

# Geotechnical problems of bridge construction in Bangladesh

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## Abstract

The paper deals with various foundations used in Bangladesh for river crossing bridge structures. Some of the problems of construction and design of these foundations are highlighted. Because of the fact that most alluvial deposits of Bangladesh contain significant percentage mica, their effect need to be assessed in the interpretation of foundation design and slope stability calculations. The existing correlations using SPT values should be verified for these soils and there is a need for research in this area. For three large bridges in Bangladesh Osterberg (O-cell) cell tests have been performed. It has been observed that use of base grouting and skin grouting can significantly increase pile load carrying capacity.

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## 1. Introduction

Bangladesh is a low-lying country crisscrossed by numerous rivers. Communication network has been a great challenge for road and rail-line construction, as most road or rail-line links require building of numerous river crossings. Three large rivers: the Padma, the Jamuna and the Meghna divide the country. Most of these rivers have braided characteristics that make the banks unstable and variable soil condition exist across the crossings. Geotechnical conditions for foundation construction for bridges has been challenging for many reasons. Distribution of soils across crossings is complex and are usually heterogeneous both in vertical and horizontal direction. Soils consist of wide varieties of material ranging from poorly graded sand to silt and clay. In general there is a predominance of silt-sized materials and most often sandy soils contain significant percentage of mica. The presence of mica itself provides some unique characteristics to these soils that have been little studied in geotechnical literature.

Most of the older bridges built in this country are founded on well or caisson foundations. Because these well foundations were open caissons, it did not require any

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heavy or specialized equipment to construct them except for equipment for grubbing soil from within the well. But there have been frequent problems of undesirable sinking or difficulty in sinking of these caissons that delayed the construction time. Some case studies of such problems are described in this paper. With the development of bored pile construction in this country, the current tendency is to build bridge piers founded on large diameter bored piles. Driven piles are seldom used for bridges in Bangladesh except for very small bridges where scouring is not significant and driving is not a problem. The only major bridge built in Bangladesh that is founded on large diameter tubular steel piles is the Jamuna Bridge. Foundation for this bridge lies on 2.5 m and 3.2m diameter battered piles of about 80m long driven in medium dense granular micaceous sand. Obviously such driving required very heavy driving equipment that very few contractors own globally.

This paper reviews some bridge foundation design and construction practice followed in Bangladesh and reflects some case studies of the type of geotechnical problems that needs to be overcome to develop towards advancement of bridge construction in Bangladesh.

## **2. Caisson foundations**

Caisson foundation have been very popular in Bangladesh for a long time because of the ease with which these could be built without use of any heavy or sophisticated machinery or equipment. Some of the large bridges have been founded on this type of foundation.

Before starting construction of a caisson within riverbed, the local practice is to build a sand island. Since the sand islands are temporary structures these are usually built of sheet pile enclosure filled with sand. In deep waters where scour is a problem often stability of the sand island requires critical examination. In shallow waters, sometimes wooden piles (shal-bolli) are driven closely with some bamboo mattress inside that retains sand for Sand Island. There have been instances where Sand Island has been washed out, tilted or displaced the caisson built inside it, which, necessitated either change in alignment or readjustment of the bridge spans. Often there are considerable problems with sinking of caissons by overcoming skin-friction. Sinking in conditions where skin friction is considerable lateral jetting becomes essential along with use of drilling fluid (usually bentonite). This requires thoughtful arrangement of internal piping and mud circulation, which is seldom followed by our contractors resulting in delay in construction, uneven sinking and even collapse of caisson.

### *2.1 Problem with construction of 2<sup>nd</sup> Buriganga bridge*

The 2<sup>nd</sup> Buriganga Bridge over river Buriganga connects heart of Dhaka city at Nayabazar with Jinjira on the other side. The total length of the bridge within the limit of contract is 1479 m while the span within the river portion is 304 m and founded on five caisson foundations. The depth of these caisson foundations varied between 30.5 m and 32.5 m. Each caisson is oval shaped with external dimensions of 6 m by 13.4 m. The construction of the bridge commenced on 29<sup>th</sup> August 1994.

