Geotechnical Engineering Journal of the SEAGS & AGSSEA Vol. 41 No.3 September 2010 ISSN 0046-5828

Enhancement of Pile Capacity by Shaft Grouting Technique in Rupsa Bridge Project

Ryuji Manai
B.Sc, PE(Japan), APEC Engineer, MCIArb, DipArb
Oriental Consultants Co., Ltd (E-mail:manair@oriconsul.com)
* Former Resident Engineer of the Rupsa Bridge Construction Project (Bridge)

ABSTRACT: This paper makes the presentation of the author’s experience of the pile construction for the Rupsa Bridge Construction Project in Bangladesh. The initial pile loading tests for cast-in-situ pile of diameter 2500mm and 75m long constructed in Rupsa River, one of the branch River of Ganges River, resulted in failure at the early stage of the loading test due to the dispersive behavior of the soil which likely have reduced the skin friction and end bearing capacity. In order to enhance the pile capacity, base grouting and shaft grouting technique was adopted. The result of application of this technique has achieved the pile capacity as high as 5 to 7 times that of plain piles which is far more than the previously reported 1.5 to 3 times in the other projects using the same shaft grouting technique. The paper describes the detailed know-how of the technique.

1. INTRODUCTUION

Roads and Highways Department (RHD), Ministry of Communications, the Government of the People’s Republic of Bangladesh with the financial assistance from Japan Bank for International Corporation (JBIC) had undertaken Rupsa Bridge Construction Project. Among 10 km long by-pass construction (Satkhira-Mongla), the pivot of the project was to construct the Main Bridge over the river Rupsa of 640m in length (5 nos. 100m middle span and 70m end spans) and Approach Bridge of 720m, total length of 1.36km.

This paper primarily covers the Base Grouting and Shaft Grouting techniques which were introduced for the first time in Bangladesh in order to enhance the cast-in-situ pile capacity for the Main Bridge of this project.

2. PILE DETAIL

The main pier foundation consists of 6 nos. 2500mm dia. cast-in-situ pile (tentative length of pile 75m) at each pier location. The each pile has a permanent steel casing from EL0 to -35m. Figure 1 show the pile layout and test piles at MP6.

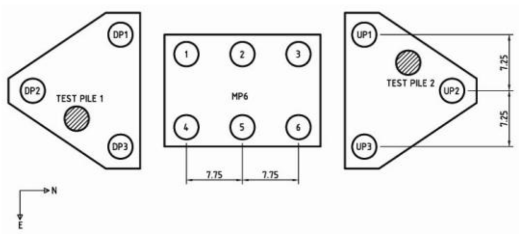


Figure 1: Pile Layout and Test Piles at MP6

Pile construction was carried out by reverse circulation drilling method using slurry replacement technique to stabilize the pile bore and tremie method for concreting. The piles were designed to a working load of 2250 ton of which major component of pile capacity is shared by skin friction and remaining by end bearing.

3. SUBSOIL CONDITION

The subsoil conditions are very sensitive character of poorly graded uniform fine silty sand / clay with low to medium SPT N values. Such soils are susceptible to loose strength due to their dispersive behavior caused during bored pile construction and that might influence the skin friction and end bearing thereby the pile capacity. Table 1 shows the typical subsoil conditions.

Table 1: Typical Subsoil Conditions

Depth (m)	Description	SPT-N Value
-6.8~-9.3	Silty Sand	20
-9.3~-19.8	Silt with Sand	5
-19.8~-34.8	Plastic Silt to Silty Clay	12
-34.8~-51.3	Silt to Silt with Sand	9
-51.3~-57.3	Silty Clay	15
-57.3~-87.3	Sand with Silt	39

4. PILE LOADING TESTS

Two test piles were constructed as shown in Figure 1 namely TP1 and TP2 and the static loading tests were carried out consecutively in May and July 2002. The results of both loading tests showed a significant plastic settlement at the early stage of the load increment.

Tables 2 and 3 show the settlement behaviour of the test piles respectively.

