BASE AND SHAFT RESISTANCE OF BORED PILES FOUNDED IN SEDIMENTARY ROCKS

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SYNOPSIS For the proposed rapid transit systems in the Taipei metropolis, over thousands of large diameter bored piles were installed to support viaduct structures. Many of these piles were founded in sedimentary rocks, including sandstone, siltstone and shale. This paper presents a close review and analysis on the results of loading tests on two sets of instrumented piles to evaluate the base and shaft resistance mobilized in sedimentary rocks.

INTRODUCTION

The Mucha Line of the Taipei Metropolitan Area Rapid Transit Systems(TRTS) in Taiwan comprises a two-way medium-capacity transit guideway extending a total length of 13 km, mainly supported on viaducts, and 12 elevated stations. The northern half of the line runs across the Taipei Basin in a north-south direction, and turns south-eastward to approach the southern boundary of the basin. The southern half of the alignment penetrates the south-bound ridges of the basin through a rock tunnel, extends to the hilly terrain of Mucha Area - the southeastern outskirt of the Taipei City (Fig.1). The entire northern half of the line is supported on long bored piles, 80 to 120 cm in diameter and 30 to over 50 m in length, penetrating through the silty Sungshan Formation to rest on the Chingmei gravel. The piers of the southern alignment are also founded on piles, except in a few occasions where spread footings are adopted on shallow bedrock. The 100 to 150 cm diameter piles of the southern alignment mostly penetrate through residual soils or alluvial deposits of the Chingmei Creek with tips embedded in sedimentary rocks such as sandstone, siltstone, shale, etc. Local experience about the behavior of piles in such a ground condition is relatively scarce, compared to what is available for the northern alignment in the Taipei Basin, where extensive construction has taken place in the past years. During the design stage, based on literature and site investigation data, some assumptions were made on the base and shaft resistance of piles embedded in local sedimentary rocks. Two sets of preliminary loading tests were performed on instrumented piles to verify the design assumptions. This paper summarizes and discusses the results of the compression and tension tests.