During middle of September 1996, difficulties developed during construction of the caisson foundation for pier no. 17. After the caisson was sunk to a depth of about 16 m. there was difficulty in further sinking although grubbing and soil removal from inside of the caisson was in progress. Some 4 to 4.5m of soil were removed from inside the

caisson but the caisson did not sink by its own weight. There were no inbuilt outer jetting arrangements within the caisson walls. At this stage the contractor used a 1.5-inch GI pipe 18 m long to inject water close to the caisson wall and this jetting continued at one-foot interval. When half the diameter of the caisson was covered, water suddenly oozed out from the caisson and the caisson sank to a depth of 6.7 m and the level of soil inside the caisson raised to a depth of about 7 m above excavated ground inside the well. The top of the caisson went under river water level. Fig. 1 shows the condition of the caisson before and after sinking. It can be seen from the figure that before sinking the caisson had a grip length of about 16 m (52 feet). Fortunately because of this grip length there was no tilting of the caisson and the sinking could be continued to desired depth despite the fact that top of the caisson went below river water level. Two important lessons were learnt from Buriganga. Adequate internal jetting arrangements should be provided for caissons where significant skin friction is likely to develop. If external jetting is to be used it should be done symmetrically so that uniform sinking takes place. One need not go for excessive removal of soil from within the open caisson without releasing skin frictional resistance in uniform manner.

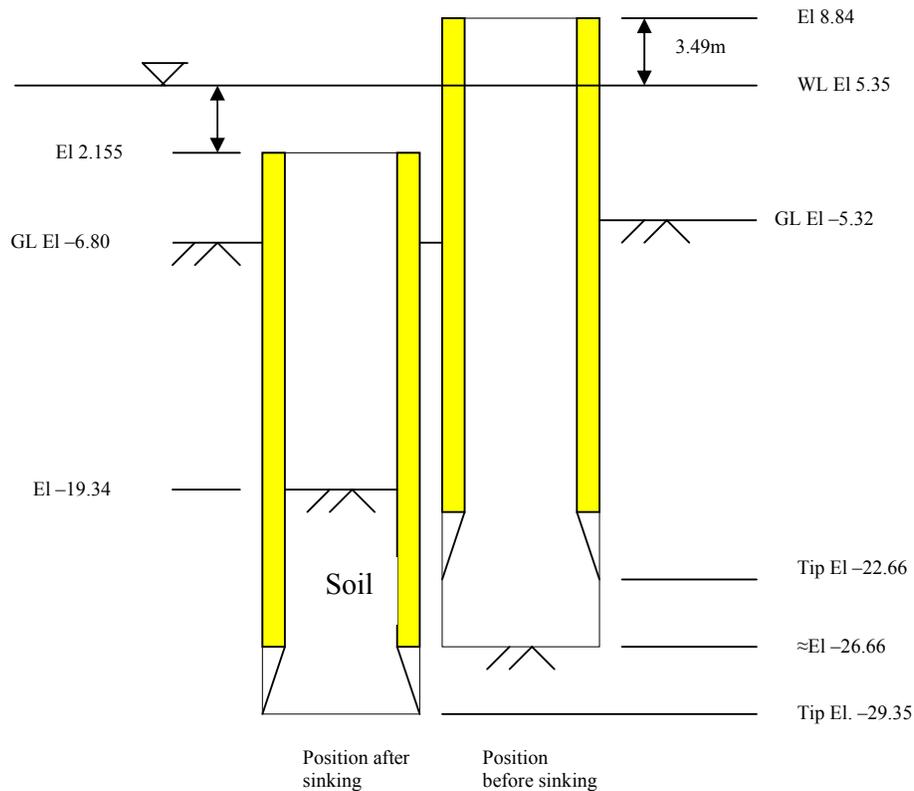


Fig. 1. Position of caisson on Pier 17 of the Buriganga bridge before and after sudden sinking on September 1996.

## 2.2 Failure of caisson on Kalidash-Pahalia Khal Bridge on Feni-by-pass

It may be recalled that in 1976 during installation of a caisson for the bridge on Kalidash Pahalia Khal on Feni-by-pass road, due to heavy inrush of water of the flashy river the

caisson tilted and failed. The reason for failure was investigated and found to be due to inadequate depth of embedment and resulting scour at the time of high river flow (Hossain et al, 1983<sup>1</sup>). Therefore due importance should be given to the grip length of the caisson at the time of construction.