Table 2 : TP 1 Settlement Behaviour

Load (ton)	1400	2200	3300	3400	3400 (Final)
Settle (mm)	73	200	534	562	618

Table 3 : TP2 Settlement Behaviour

Load (ton)	550	1650	2200	2200	2200 (Final)
Settle (mm)	50	175	250	289	328

The test results were analyzed and it was concluded that the ultimate capacities of TP1 and TP2 were only 1650 ton and 1100 ton respectively. The corresponding mobilized skin friction was 3.5 ton / sq.m and 2.2 ton / sq.m.

5. CONSIDERED REASON OF PILE FAILURE

The subsequent soil investigation revealed that the subsoil conditions are very sensitive character of poorly graded uniform fine silty sand/clay (dia. 0.1~0.01mm). It was considered that soils are susceptible to loose strength due to their dispersive behavior caused during bored pile construction and that might have influenced the skin friction and end bearing thereby ultimately reduced the pile capacity.

6. COUNTER MEASURES

The results of the pile capacity obtained and its magnitude at failure are far from the expected design capacities of the main piles. After intensive research for the possible counter measures to enhance the pile capacity, the author obtained an invaluable information regarding shaft grouting technique from Babtie Asia (HK) and Bachy Soletanche (HK) and it was concluded that adopting the combined shaft and base grouting techniques were the best way forward to resolve the problematic piling situation. To assess the pile capacity and confirm the efficacy of the grouting before adopting in the permanent piles, it was also decided to conduct the 3rd loading test by Osterberg Cell method and instrumentation. This test enables us to know the skin friction resistance of soils at different depth with grouting effect.

7. HISTORY OF SHAFT GROUTING

The first published account of the use of shaft grouting, was by Gouvenot and Gabaix (1975), who presented their finding on the construction and loading of six shaft grouted 660mm dia. bored trial piles. The results of the testing indicated an increase in skin friction of 2.5 times that of plain piles. The results of shaft grouted piles in cohesive and non-cohesive soils presented by Stocker (1983), showed a permanent increase in skin friction of 1.5 to 3 times that of plain piles. A review of published work between 1975 and 1985 is presented by Bruce (1986), on pile construction and the benefits of post grouting. The paper highlights the benefits of shaft grouting other than enhanced skin friction capacities, such as cost and programme savings. In addition, the paper also highlighted that, where conventional piling techniques have been found to be faulty or inadequate, the use of grouting as remedial measure had gained widespread application.

More recently, the shaft grouting technique is being used more widely in developments such as in Egypt in 1994 for 1.5m dia piles, in South Africa 1998 for a 1.2m dia. test pile, in Jeddah 1998 for 1.0m dia. test piles. In South East Asia, the technique has been reported from Bangkok in 1992 and now it is being widely used in Kuala Lumpar, Indonesia, Taiwan and Hong Kong. With regard to the long term durability of the effect of the shaft grouting, Littlechild et al (1998) reported as a case in Bangkok that reloading over one year after the first load test, no loss of shaft resistance was recorded for two shaft grouted piles in alluvial sand and clay.

We have studied numberous reported cases and concluded that the following are the basic features of this technique.

- Use of Tube-a-Manchette pipe for grouting
- Grout strength on 28 days around 20~25MPa
- Grouting criterion is on quantity of grout (25~35 litter per sq.m of the pile surface) and not on the applied pressure.
- Average increase of friction resistance will be in the range of 1.5 to 3 times that of plain piles.
- No reduction of the friction resistance will be envisaged over long period of time.

