.8 | 74.3 |
| 2350 | 3715 | 5129 | 46.0 | 72.4 |
| 2439 | 3538 | 5129 | 47.6 | 69.0 |
| 2580 | 4138 | 5553 | 46.5 | 74.5 |
| 2545 | 4138 | 5517 | 46.1 | 75.0 |
| 3181 | 4634 | 5765 | 55.2 | 80.4 |
| 3110 | 4598 | 5765 | 54.0 | 79.8 |

**Values of 3, 7 and 28 days compressive strength of 6" X 12" concrete cylinders prepared in the laboratory using shingles as coarse aggregates along with relative values of 3- and 7-days compressive strength in percent of 28-day strength.**

Table- 2

| $f_7$ | $f_{28}$ | $f_7/f_{28}(\%)$ | $f_7$ | $f_{28}$ | $f_7/f_{28}(\%)$ |
|---|---|---|---|---|---|
| 2316 | 3537 | 65.4 | 2884 | 4369 | 66.0 |
| 1999 | 2776 | 72.0 | 3025 | 4708 | 64.0 |
| 3113 | 4298 | 77.0 | 3395 | 4510 | 75.2 |
| 2688 | 4175 | 64.3 | 2334 | 3555 | 65.5 |
| 3396 | 4528 | 75.0 | 2351 | 3781 | 62.1 |
| 4599 | 5307 | 86.6 | 2635 | 3980 | 66.2 |
| 2157 | 3219 | 67.0 | 2529 | 3502 | 72.2 |
| 2122 | 3272 | 64.8 | 1857 | 2705 | 68.6 |
| 2334 | 3502 | 66.6 | 1750 | 2865 | 61.0 |
| 1415 | 2133 | 64.5 | 2281 | 3449 | 66.1 |
| 1380 | 2157 | 63.9 | 1362 | 2104 | 64.7 |
| 1344 | 2087 | 64.3 | 1273 | 2334 | 54.5 |
| 2198 | 2972 | 73.9 | 1486 | 2422 | 61.3 |
| 1945 | 3148 | 61.7 | 1061 | 1999 | 53.0 |
| 2334 | 3343 | 69.8 | 972 | 1734 | 56.0 |
| 2416 | 3267 | 73.9 | 1133 | 1398 | 81.0 |
| 4157 | 5395 | 77.0 | 1133 | 1680 | 67.4 |
| 2865 | 4316 | 66.3 | 1928 | 2948 | 65.4 |
| 2900 | 3927 | 73.8 | 2511 | 3657 | 68.6 |
| 2386 | 3609 | 66.1 | 2458 | 4033 | 60.9 |
| 2264 | 3449 | 65.6 | 2511 | 4227 | 59.4 |
| 2650 | 3753 | 70.6 | 2794 | 4528 | 61.7 |
| 2299 | 3166 | 72.6 | 3095 | 3750 | 82.5 |
| 2064 | 3201 | 64.6 | 2653 | 4015 | 66.0 |
| 1875 | 2653 | 70.6 | 3025 | 4561 | 66.3 |
| 2263 | 3715 | 60.9 | 3272 | 5272 | 62.0 |

**Values of 7 and 28 days compressive strength of 6" X 12" concrete cylinders prepared in the laboratory using brickchips as coarse aggregates along with relative values of 7-day compressive strength in percent of 28-day strength.**

Table- 3

| f7 | f28 | f7/2f28(%) |
|---|---|---|
| 2215 | 3830 | 57.8 |
| 1868 | 2993 | 62.4 |
| 2934 | 4224 | 69.4 |
| 2969 | 4135 | 70.7 |
| 3476 | 4960 | 70.0 |
| 2663 | 4130 | 64.4 |
| 3093 | 4843 | 63.8 |
| 3234 | 4607 | 70.2 |
| 2539 | 4053 | 62.6 |
| 3343 | 4772 | 70.0 |
| 3140 | 4218 | 74.4 |
| 2704 | 4312 | 62.7 |
| 3429 | 4454 | 76.9 |
| 3240 | 4724 | 68.5 |

**Values of 7 and 28 days compressive strength of 6" X 12" concrete cylinder prepared in the field using shingles as coarse aggregates along with relative values of 7-day compressive strength in percent of 28-day strength.**