### 3. Bored pile foundation

Although this type of foundation induces more turbulence and scour at the riverbed level it is becoming popular due to development of technical capability to built very large diameter piles, faster construction and better construction techniques and also due to elimination of the need to construct sand island. Large diameter bored piles are gradually replacing caisson foundation in bridge construction in Bangladesh.

The Japan-Bangladesh Friendship Bridge over river Meghna is built on piers founded on bored piles. The construction of foundation at riverbed level required construction of a watertight cofferdam built with steel pipe piles (diameter: 1.016 m, length: 28.5 m) with vertical interlocking system. The cofferdam had to be braced with heavy steel pipes to resist external water pressure when inside is drained out. After installation of bored piles within the cofferdam, the inside had to be dewatered for excavation and preparation of bed for pile cap and pier. Fig. 2 shows pile head treatment for construction of pier within the cofferdam. Cast-in-situ concrete piles of 1.5m in diameter were constructed by using reverse circulation drilling method. Special measuring system was adopted for the vertical accuracy of the boreholes. The lengths of these piles are variable ranging from 40.0 m to 58.0 m depending on the level of the bearing strata for the piles.



Fig. 2. Pile head treatment for construction pier foundation inside the cofferdam.  
(Courtesy: Roads and Highway Department, Bangladesh)

Most of the bored pile foundations used for smaller bridges use Sand Island instead of the type of cofferdam used for the Meghna Bridge. Because the pile tops are placed at the surface of the sand island, which is above river water level, bottom of pile caps are above riverbed, creating turbulence and excessive scour at riverbed level. Some times

<sup>1</sup> Hossain, A.S.M.M; Salahuddin, M and Hasan, A (1983): A case study on Kalidas Khal Bridge caisson failure. Unpublished undergraduate thesis, Department of Civil Engineering, BUET.

piles develop defect due to bad construction practice that relates to borehole formation, borehole cleaning and underwater or tremie concreting. Fig. 3 shows an example of bad concreting that developed within the piles just below pile cap for one of the pier of Dhaleshwari-1 Bridge. From the photograph it can be observed that there is discontinuity in the top casing where the concreting was affected.



Fig. 3. Photograph shows defective concrete work in the piles for Dhaleshwari-I Bridge.

One of the major problems faced by the engineers in Bangladesh is the estimation of pile capacity for bored piles installed in riverbed for bridge structures by using static formulas. Since most of the riverbed formations are of granular deposits no undisturbed samples are usually collected. Design is normally based on field test such as standard penetration test N-values. The N values are correlated to angle of internal friction ( $\phi$ ) values, that is normally required in static analysis. For granular soils it usual to use  $\phi$ -N value relations proposed by various authors such as Peck et al (1974)<sup>2</sup> and Kishida (1967)<sup>3</sup>. Here N is the Standard Penetration Test value corrected for field conditions and normalized for overburden pressure. Unfortunately, due to presence of large quantity of mica in the riverbed, there is uncertainty as to the applicability of these correlations for granular deposits of Bangladesh. Besides it is not possible to collect undisturbed granular deposits from riverbed and test these in the laboratory for shear strength and deformation parameters for field condition. Tests at BUET have indicated that sands containing mica can have significantly lower  $\phi$  values and lower density during sedimentation process.

### 3.1 Load test of piles

Because of the uncertainty with estimation of pile capacity from static formula, it is better to evaluate pile capacity from static load tests. Since large diameter piles are commonly used for bridge foundations it is a formidable task to perform field pile load test using normal pile load test practice, which uses load platform or anchor piles. Because of the huge loading platform that would be required for such test, it is not practicable. Besides normal load test procedure, followed for bored piles, furnish

<sup>2</sup> Peck, R.B.; Hanson, W.E; and Thornburn, T.H.(1974): Foundation Engineering; 2<sup>nd</sup> Edition; Wiley Eastern limited.

<sup>3</sup> Kishida, H.(1967): Ultimate bearing capacity of piles driven into loose sand; "Soils and Foundations", vol. 7, no.3:20-29.

insufficient information on skin friction and end bearing components separately. Such information is required in applying appropriate factor of safety in pile capacity at various deformation stages. For the large diameter bored piles in Bangladesh, much of the soil parameters for design related to estimation of skin frictional resistance and end bearing is estimated from 'Bi-directional Osterberg Cell Load Testing' or O-cell testing.