8. THE TECHNIQUE IN DETAIL

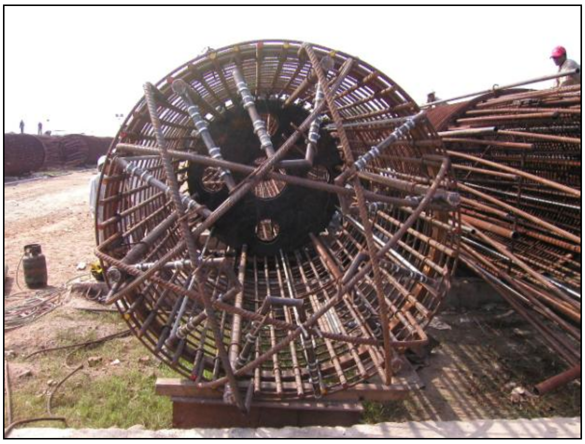
8.1 Base Grouting

Base grouting is the technique that is to provide cement grout injection from the bottom tip of the constructed piles. The grout material will be delivered through the installed grout pipe to the bottom of the pile where a set of Tube-a-Manchette (TAM) is installed. The base grouting takes care of possible disturbance at the pile tip caused by the drilling operation or accumulation of bentonite sediment. The process of base grouting compresses the loosened pile tip and improves the end bearing. Further the upward travel of grout along shaft, creates a sort of socketing effect, thereby improves skin friction as well. This greatly enhances the overall pile capacity.

8.2 <u>Grout Procedure</u>:

8.2.1Grout tubes

The grout arrangement consists of 6 nos TAM tube circuits in the form of U- loops one end attached to bottom of sonic logging pipe and other end to a 3/4" dia pipe for grout injection. Each U – loop has 4nos manchettes. Figure 2 shows the fabrication of base grout pipes.



Figue 2. Fabrication of Base Grout Pipes

8.2.2 Grout process

After the initial setting of pile concrete, inject water under pressure through grout pipe to crack the cover concrete. When the cover concrete cracks, sudden drop in pressure can be noticed and grout paths will be formed. Inject the grout mix through 3/4" dia pipe and manchettes at pile tip. The cement grout penetrates into soil formation immediately below pile tip.

8.2.3Grout Parameters and Target Volumes

1. The grout is injected in two rounds through the 6 nos – U loops one after the other.
2. The grout volume is limited to 200 liters through all 4 nos manchettes for each loop.
3. The grouting in 2nd round is carried in the same process as above but the grouting operation is ceased if the observed grout pressure becomes higher than 60 bars.
4. When grout pressures are low compared to the corresponding overburden pressures or initial cracking pressures, the grout injections are continued but not more than 200liters per U-loop in the 2nd round.
5. Maximum limit of overall grout volume is considered as 2400 liters through all 6 nos U-loops in both the rounds.

8.3 Shaft Grouting

Shaft Grouting is achieved by cracking the pile concrete surface with high pressure water through TAM grout pipes which are attached to the outside of the pile reinforcement cage and the grouting through the crack will be applied as a continuous process for the surface of the pile surface in the regular intervals. Cracking the concrete of the pile and pushing it out against the surrounding soil and subsequent grouting develops the lateral pressure and hence the density of the soil which had otherwise been soften during the excavation. The grout travels through the least resistance and fills the inter face zone as well as any voids or cavities in pile shaft.

8.3.1Grout Procedure

8.3.1.1 Pipe Arrangement

TAM are made of 1 ½" dia GI pipe with 35 manchettes spaced at 1m c/c. Each manchette has 2 nos holes drilled in the body of the GI pipe at 180° position which are covered by PVC or rubber sleeve with the provision of two holes at 30° apart thus acts as a non-return valve. The holes in the sleeve are positioned toward the surface of the piles in order to make sure that the pressured water and the grout will be injected outside. 2" dia plain pipe is installed within the non-grout zone where the permanent casing are positioned in the pile shaft and the pipe is tapered down to the 1½" dia TAM for the grout zone. Figure 3 shows the fixture of TAM pipe on Pile Cage. Figure 4 shows the TAM arrangement.

Figure 3. Fixture of TAM pipe on Pile Cage

The larger diameter pipe at the upper portion helps easing the insertion of double packered grout pipe.

8 nos of GI TAM pipes are equally spaced along the periphery of pile shaft. The manchettes holes are positioned at 250mm staggered with the adjacent pipe holes and such a manner that the series of holes will form a spiral shape.