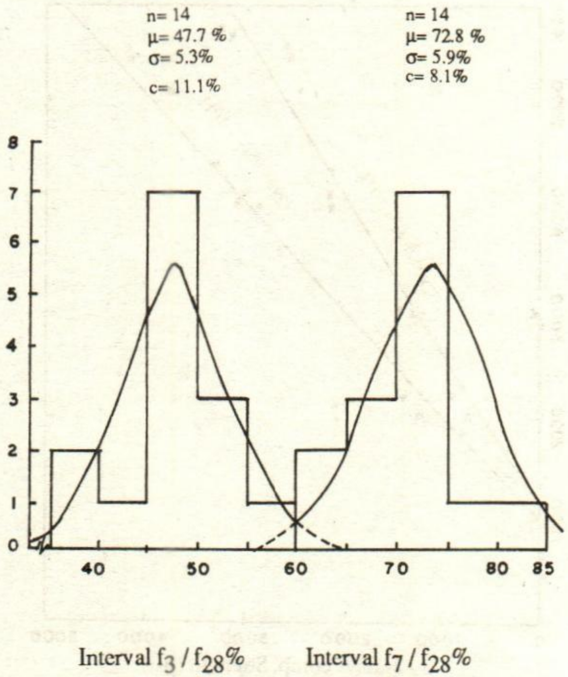


**Fig. 1 : Lab. spcimen with shingles**

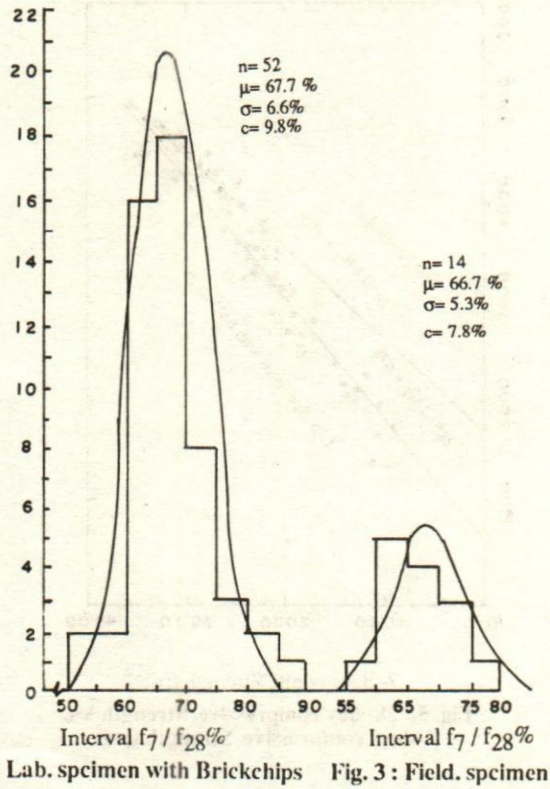


**Fig. 2 : Lab. spcimen with Brickchips** **Fig. 3 : Field. spcimen with shingles**

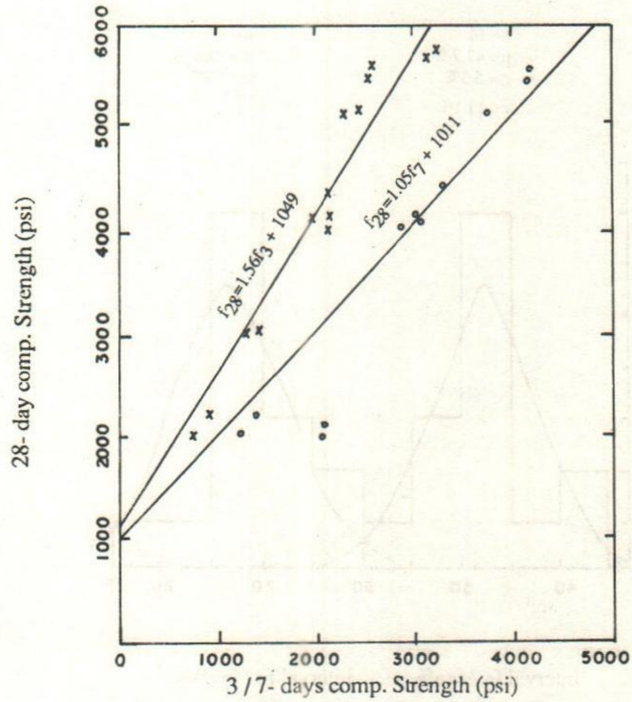


**Fig. 4 : 28- day compressive Strength Vs. 3 / 7- days compressive Strength (psi)**

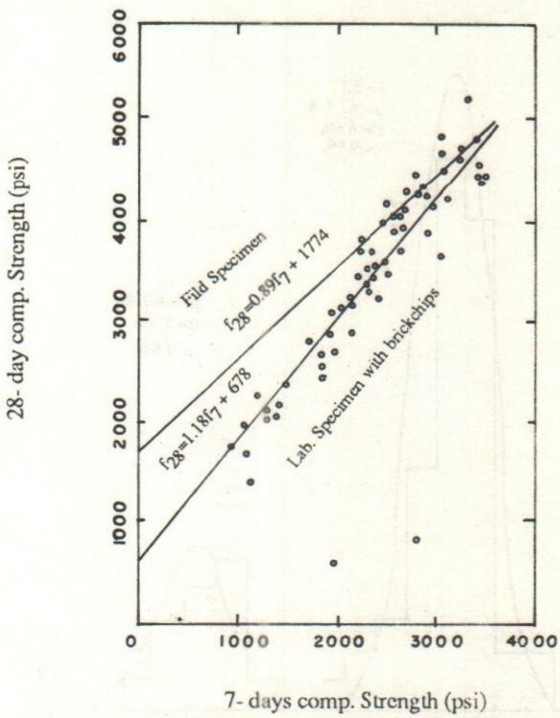


**Fig. 5 : 28- day compressive Strength Vs. 7- days compressive Strength (psi)**

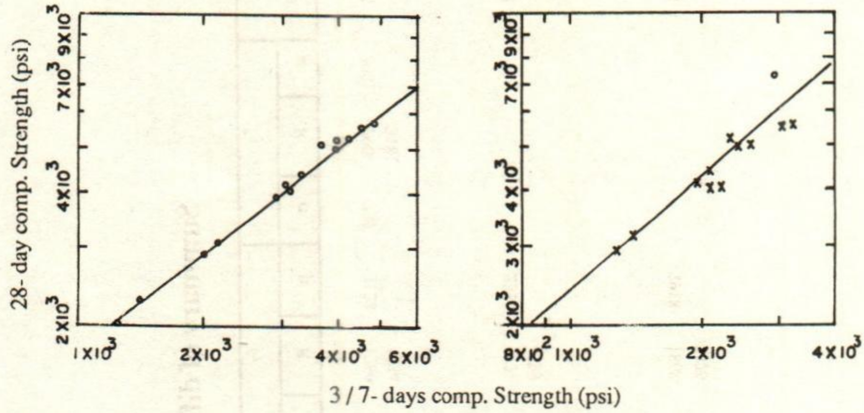


**Fig. 6 : 28-day comp. Strength Vs. 3/7-day comp. strength (Lab. specimen with shingels)**

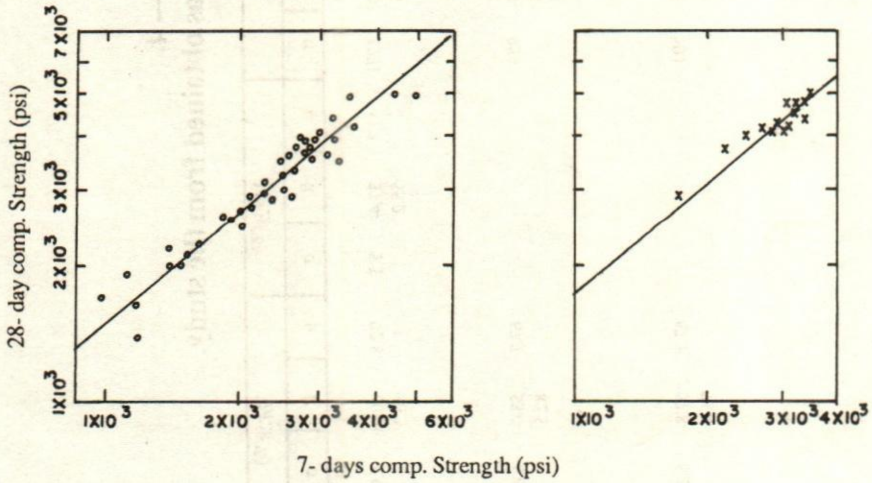


**Fig. 7 : 28- day comp. Strength Vs. 7- days comp. Strength (Lab. Specimen with brickchips)**

**Fig. 8 : 28- day comp. Strength Vs. 7- days comp. Strength (Field Specimen with shingles)**

# Table— 4.
## Summary of different values as obtained from the study.

