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*Reprinted from
Regional Symposium on Sedimentary Rock Engineering
Taipei, Taiwan
November 20~22, 1998, pp.173~178*

LOAD TESTS ON BORED PILES EMBEDDED IN SANDSTONES

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Abstract

A pile load test program comprising eight instrumented cast-in-situ piles was conducted for studying the effectiveness of base grouting on improving bearing capacity of piles. The diameters of these piles were 850 mm or 1500 mm and the lengths of them ranged from 18 m to 25 m. These piles were embedded in weakly cemented sandstones of which the degree of weathering varied from highly weathered to fresh with the blow counts of the standard penetration tests varying from 20 to greater than 100. In order to observe the effects of soft toe to the load transfer characteristics, 4 of these test piles were pressure-grouted at the bases. Results of the load tests indicated that the ultimate unit skin friction of the shaft embedded in highly weathered, moderately weathered and fresh sandstones were about 90 kPa, 100 kPa and 150 kPa, respectively, and were mobilized at the critical settlements of 1 to 10 mm. The base resistances of the piles which were founded in slightly weathered or fresh sandstones were up to 2.2 MPa and 4.4 MPa for the un-grouted and the base-grouted piles, respectively, at a settlement of 20 mm. This paper presents the performance of these test piles. The ultimate unit skin friction and the base resistance were correlated with the N values of the weathered sandstones so the experience can be applied to future analysis and design of bored piles embedded in similar geological formations.

INTRODUCTION

The design of piles generally requires the estimation of shaft friction and base resistance. However, experience is very limited on piles embedded in weathered rock. A static pile load test program which comprised of a total of eight test piles was carried out at a site which was located at the eastern edge of the Taipei basin in northern Taiwan. The piles were embedded up to 17 m into weakly cemented sandstone of which the degree of weathering varied from highly weathered to fresh. They were constructed by using the reverse-circulation method.

It was aware that piles constructed by using such a method would suffer from the soft toe problems. As reported by Moh et al. (1993) and Teparaksa (1994), a considerable amount of settlement would take place for the mobilization of end bearing. Pressure-grouting at the base of the pile as an improvement scheme was adopted by many professionals such as Stocker (1983) and Bustamante and Gouvenot (1983). In order to study the likely effects of

grouting, four of the test piles were base-grouted while the remaining four were un-grouted.

This paper describes the performance of these instrumented cast-in-situ piles. The load settlement characteristics of the un-grouted and the base-grouted piles are discussed. Design parameters of the unit skin friction and base resistance for sandstone of various degrees of weathering are suggested.

GROUND CONDITIONS

The site was covered with alluvial deposits underlain by weakly cemented sandstone. While the rock-bed was gently inclined the thickness of the alluvial stratum varied from 4 to 50 m. The weathering of the sandstone reached depths of, as much as, 20 m. According to the criteria recommended by BS 5930: 1981, the degree of weathering varied from highly weathered to fresh. Table 1 presents the engineering properties of the weathered sandstone. The

depth to the groundwater table ranged from 2 to 10 m.

INSTALLATION OF TEST PILES

The test piles were in two diameters, i.e., 1500 mm or 850 mm. They were embedded 7 to 8 m in the alluvial clay or sand and then penetrated 14 to 17 m into sandstone. Figure 1 shows, for example, the sub-strata at the location of test piles TP2 and BTP2. Table 2 presents the sub-strata encountered at the location of all the piles. The piles were divided into four pairs. Piles in each pair were

constructed at nearby locations so that they essentially encountered the same sub-strata except that one of them was grouted at the pile toe. The method of base grouting was similar to that reported by Stocker (1983), except that the injection valves were provided at the base of the pile. In short, U-shape grout pipes were attached to the reinforcement cage so that they were let down to the bottom of the pile. Pressure grouting with cement grout was conducted 12 to 24 hours after casting of concrete. Table 3 shows the record of grout injection.

Table 1 Engineering Properties of the Weathered Sandstone

Degree of Weathering	Grade	SPT Blow Counts		Uniaxial Compressive Strength kPa	Descriptions	Direct Shear Strength for Design	
		Range	Average			c', kPa	ϕ' , degree
Highly Weathered Sandstone	IV	10~34	23	80	Brown to yellowish brown silt	0	30
Moderately Weathered Sandstone	III	25~77	25	170	Brown silt	0	31
Slightly Weathered Sandstone	II	>100	>100	180	Grey weak rock	0	32
Fresh Sandstone	I	>100	>100	560	Grey weak rock	0	35