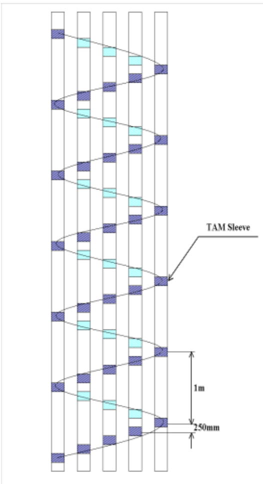
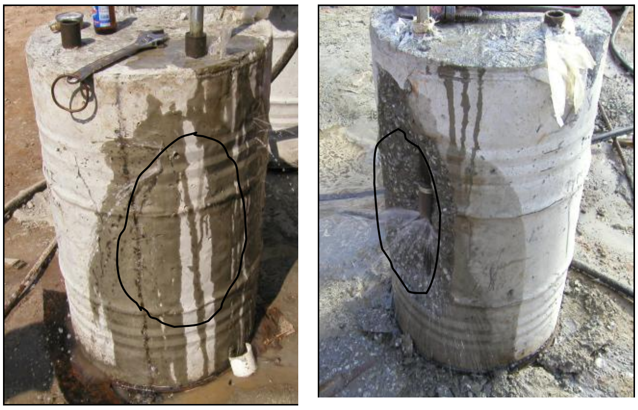


Figure 4: TAM Arrangement

The spiral arrangement of manchettes holes is considered to be a good practice to ensure that the grout spreads evenly over the entire shaft area and to provide the continuous spiral grout lines.

8.3.2.Cracking Operation

Although there is a case report from Kuala Lumpur that water cracking of the manchettes could be dispensed and instead the manchettes were cracked as part of the grouting operation, we considered that the cracking operation was important in order not to miss the grouting time and in order to provide a good grout path for the subsequent grouting.
In order to ascertain the timing of the cracking operation and its water pressure, a trial was carried out using concrete in drums with same TAM arrangement. Figures 5 a & 5 b show crack trails at 24hrs and 51 hrs respectively.



a. b.
Figure 5 Crack Trial at 24 hrs (left), 51 hrs (right)

The results of the trial show that cracking should be taken place later than 24 hours after the concrete casting as it was found that early cracking (even after the initial setting, say in 12 hours) will create only hair cracks to the concrete cover and there was a chance of closing the crack again by the lateral soil pressure. In fact it was observed that the harder the concrete, the wider the width of cracking was created and the cracking could carry out even after the concrete reaches the design strength (30Mpa) with cracking pressure not more than 50 bars. The shape of the crack was always elliptical cone-shaped. With this result, we set out the simple rule as to the timing of cracking that the operation should not be taken place before 24 hours of the last pour of the pile concrete.

In the real field condition at TP3 and Main piles, 3/4" dia GI pipe with double packered grout injection hole is positioned at the correct level of TAM from bottom and inject the pressured water to crack the cover concrete and create grout paths as a continuous operation for 35 steps per grout pipe and 8 pipes a pile. The pressure was normally built up to 20~30 bars and the sudden pressure drops indicates the moment of the cracking the concrete.

8.3.3. Grouting Operation

The shaft grouting to the pile shaft was divided in 3 zones from the bottom (12+12+11 steps).
Grouting is carried upwards step-by-step starting from bottom most step of the pipe. The grouting of the first 12 steps was carried out continuously for all 8 nos pipes in sequence and continued the operation of the remaining zones in the same manner for total of 35 steps. By this way, injection of grout in spiral form and uniform spread of grout around the shaft were ensured. This process also facilitates to inject compensatory additional grout from the adjacent step when target grout volume is not achieved in a particular step due to any blockage or excessive grout pressures.

8.3.4. Grout Parameters and Target Volumes

- The target volume was set out as 50 liters/m^2 (means per step) While many of reports for other projects are set out as 25~35liters/m^2.
- Grout injection in 1st round is maximum 50 liters per step for all 35 steps and the total volume for all 8 pipes is 14,000 liters.
- If the grout pressures was low compared to the corresponding overburden pressures or to the initial cracking pressure, the grout inject- ions are to be repeated at the particular zone but limiting the volume to 35 liters.
- In case the grout pressures are relatively high compared to overburden pressures the 2nd round grout injection is suspended.

Although we set out the criteria of 2nd round grouting and made a provision of re-grouting by way of washing the TAM after the grouting, the 2nd round of grouting was not necessary as the grout pressures constantly reached more than overburden pressures. (Max. 15 bars at the tip level) This is in fact in line with the conclusion of reports of other projects such as by Littlechild et al (1998) among others.