| Type of 6"x12" Con. Cylinder | n | $f_3$ | | | $f_7$ | | | $f_{28}$ | | | $f_3/f_{28}$(%) | | | $f_7/f_{28}$(%) | | | Correlation | r | S y. x |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | μ | R | σ | μ | R | σ | μ | R | σ | μ | R | σ | μ | R | σ | | | |
| Lab. specime with shingles | 14 | 2081 | 790-3181 | 739 | 3128 | 1260-4634 | 1108 | 4295 | 2062-5765 | 1271 | 47.7 | 37.4-55.2 | 5.3 | 72.8 | 60.9-80.3 | 5.9 | $f_{28}=1.56f_3+1049$<br>$f_{28}=7.07f_3^{0.83}$ | 0.91 | 537 |
| | | | | | | | | | | | | | | | | | $f_{28}=1.05f_7+1011$<br>$f_{28}=5.61f_7^{0.827}$ | 0.92 | 498 |
| Laboratory Specimen with good quality Type-1 Porfiland cement, sand and brickships. | 52 | | | | 2331 | 972-4599 | 750 | 3443 | 1398-5395 | 953 | | | | 67.7 | 53.0-82.5 | 6.66 | $f_{28}=1.186f_7+678$<br>$f_{28}=4.48f_7^{0.857}$ | 0.93 | 350 |
| Field specimen with good quality Tye-I Portland cement, sand and shingles. | 14 | | | | 2918 | 1868-3476 | 472 | 4308 | 2993-4843 | 503 | | | | 67.7 | 57.8-76.9 | 5.3 | $f_{28}=0.89f_7+1774$<br>$f_{28}=4.77f_7^{0.855}$ | 0.83 | 280 |