Table 2 Soil and Rock Encountered at Test Piles

Pile No.	Length in Soil or Rock, m						Total Length m	Diameter of Pile m
	Alluvial Deposits		Sandstone					
	Clay	Sand	Highly Weathered	Moderately Weathered	Slightly Weathered	Fresh		
TP1, BTP1	7	-	-	13	2	-	22	1.5
TP2, BTP2	3	5	5	4	5	-	22	1.5
TP3, BTP3	-	1	2	-	11	4	18	1.5
TP4, BTP4	4	4	-	9	8	-	25	0.85

Table 3 Summary of Record of Base Grouting

Pile No.	Injection Pressure MPa	Injected Volume Litre	Heave at Pile Head mm
BTP1	4.5	382	-
BTP2	4	1500	0
BTP3	6~7	240	2~6
BTP4	3	500	0

As shown in Fig. 1, strain gauges were installed on the reinforcement of the piles at the pile head, the toe and at changes of soil strata to monitor the stresses and strains in the piles. At each level, 3 strain gauges were spaced at angles of 120 degrees apart.

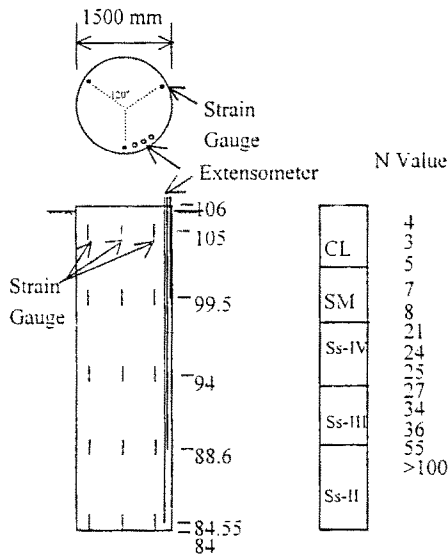


Fig.1 Instrumentation at Piles TP2

STATIC LOAD TEST

The test piles were loaded at 12 increments to three times of their nominal design loads and were unloaded at 6 decrements. The pile heads were jacked against reaction beams supported on two anchor piles, which had the same diameter with the test piles. Figure 2 shows the load-

settlement curves observed at the heads of the test piles TP2 and BTP2 and Table 4 summarized the settlements at pile heads of all the 8 test piles. As piles TP1 and BTP3 had pile head settlements exceeding 50 mm at design loads and pile BTP4 could not even reach the maximum test load of 6.3 MN, these piles were regarded as unsuccessful piles. Core-drilling revealed the presence of sediments at the

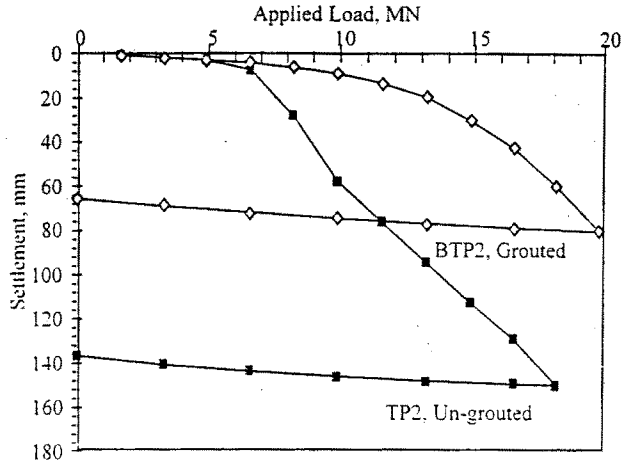


Fig. 2 Settlement at Head of Piles TP2 and BTP2

base of TP1, inferior concrete at the toe of BTP3, and inferior concrete at the depth of 17.6 m of BTP4.

LOAD SETTLEMENT CHARACTERISTICS

The design of piles using the load transfer method was proposed by researchers such as Chang and Goh (1989) and Moh et al. (1993). The load settlement characteristics of piles can be interpreted from the results of the strain gauges. The unit skin friction for each segment of the pile can be deduced from the load distribution curves as follow:

$$f_{si} = (P_i - P_{i-1}) / A_{si} \tag{Eq.1}$$

where: f_{si} is the unit skin friction for the segment between levels i and j , P_i is the load recorded at level i and A_{si} is the area of that segment of the pile shaft.

For the base resistance,

$$f_b = P_i / A_i \tag{Eq.2}$$

where: f_b is the base resistance, P_t is the load recorded at the toe of the pile and A_t is the area of the toe.