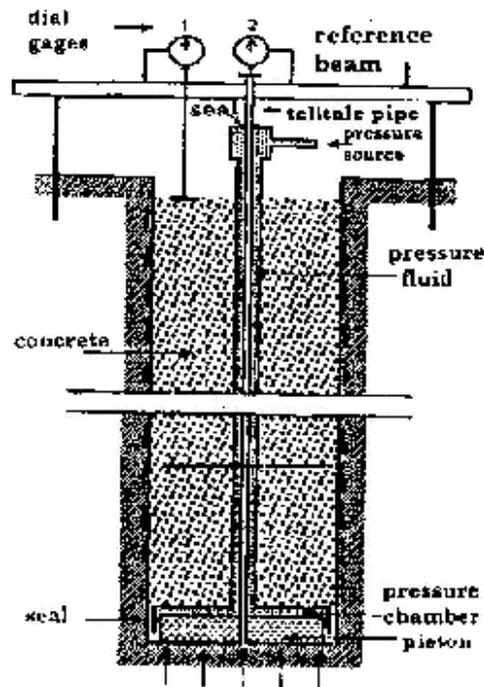


Fig. 4. Osterberg cell schematic

Invented and developed by Dr. J. Osterberg of Northwestern University, O-cell test for testing high capacity piles has changed the way foundation load tests are designed, performed and interpreted. The Osterberg Load Cell (also called O-cell) consists of a specially designed hydraulic jack capable of exerting very large loads at high internal pressures. Fig. 4 is a schematic diagram illustrating how the load cell works. A small amount of concrete is placed on the bottom of a bored pile hole after which the O-cell is lowered into the hole, which is then filled with concrete. A pipe welded to the top of the center of the cell extending to above ground surface acts as a conduit for applying fluid pressure to the previously calibrated cell. Inside the pipe is a smaller pipe connected to the bottom with an open end. It extends to the surface and emerges from the large pipe through an O-ring seal. This pipe acts as a telltale to measure the downward movement of the bottom of the cell as load is applied. The fluid for applying pressure can be oil or water. The liquid most often used is water with a small amount of miscible oil added to keep pump equipment from rusting. After the concrete has reached its desired strength, the cell is pressurized internally creating an upward force on the bottom of the shaft and an equal but opposite force in end bearing. As the pressure increases, the telltale moves downwards as the load in the end bearing increases and the shaft moves upward as the side shear on the shaft is mobilized. It should be noted that at all times the total side shear resistance above the O-cell is equal to the end bearing. Because of this no reaction

load or hold down piles with a frame is needed as in the conventional test where the load is applied downward on the pile head.

The downward movement is measured by dial gage 2, and the upward movement of the top of the concrete is measured by dial gage 1. Not shown on the figure is a pipe extending from top of the of the load cell to above the surface in which telltale rod that measures the upward movement of the top of the cell. Thus the difference between the measurement of this rod and dial gage 1 gives the compression of the concrete. From data obtained as the load is increased, the load-upward movement curves and the load-downward movement curves can be plotted. After the test is completed, the area below the bottom of the O-cell and the cell chamber can be grouted if the pile is to be used as a working pile.

For the largest capacity size O-cell (three feet diameter) the maximum force, which can be applied, is 3,000 tons up and 3,000 tons down. Details of testing procedure and interpretation of results can be referred to Osterberg (1999)<sup>4</sup>.

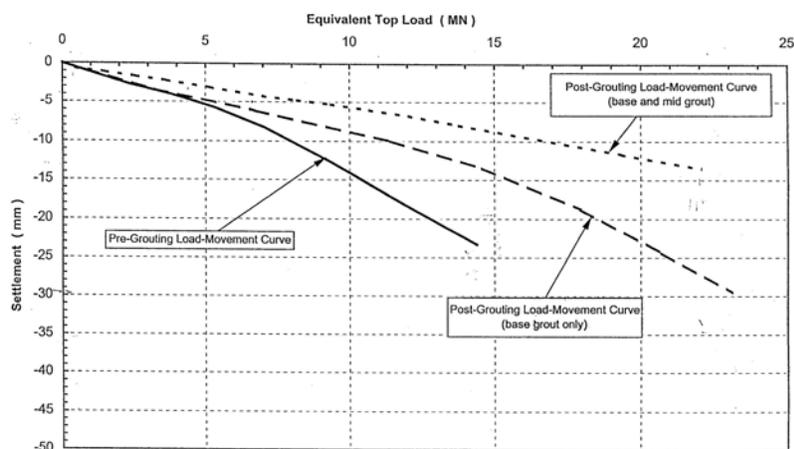


Fig. 5. Comparison of pre and post grouting capacity of test piles for Paksey Bridge as obtained by O-cell tests.