**Table. 5**

| Values as given in or suggested by | $f_3/f_{28}$ (%) | $f_7/f_{28}$ (%) | linear relation | exponential |
|---|---|---|---|---|
| Troxelland Devis | 38-40 | 62 | - | - |
| USBR | 40 | 62 | - | - |
| Shetty | 40-56 | 68-80 | - | - |
| Used in Germany | | | $f_{28} = 1.4f_7 + 150$<br>$f_{28} = 1.7f_7 + 850$<br>$k_2 = 3$ to 6 | $f_{28} = k_2 (f_7) K_1$<br>$k_1 = 0.3$ to 0.8 |

Comparative statement of the values obtained in the study to that suggessed by different authors

appear that all the co-efficient of correlation obtained are very close to unity and almost 95% of the values analysed are expected to appear to fall within two standard errors of estimates and by trying the correlations obtained from the study with the respective maximum, minimum and average 3 & 7 days strength corresponding to those of 28 days in all the three sets of linear and exponential relations, the estimated 28 days strength are found to differ by about o to 18% only of the actual strength obtained. The correlations obtained can, therefore, be fairly used both in the laboratory and in the field.

It is, however, suggested that to be more precise and to remain in safer side in estimating the 28-day strength from that of 3 & 7 days, both linear and exponential relations may be tried and the more conservative value may be used as estimated 28-day strength as a guide-line for determining the future course of action. The study may be continued further with various types of materials and by keeping the temperature of mixing. casting and curing constant as and when better laboratory facilities for the same are developed.

**SYMBOLES**

$C$ = Coefficient of variation
$f_3$ = 3-day compressive strength in psi
$f_7$ = 7-day " "
$f_{28}$ = 28-day " "
$k_1$ = Constant
$k_2$ = Constant
$n$ = number of specimen
$r$ = coefficient of correlation
$R$ = range of values in psi
$\mu$ = average value in psi
$\delta$ = standard deviation
$S_Y$ x = Standard errpr in psi

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# SEDIMENT CHARACTERISTICS OF SOME RIVERS OF BANGLADESH

BY

**A. M. ZAHURAL ISLAM* & MD. NURUL HAQUE****

**ABSTRACT**

*Bangladesh is a flat deltaic riverine country having a unique system of rivers, tributeries and distributeries which play an important role in her overall economic activities.*

*The rivers are being silted up each year, thereby causing problems for irrigation, navigation, flood inundation, etc. Flood has become almost a regular phenomenon in Bangladesh causing serious damage to standing crops and properties, Extensive and proper studies of sediment can help in a long term solution of these sediment problems.*

*The paper deals with the mechanism of sediment transportation, classification of sediments into bed material load and wash load, and into suspended load, bed load and saltation load, and the samplers usually used for collection of different types of sediment samples. It states that in the laboratory, suspended sediment samples are analysed to determine their sediment concentration, and suspended sediments, bed load and bed materials are analysed to determine their grain size distribution and the results are presented in the form of summation curves.*

*To get a reasonable assessment of sediment in a stream, a number of formulae has been proposed by different authors, the paper deals with the relative performances of those formulae. Finally, the paper presents some of the sediment parameters of the Brahmaputra, Gumti, Surma and Gorai rivers. Graphs presented in the paper may be used as a tool evaluating one variable when the other is known.*

## INTRODUCTION

Bangladesh is a flat deltaic reverine country formed by the sediments carried by three international river systems, namely, the Ganges-Padma, the Brahmaputra-Jamuna and the Meghna. It has a unique system of rivers, tributeries and distributeries which play an important role in the development of agriculture, industries, communications and overall economy of the country.

The river systems are being silted up each year, thereby causing problems for irrigation, navigation, communication, flood inundation and maintenance of port & harbour. Flood has become almost a regular phenomenon in Bangladesh causing serious damage to standing crops and properties, and taking a huge toll of cattleheads and other livestocks. Bangladesh Inland Water Transport Authority (BIWTA) is spending a large sum of money for dredging river beds to maintain their navigability and Chittagong and Chalna port Authorities to maintain accessibility to their ports & harbours. Bangladesh Water Development Board (BWDB) is also facing much problems in keeping the intake channel of G.K. Project from being silted up. Extensive and proper studies of sediment can help in a long term solution of these sediment problems. Sediment studies are also essential in assessing

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the life-span of storage reservoir as well as in the design of hydraulic structures and other problems of siltation.