Table 4 Summary of Results of Static Load Tests

Pile No.	Diameter of Pile m	Length of Pile m	Design Load MN	Max Test Load MN	Settlement at Pile Head, mm		Comments
					At Design Load	At Max. Test Load	
TP1	1.5	22	6.7	19.8	77	158	Soft Toe
BTP1			6.7	19.8	8	74	Good Pile
TP2	1.5	22	6.7	18.5	8	150	Good Pile
BTP2			6.7	19.8	3	83	Good Pile
TP3	1.5	18	6.7	19.8	3	110	Good Pile
BTP3			6.7	14.9	50	152	Inferior Concrete at Toe
TP4	0.85	25	2.1	6.3	2	98	Good Pile
BTP4			2.1	3.2	3	102	Inferior Concrete at Depth of 17.6m

Figure 3 shows the load distribution curves for the base-grouted pile BTP2. About 40% of the applied load was transferred to the base of the pile at the maximum load. The abnormal readings recorded at the depth of 6.5 m may be due to malfunctioning of the strain gauges at that depth.

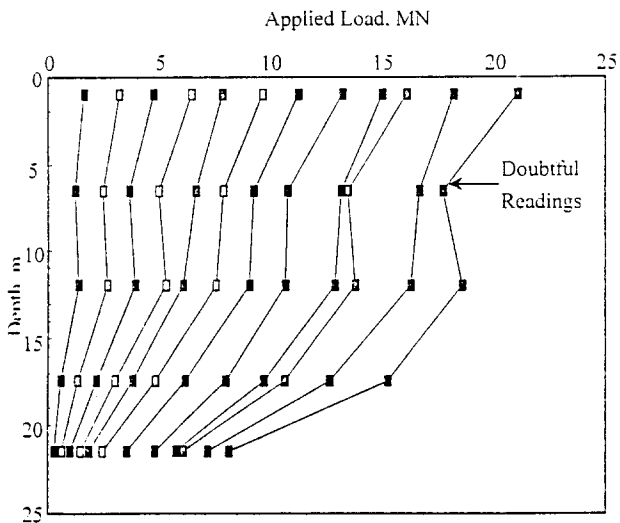


Fig. 3 Load Distribution Profiles for Pile BTP2

Figures 4 and 5 present the unit skin friction versus settlement relationships, i.e., the t-z curves, for piles TP2 and BTP2. It can be noted that the unit skin friction values for various soils or weathered rock materials attained the ultimate after the critical settlements were reached. The large variation in the unit skin friction may be due to malfunctioning of the strain gauges at the second level. If the results of these strain gauges were ignored, the shaft resistance of the alluvial deposits is about 50 kPa, which was similar to that obtained by TP2.