8.3.5. Records During the Progress of Base and Shaft Grouting

- Date and time of start and end of grouting for all steps, Manchette number, depth of grouting point.
- Individual and cumulative grout volume, grout injection rate.
- Cracking and grouting pressures.
- Pile up lift if any.

8.3.6. Grout Mix Proportions

The mix proportions for both base and shaft grouting are carefully designed to have good flowability, long term durability and adequate strength to make the shaft grouted pile act homogeneously. Following to the other reported case, we concluded to use the grout mix of following proportions aiming to gain 28 days strength of 20MPa.
Water (0.74) : Cement (1.0) : Bentonite (5% of Cement) : Admixture (1.20cc/1kg of Cement)
Cement : Portland Type I (Holcim and SIAM Brands)
Admixture : Mighty 150 Superplastisizer

8.3.7. Quality Control, Sampling and Testing

- Field grout samples are collected spread over the entire day's work and following parameters are verified to compare with approved design mix
- Marsh Cone Viscosity – 55 +/- 5 sec
- Mud balance Density – 1.65 to 1.75 g/cc
- 100mm grout cubes of 9 nos per day's work
- In addition to above percentage bleeding and setting times of grout are also monitored.

8.4. Osterberg Cell and TP3

Osterberg Cell Pile Loading method was invented by Dr. Jorj O. Osterberg, Professor Emeritus of Civil Engineering at Northwestern University and the method is innovative effective method for testing high capacity drilled shafts and piles. The Osterberg Cell, or O-Cell is a hydraulically driven, calibrated, sacrificial jacking device installed within the foundation unit. Using the pile resistance by end bearing or skin friction, O-Cell apply load in two directions, upward against side-shear and downward against end–bearing and the method automatically separates the resistance data to skin friction and end bearing with pre-installed electrical sensor device, strain gauges.

In our case, 2 layers of twin O-Cell were installed, which enable us to divide the pile into three portions to monitor the each part of skin friction factor as well as the end bearing. First, the bottom O-Cell was mobilized using middle and upper shaft as resistance to detect the end bearing capacity and then middle portion O-Cell was mobilized downward, with releasing the bottom O-Cell free, using upper shaft as resistance to check the skin friction of the middle portion of shaft. Lastly, after locking the bottom O-Cell, the upper O-Cell was mobilized upward to detect the skin friction of the upper shaft.

TP3 was constructed with base and shaft grout enhancement measures upstream side of MP4. The Pile Loading Test (PLT) was carried out using O-Cell method by Loadtest Inc. Asia in January 2003. Figure 6 shows the assembly of O-Cell Jacks. Figure 7 shows the arrangement of TP3 with double layers of O-Cell.

Figure 6. Assembling of O-Cell Jacks

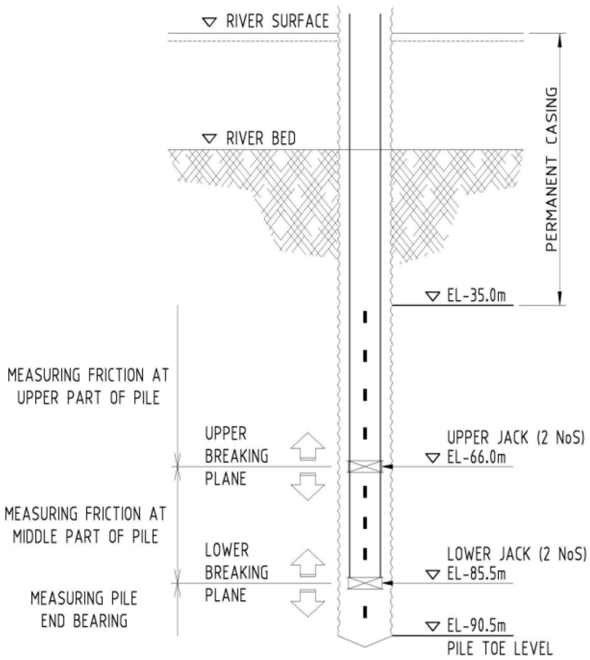


Figure 7. Arrangement of TP3 with O-Cell

After the PLT no.3, data of the test was analyzed to evaluate the capacity of the TP3 with figures of both end bearing and skin friction of the pile. Settlement of TP3 at the stage 1 loading of 2300 ton by the lower O-Cell was only 51mm with residual settlement of

39mm. Movement of the upper O-Cell was 41 mm at the stage 2 loading of 3400 ton by the upper O-Cell.