Let us consider the flow of water over non-cohesive sandy bed (1) when the flow passes over an individual particle, the streamlines are deflected around the particles as a result of which several forces are exerted on individual particles. These are known as lift, pressure drag and skin friction drag. In response to these forces, the particles roll or slide over neighbouring particles and the motion of bed particles commence at some favourable values of them.

The most important factor governing the sediment transport is shear stress which generates turbulent motion in the water and thus causes upward transportation of sediment. In terms of diffusion equation based on Fick's law, the vertical sediment distribution can be expressed as :

$$\frac{T_V}{\rho} = CW - K\frac{dc}{dy} \text{ .......... (1)}$$

For uniform flow at equilibrium condition $T_V = O$. Then eq. 1 reduces to :

$$C = C_O e^{\frac{W}{K}.Y} \text{ .......... (2)}$$

At any time a good number of bed particles may jump but eventually some of them return to be again but by that time other particles go into suspension and the process goes on continually. As a result, depending on the magnitude of turbulance and the shape and size of bed particles, some of them remain in suspension and move along with the flowing water. It may be pointed out that at one stage a particle may move as suspended material but in the next instant it may move as saltation or bed load particle.

## SEDIMENT AND ITS CLASSIFICATION

According to source, sediments can be divided into two groups viz. bed material load and wash load. But according to mode of transportation, the sediment can be classified into three categories, viz, bed load , saltation load and suspended load. The above classifications are shown schematically in Fig. 1.

In the light of above discussions, the different types of loads are described below :

Bed load is defined as sediment particles which move by sliding or rolling over and near the bed, generally in the propagating bed forms such as ripples and dunes.

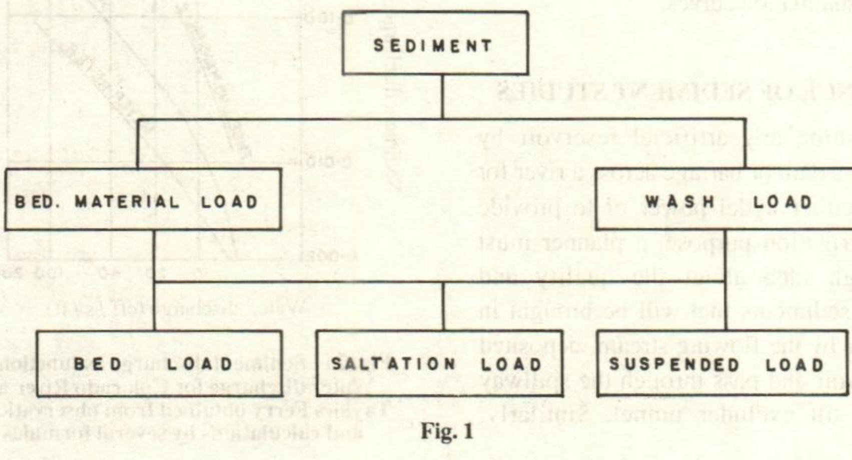


Fig. 1

Saltation load is defined as sediment particles which make their journey by hopping and jumping as a result of drag force. These particles very frequently move as bed load and sometimes as suspended load.

Suspended load consists of sediment particles held in suspension by balancing their gravitational force with the upward forces due to the turbulence of the fluid.

## COLLECTION AND ANALYSIS OF SEDIMENT SAMPLES

Depending on the mode of transportation, instruments used for collecting different types of sediment samples are also different. In the advanced countries many improved instruments and sophisticated methods are used for the collection and measurement of different types of sediments. In Bangladesh bottle sampler. Binckley sampler, point integrated sampler and depth integrated sampler are used for the collection of suspended sediments samples, bed load transport meter of Arnhem is used for bed load sediments and BM 54. BM 60 and other grab-type samplers are used for bed material samples.

In the laboratory, the suspended sediment samples are analysed to determine their sediment concentration and the suspended sediments, bed load sediments and bed materials are analysed to determine their grain size distributions. The results are presented in the form of summation curves.

## IMPORTANCE OF SEDIMENT STUDIES

For planning any artificial reservoir by constructing a dam or barrage across a river for generating either hydel power or to provide water for irrigation purpose, a planner must have enough idea about the quality and quantity of sediments that will be brought in the reservoir by the flowing stream, deposited in the reservoir and pass through the spillway by-pass or silt excluder tunnel. Similarly, sound knowledge about the sediments are also needed in the design of any hydraulic structure. Grain size distribution of bed material is needed in the design of stable canal as will be evident from the Lacey's (3) regime equation for slope as :

$$S = \frac{f^{5/3}}{178.8} Q^{1/6} \quad \text{.............. (3)}$$

or, for mean width, W, as :

$$W = \sqrt{b/s}\ Q^{1/2} \quad \text{............. (4)}$$

### Sediment Transport Formulae

There are quite a large number of formulae for assessing the total transported load of a stream. Most of the formulae were derived from laboratory studies as functions of bed load. Some of the formulae relate unit sediment discharge as function of a selected diameter of bed material distribution while others relate them as junction of bed shear stress. ASCE Task Committee studied a good number of such formulae and compared the values of sediment load botained from these relations with those obtained by field measurements for Niobrara and Colorado rivers Fig.2a, 2b.