O-cell tests have been very useful in interpretation of skin-friction and end-bearing characteristics of large diameter bored piles used in three major bridges in Bangladesh such as the Bhairab Bridge, the Paksey Bridge and the Rupsa Bridge. It may be recalled that pile capacity in these bridges had to be increased by skin and base grouting as the un-grouted piles did not provide desired capacity required for design. Fig. 5 shows load-settlement relations for test piles used in Paksey bridge using O-cell test for condition of (1) no grouting, (2) base grouting only and (3) combination of skin and base grouting. The diameter for these piles was 1.6 m and the embedded length was about 70m. Using O-cell, load tests could be performed to a very high capacity. It can be seen from the test results that significant improvement in pile capacity in bored piles is possible in the granular deposits of Bangladesh. A specialist pile load-test contractor 'Load Test Inc.' performed all these tests.

<sup>4</sup> J.O. Osterberg (1999): What has been learned about drilled shafts from Osterberg test. Paper presented at the Deep Foundation Institute Annual Meeting- October 1999.

#### 4. Large diameter pipe or tubular piles

Small displacement tubular or pipe piles are not much in use in bridge construction in Bangladesh because of lack of driving equipment required for such piles. But these piles have the advantage that they can be easily spliced and welded to achieve significant depth of penetration sometimes required due to scour. Also these piles can be driven in battered position to resist lateral loads. Development of offshore technology introduced heavier pile driving hammers that are capable to drive large diameter piles to considerable depth within a very short time. It is for such reason large diameter (2.5 m and 3.2 m) driven tubular piles were selected for use in the Bangabandhu Bridge (Jamuna Bridge) over river Jamuna.

Fig. 6 shows typical 2-pile arrangement for pier support of the Bangabandhu Bridge. The tubular structural steel piles were originally supposed to be left empty inside, but later, the inside had to be removed of soil and filled with concrete except for bottom 5 m. of the tube. Grouting pipes were installed within the filled concrete and when the concrete had hardened, pressure grouting was done to improve the base soil inside the tube against any possible loose condition. Tappin et al (1998)<sup>5</sup> has described the construction of the Jamuna Bridge in Bangladesh.

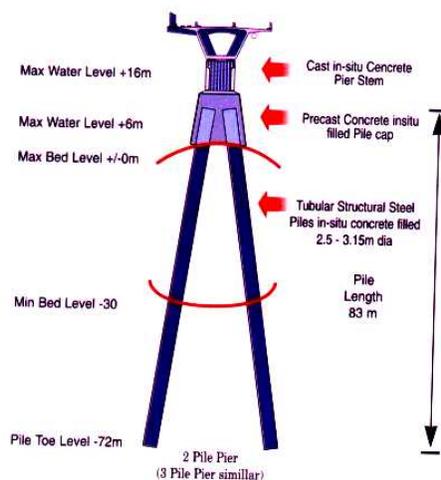


Fig. 6. Typical 2-pile arrangement for Jamuna bridge piers.

#### 5. River training works and guide bunds

Guide bunds are essential to confine the river flow within the bridge length. Guide bunds require bank protection from waves, bed scour and stability of the slope. In Bangladesh all the river courses are in loose alluvial soil and are prone to rapid scour, which can significantly change the cross-section and course of the river. Hydraulic and soil conditions determine the stability condition of the channel.

Scour depth around piers and guide bunds should be determined from estimation of general scour, constriction scour and local scour. Bridge Engineers' Hand Book

<sup>5</sup> Tappin, R.G.R, Duivendijk, J.V. & Haque, M. (1998): The design and construction of Jamuna Bridge, Bangladesh; Proc. Instn. Civ. Engng, 126, Nov., paper 11704:150-162.

published by the Roads and Highway Department is normally used for these computations.