TP3 did not reach the ultimate stage by the maximum test load and skin frictions of the TP3 were still developing when the maximum load was applied. Then the ultimate pile capacity of the pile could not be obtained directly by the result of PLT no.3. However the minimum ultimate figure of the pile capacity can be estimated by the results of the PLT no.3 as shown on the Table 4. Figure 8 shows the pressure gauge at the end of the loading test.

From the result of TP3, it can be said that the expected ultimate pile capacity is more than 8500 ton and the long term capacity could reach as high as over 10,000 ton due to the remolding effect of the surrounding soil over the years.
The mobilized average skin friction over the pile could be calculated as high as 16 ton/m^2 and the maximum skin friction detected by the installed strain gauge was 23.5 ton/m^2 at silt layer. TP3 with shaft grouting has demonstrated enhancement in skin friction as high as 5 to 7 times that of TP1 and TP2.

Table 4 : Ultimate Pile Capacity for TP3

Soil Type	Depth of Strata	Ultimate Pile Capacity			
		Skin Friction		End Bear-Ing	Pile Capa-City
		Unit	Total		
Unit	M	t/m2	Ton	Ton	Ton
Sand/ Silt 1	29.5	2.0	490		490
Silt 1	11.0	5.3	460		460
Clay	6.0	10.4	490		490
Silt 3	6.0	23.5	1,110		1,110
Sand	22.3	22.3	4,820	1,220	6,040
Weight				-250	-250
Total	80.0		7,370	970	8,340

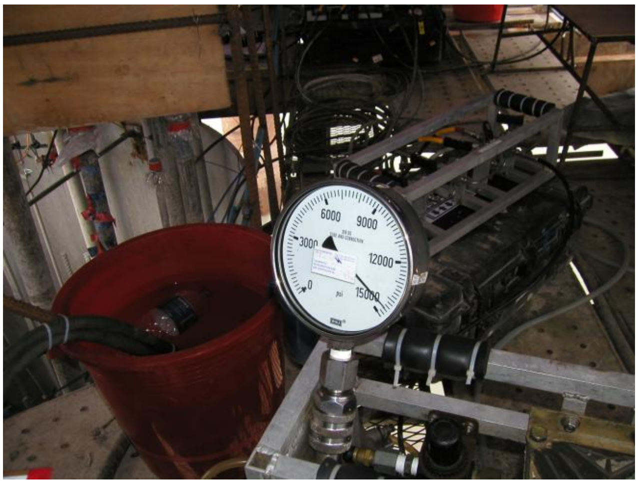


Figure 8. Pressure Gauge at the end of the Loading Test

8 EXECUTION TO MAIN PILES

Following to the above experience of base and shaft grout at TP3, the same techniques are adopted for Rupsa Bridge Main Piles from MP2 to MP6 in total 30 piles. The execution of the base and shaft grouting for those 30 piles were carried out with two sets of the grouting gangs and the entire operation was completed within 4 months time. Figure 9 shows the operation of the cracking work. Figure 10 shows the last grouting operation at MP4.

Figure 9. The Operation of Cracking Work

Figure 10. The Last Shaft Grouting at MP4

9 CONCLUSION

The result of TP3 demonstrated that the effect of enhancement of pile capacity with shaft and base grouting techniques could be achieved as high as 5 to 7 times that of plain piles which is far more than the previously reported 1.5 to 3 times in the other projects. Author consider that the following are the main reason of this result.

1) The increased volume of grouting (50 liters/m^2) compared with other reported case (25~35 liters/m^2).
2) The spiral shape of grouting pattern around the pile shaft.
3) Ensuring the minimum grouting pressure over the overburden pressure.
4) As reported, the marked improvements are seen in weaker the ground condition (N<50).