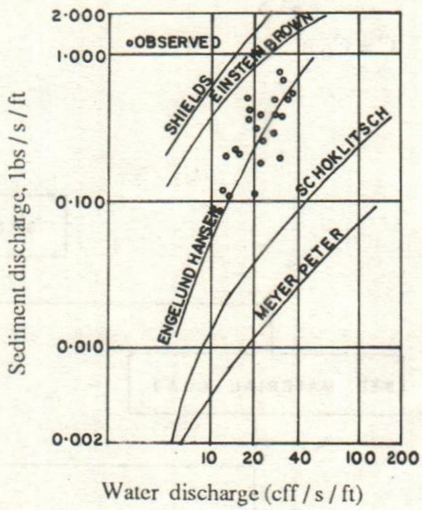


**Fig. 2a : Sediment discharge as function of Water discharge for Colorado River at Taylors Ferry obtained from observations and calculations by several formulas**

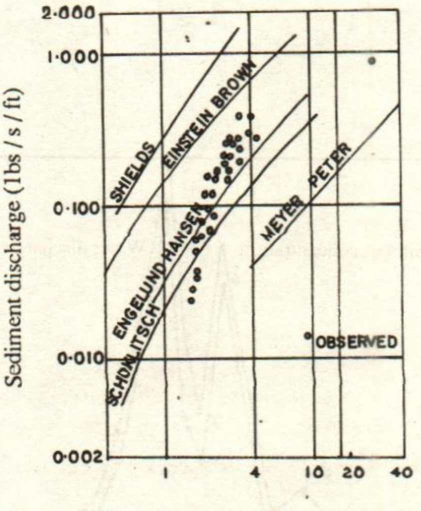


**Fig. 2b : Sediment discharge as function of Water discharge for Niobrara River near Cody, NEB obtained from observations and calculations by several formulas**

Some of these formulae where diameters of bed sediments come in are cited below for the convenience of the readers.

**Engelund-Hansen formula :**

$$g_s = 0.05\, \Upsilon_s\, V^2 \left[ \frac{d_{50}}{g(\gamma_s/\gamma - 1)} \right]^{1/2} \left[ \frac{\tau_o}{(\gamma_s - \gamma d_{50})} \right] \text{............ (5)}$$

**Meyer-Peter formula :**

$$g_s^{2/3} = 39.25 g^{2/3}\ S\text{-}9.95\ d_{50} \text{ ...(6)}$$

**Schoklitsch formula :**

$$g_s = \sum pi\ 25.3/\sqrt{dsi}\ S^{3/2}\ (q\text{-}q_{ci}) \text{......(7)}$$

**Shields formula :**

$$g_s = 10q\ S\ \frac{(\tau o\text{-}\tau c)}{(\gamma s/\gamma\text{-}1)^2} d_{50} \text{ ........(8)}$$

**Eintein's formula (5)**

$$g_s = 11.6 u_*\ Ca.a \left[ 2.3 \log_{10} \left(\frac{\gamma}{Rs}\right) I_1 + I_2 \right] \text{..(9)}$$

## PRESENTATION OF TEST RESULTS

Characteristics of four rivers, namely, the Brahmaputra at Bahadurabad, Surma at Sylhet, Gumti at Comilla and Gorai at Railway bridge in respect of their minimum and maximum values of water level, water discharge, sediment discharge, sediment concentration, flow velocity, bed materials and suspended materials have been presented in Table- 1 along with the corresponding values of Niobrara and Colorado rivers, as reported by the ASCE Task Committee for comparison. Annual variations of sediment concentration and water discharge of the Brahmaputre at Bahadurabad, Gumti at Comilla and Bogkhali at Ramu have been presented graphically in Fig. 3a, 3b, & 3c. Variations of water discharge against sediment discharge of the Brahmaputra at Bahadurabad and Gorai at Railway bridge, and those of water discharge with water level of the Brahmaputra at Bahadurabad and Gumti at Comilla have been presented graphically in Fig. 4 and Fig. 5 respectively.