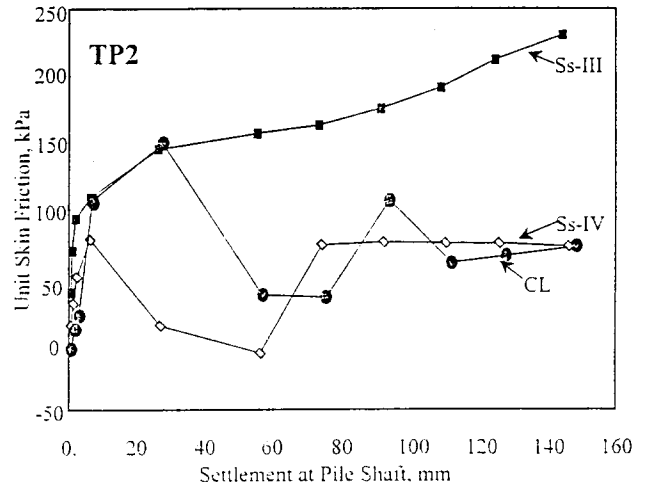


Fig. 4 Unit Skin Friction Versus Settlement (t-z) Curves For TP2

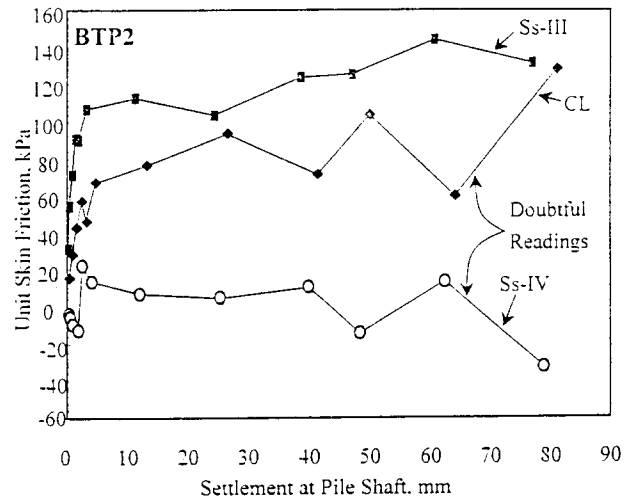


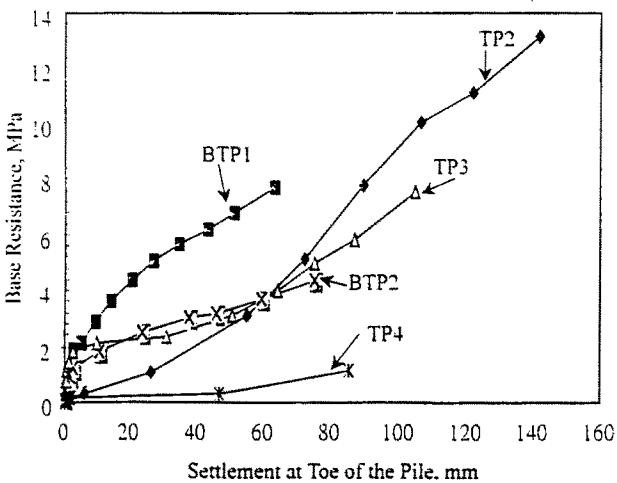
Fig. 5 Unit Skin Friction Versus Settlement (t-z) Curves For BTP2

Table 5 Load Settlement Parameters for Weathered Sandstones

Material Type	Average N Value	Critical Settlement mm	Ultimate Unit Skin Friction f_s , kPa	Base Resistance at 20 mm Toe Settlement, f_b , MPa		Correlation with N Value, kPa			
				Un-grouted	Base-grouted	$\frac{f_s}{N}$	Un-grouted $\frac{f_b}{N}$	Base-grouted $\frac{f_b}{N}$	
Silty Clay	3	4~12	20	—	—	6.7	—	—	
Silty Sand	23	3	80	—	—	3.5	—	—	
Highly Weathered Sandstone	23	2~10	90	—	—	3.9	—	—	
Moderately Weathered Sandstone	45	1~8	100	—	—	2.2	—	—	
Slightly Weathered Sandstone	> 100	2~10	150	0.8~2.2	2.4~4.4	1.5	< 15	< 34	
Fresh Sandstone	> 100	1~2	150	—	—	—	—	—	

Table 5 summarizes the ultimate unit skin friction values and the corresponding critical settlements of the materials encountered by these test piles. The ultimate unit skin frictions for the highly weathered sandstone, moderately weathered sandstone and the silty sand vary

from 80 to 100 kPa, which are not much different from each other. On the other hand, the ultimate unit skin frictions for the slightly weathered or fresh sandstone are about 1.5 times greater than those can be mobilized in the highly weathered or moderately weathered sandstone.



The base resistance versus settlement relationships, i.e., the so call q-z curves, of the slightly weathered sandstone are presented in Fig. 6. Unlike the t-z curves, the base resistance did not fully develop even the piles were loaded to 3 times of the design load. At a toe settlement of 20 mm, the base resistance of the slightly weathered sandstone ranged from 0.8 to 2.2 MPa for the un-grouted piles and from 2.4 to 4.4 MPa for the base-grouted piles.

CONCLUSIONS

Based on the results of static load tests conducted on these eight cast-in-situ piles, the following concluding remarks can be drawn:

- (1) The unit skin friction which was mobilized in slightly

weathered or in fresh sandstone is about 1.5 times greater than those mobilized in highly weathered or moderately weathered sandstone.

- (2) The critical settlements for the mobilization of the ultimate unit skin friction along the pile shaft range from 1 to 10 mm.
- (3) Base-grouted piles perform significantly better than un-grouted piles. At a toe settlement of 20 mm, the base resistance which was mobilized at the toe of the base-grouted piles is about 2 to 3 times greater than those of the un-grouted piles.

Because the unit skin frictions of piles in highly weathered or moderately weathered sandstone are not much greater than that of the silty sand, from the foundation engineering's point of view the highly weathered and moderately weathered sandstone shall not be regarded as the 'bedrock'.

The importance of quality control for the installation of cast-in-situ piles could not be over-emphasized. In this pile test program, three out of the eight test piles experienced the problems of soft toe or inferior concrete although they all were installed under closer supervision. In view of the fact that cleaning of the toe prior to casting of concrete would be difficult, it is envisaged that the application of base grouting on cast-in-situ piles would be a cost-effective solution to the problem.

ACKNOWLEDGEMENTS

The authors wish to express their sincere thanks to Department of Rapid Transit Systems, Taipei Municipal Government, for the kindly permission on the publication

of the relevant data reported in this paper.

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