As the techniques using shaft/base grouting to enhance pile capacity of large diameter bored pile is still relatively new, the long term durability of the effect of the shaft grouting would be a natural concern. Only referable paper in this regard we have found was the reported case in Bangkok by Littlechild et al (1998) whereby reloading was applied on the shaft grouted piles over one year after the first load test and the result showed no loss of shaft resistance for two shaft grouted piles in alluvial sand and clay. In order to detect any change to the shaft resistance in the installed piles in Rupsa Project, We have installed strain gauges to the selected main piles for long term monitoring of the stress in piles and generated skin friction. No reduction of skin friction has been detected so far.

Adoption of such extensive shaft grouting technique is first of its kind in Bangladesh and authors consider that the experience in this project will be a predecessor for many challenging future projects in Bangladesh not only for resolving problematic piles but also for achieving more economical pile design by using this technique.

ACKNOWLEDGEMENT

This paper is published with the permission of Md. Mominul Huq, Project Director of Rupsa Bridge Construction Project, Roads and Highway of the Government of the People's Republic of Bangladesh. Special thanks are extended to Mr. Uchida of JBIC, Md. Mominul Huq, PD and Mr. Tapash Kumar Paul, PM of RHD, for their understanding of the merits of the implementation of shaft grouting technique, which was new technique to Bangladesh. Thanks are extended to the large number of DDC staff who were involved in the supervision of the technique on site. Thanks are also extended to staff of SIAM TONE Co., Ltd who executed the shaft grout work, staff of Teikeki Drilling Co., LTD who executed the construction of piles, Mr. Peter Jones, PM of Loadtest Asia PTE Ltd. who conducted O-Cell testing and Mr. Iwata and his staff of Shimizu-ITD Joint Venture, the Main Contractor.

Lastly but not least, many thanks must be extended to Messrs. Lesle H. Swann, Henry Leung of Babtie Asia (HK), Mr. Raymond Wong of Bachy Soletanche (HK), Dr. W. Teparaksa and of Chulalongkorn University, Bangkok, Thailand, Dr. Saifullah of BUET and Mr. Chris Round of Parsons Brinkerhoff Int. Inc. USA, TL of Paksey Bridge Project who provided us invaluable information regarding this technique and supported with their breadth of knowledge on pile foundation in general. Figure 11 shows the general view of the completed Main Bridge later inaugurated under the name of Khan Zahan Ali Bridge. Figure 12 shows the location of Khulna.

Figure 11. Main Bridge General View from North West side (2005)

Figure 12. The location of Khula in Bangladesh

11. REFERENCES

[1] Base and Shaft Grouted Piles :
V.M. Troughton and M.Stocker Proc Instn Civil Engineers, Geotechnical Engineering

[2] Enhancing the Performance of Large Diameter Piles by Grouting 1 & 2:
D.A. Bruce, Contracts Director, GKN Colcrete, Wetherby, West Yorks

[3] Full Scale Shaft Grouted Piles and Barrettes in Hong Kong – A First :
Brian Littlechild, Ove Arup & Partners, HK
Glen Plumbridge, Ove Arup & Partners, HK
Stephen Hill, Ove Arup & Partners, HK
Martin Pratt, Bachy Soletanche Group, HK

[4] Innovation in South East Asia :
Brian Littlechild, Ove Arup & Partners, HK
Glen Plumbridge, Ove Arup & Partners, HK
Stephen Hill, Ove Arup & Partners, HK
Martin Pratt, Bachy Soletanche Group, HK

[5] Method Statement and Quality Control Measures for Shaft Grouted Barretes :
Martin Pratt, Bachy Soletanche Group, HK

[6] Performance of Shaft Grouted Piles and Barrettes
Glen Plumbridge, Ove Arup & Partners, HK
Stephen Hill, Ove Arup & Partners, HK

[7] Pressure Grouted Minipiles for A 12-Story Residential Building at the Mid Levels Scheduled Area in Hong Kong
J.Y.H. Lui, S.P.Y. Cheung and A.K.C. Chan
Ove Arup & Partners Hong Kong Limited

[8] Shaft Grouted Piles in Sand Clay in Bangkok :
Brian Littlechild, Ove Arup & Partners, HK
Glen Plumbridge, Ove Arup & Partners, HK
M.W. Free, Ove Arup & Partners, HK