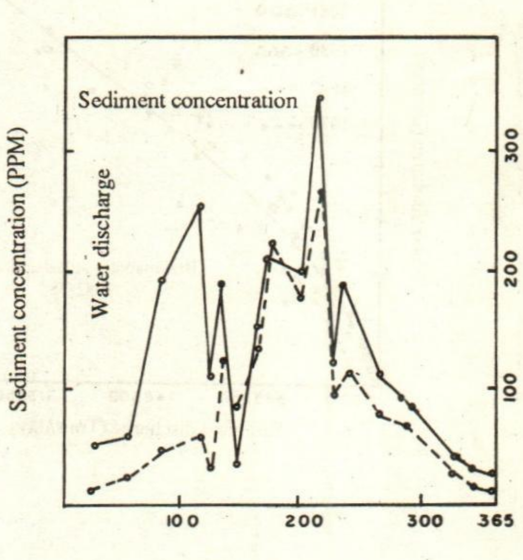


**Fig. 3a Gumti at Comilla during the year 1982**

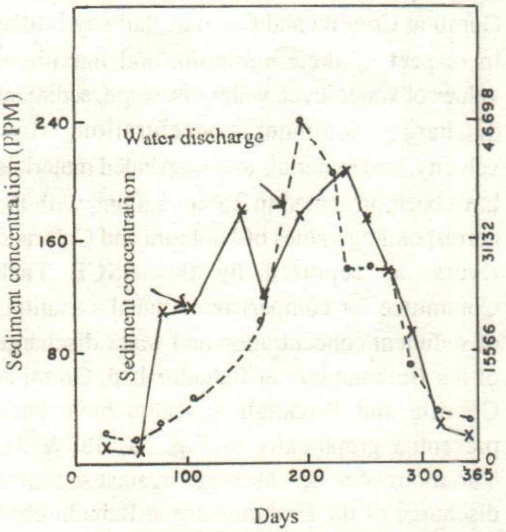


**Fig. 3b : Brahmaputra at Bahadurabad during the year 1982**

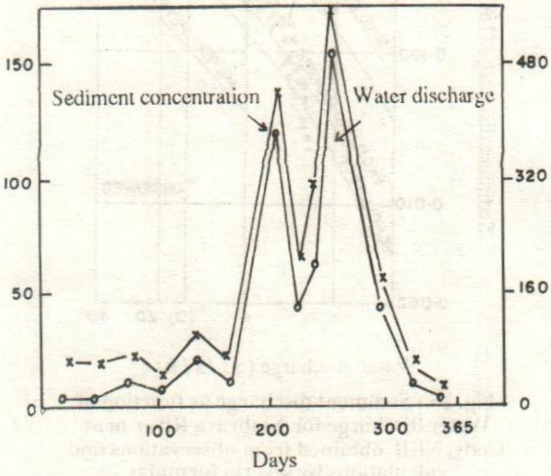


**Fig. 3c :Bogkhli at Ramu during the year 1982**

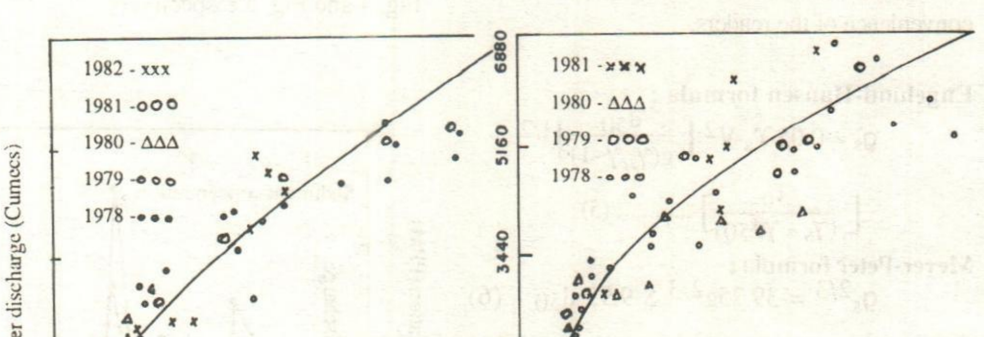


**Fig. 4**

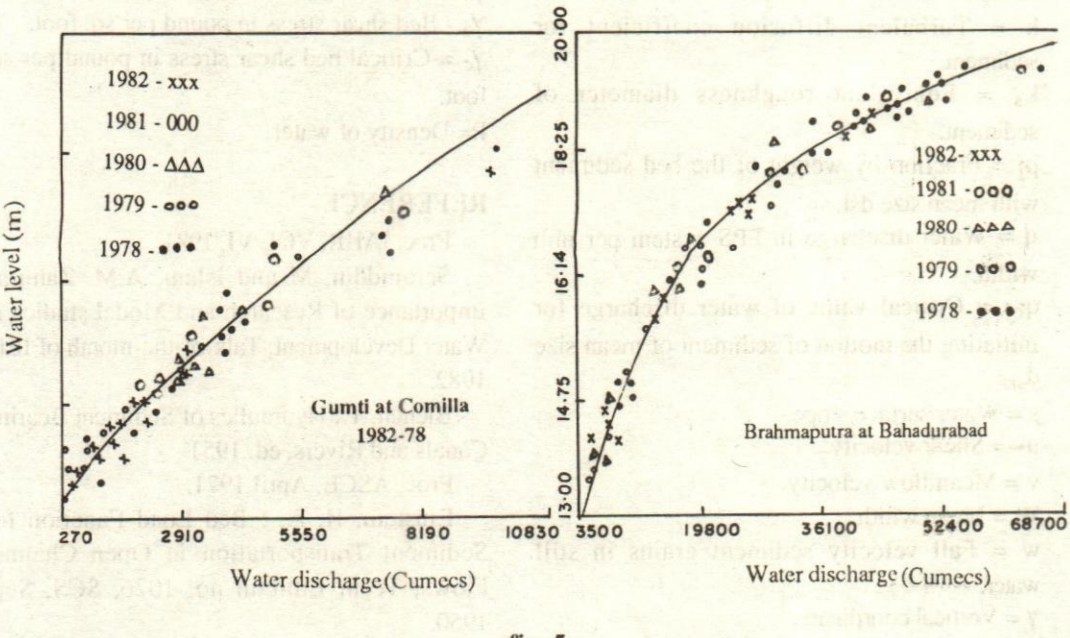


fig. 5

## DISCUSSIONS :

The results presented in Table- 1 will give an indication about the range of variations of the concerned parameters of the rivers where as the graphs provided in Figs. 3, 4& 5 may be used as tools for computing one variable when the other is known.

## CONCLUSION

A design engineer at the time of designing a hydraulic structure like barrage, drainage channel, irrigation canal, reservoir, flushing sluice, closure etc, has to consider to what extent his design will be affected by scouring or deposition of sediment. In this respect a sound knowledge among others about the water discharge, sediment concentration, sediment discharge, grain size distribution of bed material, bed load material and suspended material is almost indispensable for proper design.

The data presented in the paper may help an engineer in the planing phase of ascheme and the formulae stated in the paper may help a design engineer in assessing the sediment load he is concerned.

**List of symbols**

a = Reference elevation above the bed.

$b = 2\sqrt{m}$, bed factor.

C = Sediment concentration (by wt.)

$C_a$ = Sediment concentration at elevation a above the bed.

$C_o$ = Sediment concentration at zero vertical depth.

$d_{50}$ = Medium size of the bed sediment in feet.

$d_{si}$ = Mean sediment size.

f = Lacey's silt factor.

g = Acceleration due to gravity.

$g_s$ = Sediment discharge in pound per see per foot of width.

$I_1$ & $I_2$ = Integral functions evaluated by Einstein.

k = Turbulent diffusion coefficient for sediment.

$k_s$ = Equivalent roughness diameter of sediment.