Stability of riverbanks, which contains mainly sandy silts and mica in loose state, is a serious problem for stability analysis in Bangladesh. Lessons learned from the construction of the Jamuna Bridge are of great significance in designing guide bunds and bank protection works. In the Jamuna Bridge, important elements of the construction were the West and East Guide Bunds, designed to control the flow of the river. The guide bunds, which comprise heavily protected sand slopes, are 3.3 km long, with the northern tip 2.5 km north of the bridge site. To enable slope protection materials to be placed, a trench, varying in depth from 27 to 30 m below original ground level, was excavated below water by cutter-suction dredgers. The trench comprised of the permanent protected slope and a temporary unprotected slope. The temporary slope had to remain sufficiently stable during construction so as not to interfere with the formation of the permanent slope and to maintain a barrier to the river flow, so to ensure current free condition for placement of the protective mattresses.

The West Guide Bund was constructed at the site of a rapidly formed island. The materials forming the dredged slopes consisted of young, rapidly deposited sand sediments. During its construction, a number of slips occurred in both the permanent and temporary unprotected slopes. A plan of the location of interest where the slips had occurred in both the permanent and temporary slopes while it was being dredged at original design slope of 1 in 3.5 are shown in Fig. 7. These slips represented approximately 50 % of the constructed length. At this time, the temporary slope being dredged at 1 in 3 slope was affected almost 100% of the dredged length. After the incidence a modification into the original designed slope was made which is shown in Fig.8. After the modification in the design profile was made, only three minor slips, representing less than 5% of the constructed length, occurred in the permanent slope when it was being dredged at 1 in 6 slope. In contrast, slips continued apace in temporary slopes, which was now at 1 in 5; these slips were associated with dredging, with storm activity, and draw down of river level. It is important to note marked difference in performance of the two slopes, at 1 in 6 and 1 in 5. Hight and Leroueil (2003)<sup>6</sup> provided a detailed account of this difference in behavior, which is described below.

The slips had the characteristics of underwater flow slides. Bathymetric surveys showed that, in plan, the slips had a classic hourglass shape, comprising a bowl-shaped depression from which the materials have been removed and an alluvial fan, where the slipped materials collected. In cross-section, the post failure profile was stepped and had a slope between 1 in 10 and 1 in 20. The failures took several hours to develop and involved between 50,000 and 100, 000 m<sup>3</sup> of material.

The slopes were being dredged in micaceous sands. The mica comprised thin sand-sized plates, generally biotite. The quantity of mica, its distribution and its orientation varied. Grain counting indicated mica contents of 5-10%. SPT tests at the site suggested that relative density of these micaceous sands was between 40 and 60%, values that would not normally be associated with flow slides.

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<sup>6</sup> Hight, D.W., & Leroueil, S. (2003): Characterization of soils for engineering purposes. *Characterization and Engineering Properties of Natural Soils*; Tan et al (eds), Vol.1, A.A. Balkema Publishers: 255-360.

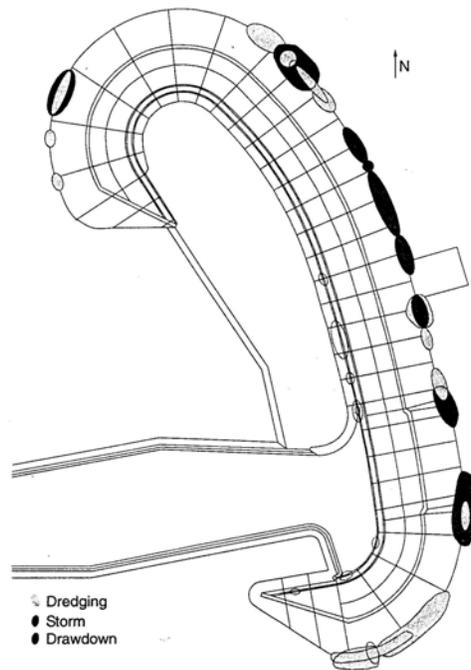


Fig.7. Plan on west guide bund to Jamuna bridge, showing locations of flow slides and trigger (Hight et al, 1999)