$p_i$ = Fraction by weight of the bed sediment with mean size dsi.

q = Water discharge in FPS system per unit width.

$q_{ci}$ = Critical value of water discharge for initiating the motion of sediment of mean size $d_{si}$.

s = Water surface slope.

$u_*$ = Shear velocity.

v = Mean flow velocity.

W = Mean width.

w = Fall velocity sediment grains in still water.

$\gamma$ = Vertical coordinate.

$\gamma$ = sp. wt. of water in pound per cubic foot.

$\gamma$ = sp. wt. of the sediment grain.

$\gamma_s$ = Bed shear stress in pound per sq. foot.

$\gamma_o$ - Bed shear stress in pound per sq. foot.

$\gamma_c$ = Critical bed shear stress in pound per sq. foot.

P= Density of water.

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## Table- 1
## Comparison of parameters of several rivers for the year 1982

| Characteris tics. River | Water discharge in $m^3$/sec. | | Sediment discharge in tons/day | | Sediment conc. in ppm by wt. | | Water level in m (PWD) | | Flow velocity in m/sec. | | Ranges of material in mm | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | max. | min. | max | min. | max. | min. | max. | man | max. | min. | bed. | suspended |
| 1. Gumti at Comilla | 275 | 142 | 13519 | 73 | 569 | 60 | 10.72 | 7.98 | 1.03 | 0.41 | .90-.001 | .39-.001 |
| 2. Gorai at Gorai Rly. | 5440 | 630 | 159,995 | 929 | 278 | 29 | 12.66 | 3.68 | 1.65 | 1.02 | .40-.002 | .40-.001 |
| 3. Surma at Sylhet | 1292 | 5 | 44,651 | 12 | 400 | 26 | 33.61 | 10.66 | 1.11 | 0.14 | .84-.001 | - |
| 4. Brahma-Putra at Bahadurabad. | 46,700 | 3515 | 9,39,645 | 3210 | 233 | 10 | 18.77 | 13.06 | 2.70 | 1.00 | .27.011 | .07-.009 |
| 5. Colorado at Taylor's Ferry | 312 | 105 | - | - | - | - | - | - | 1.80 | 0.70 | - | - |
| 6. Niobrara near Cody | 20 | 6 | - | - | - | - | - | - | 1.37 | 0.54 | - | - |

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Maunscripts should be type written in triplicate including all tracing, printings ans photographs.

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