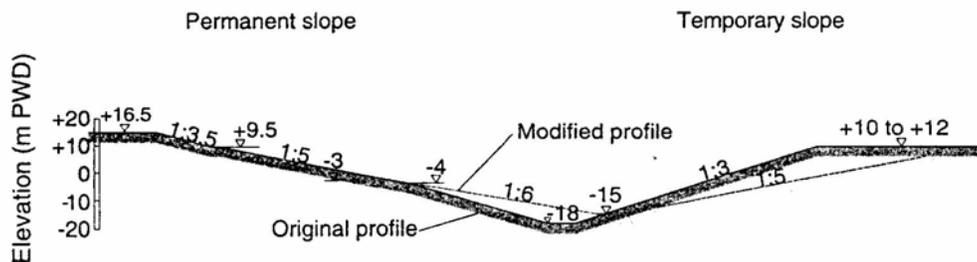


Fig. 8. Cross-section through west guide bund (Hight et al, 1999)

Leroueil and Hight (2002)<sup>7</sup> have described the effect of mica on the behavior of the sand. They have compared the undrained behavior in simple shear of a clean sand and the same sand with just 1% mica by weight added. The clean sand, although loose, is ductile; a tendency to dilate at large strains causing the effective stress path to climb up the failure line. In stark contrast, the sand with 1% mica is brittle and shows a potential to collapse. The addition of just 1% of mica by weight has suppressed almost completely the tendency to dilate. They have also shown results of undrained triaxial compression and extension tests on samples from one batch of natural materials, prepared by dry spooning at relative densities of 58% and 55%, respectively. In compression, the material is ductile and has an undrained strength in excess of 180 kPa. In undrained extension, the material is brittle and has a strength of only 10 kPa.

<sup>7</sup> Leroueil, S., & Hight, D.W. (2002): Mechanical behaviour and properties of natural soils and soft rocks Proc. Int. Workshop on Characterization and Engineering Properties of Natural Soils, Singapore.

The key points of their study may be summarized as follows:

- The presence of sand sized mica in a sand leads to an increase in void ratio, to extreme levels of undrained anisotropy and extreme sensitivity to fabric. Volume change characteristics are modified and a collapse potential may exist for certain loading directions and for certain quantities, distribution and orientations of the mica. In terms of brittleness and collapse potential, small quantities of mica, less than 2.5 % by weight, are probably critical.
- Micaceous sands appear to be particularly weak in extension loading and most vulnerable to collapse when under low stress. These conditions apply just beyond the toe of an underwater slope subject to unloading by dredging or scour.
- In evaluating flow potential, anisotropy and principal stress directions of potential perturbation must be taken into account.
- Current design approaches that rely on triaxial compression tests on reconstituted samples are not valid because they implicitly assume that steady state is isotropic and independent of stress path and initial fabric.

The case study emphasizes the importance of considering quantity, distribution and orientation of mica particles in a sand, its full stress path including possible rotation of principal stress direction, and the range of drainage conditions that may apply.

## **6. Conclusions**

Bridge design and construction in Bangladesh involves a very good understanding of the local soil conditions. At present due to lack of understanding and research on the geotechnical parameters to be used for river borne granular deposits containing mica application of existing parametric correlations remains questionable. The SPT- $\phi$  correlations suggested in most literatures need to be validated for Bangladesh soils.

Three types of deep foundations are now in use for bridge construction in riverbeds: caissons, cast-in-situ bored piles and tubular bored piles with or without concrete infill. Construction of most of these foundations requires Sand Island whose stability is critical to hydraulic flow and scour during construction. Some case studies presented in the paper demonstrate that quality control and monitoring during construction stage should be carefully performed to avoid development of unwanted situations like failure, delay in construction or modification of the design.

Use of large diameter bored piles for three major bridges in Bangladesh has revealed that considerable improvement in pile capacity can be achieved by skin and base grouting techniques. Use of O-cell test has been very useful in interpreting pile capacity and deriving skin frictional resistance and base resistance of large diameter piles.

Case study of guide bund failure for Jamuna bridge has revealed that slope stability analysis during dredging can be very critical which requires considerations for relative density of soil, quantity, distribution and orientation of mica particles in the soil, its full stress path including possible rotation of principal stress direction, and the range of drainage conditions that may apply

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