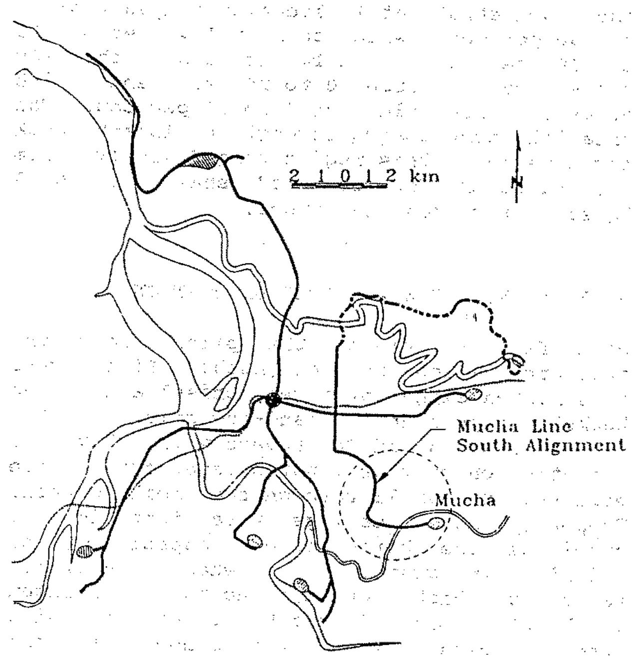


FIG. 1 PRIORITY NETWORK OF TAIPEI RAPID TRANSIT SYSTEMS

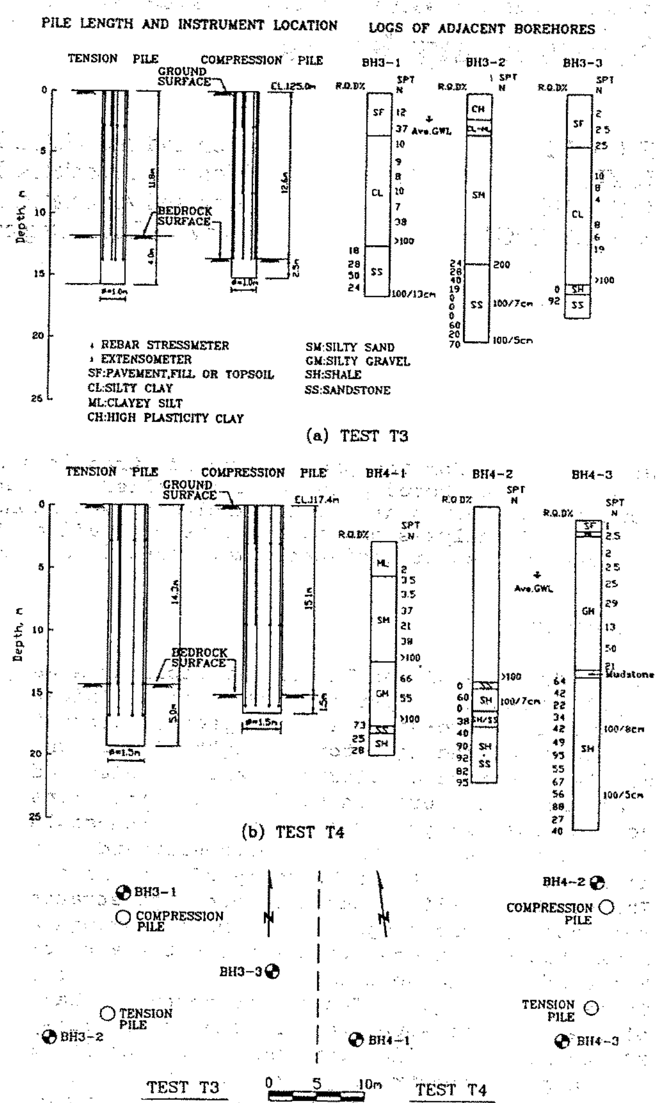
SITE AND GROUND CONDITIONS

The hilly terrain in the southeastern outskirt of Taipei City belongs to the Western Imbricate Fault Mountain Block physiographic province. Based on the geological report produced by The Ministry of Economic Affairs(1980), bedrock in the region is folded and faulted Tertiary Miocene Formation strata overlain by terrace deposits and recent alluvial deposits. The two sets of preliminary pile loading tests, namely T3 and T4, are located at two different locations in this region. T3 is located next to Station BR11 in a valley area. The soil strata here, consisting of residual silty sand and silty clay overlain by some backfilled soil, are about 12 to 15 m thick(Fig. 2a). Underlying the soil strata, the rock formation comprises sandstone and shale. The sandstone is gray or yellowish brown, slightly to moderately weathered, and medium strong to very strong. The shale , varying from fresh to highly weathered, is medium strong to weak. The other test set, T4, is located in the flat ground south of the main channel of Chingmei Creek , an area reserved as a stand-by flood channel for the Chingmei Creek. The soil deposit in this area is approximately 12 to 16 m thick comprising soft clay/silt, medium dense to very dense silty sand and gravel. The underlying rock strata consist of mainly weak to medium strong shale, occasionally with interbedded sandstone and shale(Fig. 2b). Figure 2c indicates the relative locations of the test piles with adjacent boreholes.

The Rock Quality Designation (RQD) values of the rock strata at T3 location ranged from 0 to 50 per cent, with occasional values of 70 to 90 per cent. At T4 location, the RQD values varied from 40 to 95 per cent, with occasional low values of 0 to 27 per cent. The uniaxial compressive strength of intact rock cores taken in this region ranged widely from 0.2 kg/sq.cm for weathered shale to 169.9 kg/sq.cm for fresh sandstone.

PILE INSTALLATION AND INSTRUMENTATION

With few exceptions, the piles along the southern alignment of Mucha Line were constructed either by auger, chisel and coring bucket or by means of hand-dug caisson. For the piles of T3 and T4, augers with kelly bar operated on crawler mounted base crane were employed to drill through the soil strata. Temporary steel casings were installed to stabilize the bored holes, especially for drilling in gravel and/or when groundwater table was high. Chisels up to a maximum weight of 8 tons and core barrel mounted with tungsten bits were utilized to penetrate the rock strata. When the required depth was reached, a cleaning bucket was used to remove soil and rock debris from the base of the hole. Groundwater inflow, mixed with soil, often formed a considerable amount of thick muddy slurry which was difficult to be removed by cleaning bucket. Clean water was then introduced before placement of reinforcement cage to dilute the slurry and air-lifting was performed to reduce the density of the slurry at the base of the hole, so that it could be effectively replaced by tremie concrete. Concrete through tremie pipe was poured to a level several meters higher than the required top level of the pile , in order to compensate for the voids left by the extraction of temporary casing , and to allow for chipping-off poor concrete. Finally, the casing was withdrawn before the concrete set.



(c) Relative locations of test piles and boreholes

FIG. 2 LOCATION OF INSTRUMENTS IN TEST PILES AND LOGS OF NEARBY BOREHOLES

Figures 2a and 2b present the lengths of the installed test piles in relation to adjacent ground stratification. The diameters of the piles are 1.0 and 1.5 m for T3 and T4 respectively. The loading tests for both T3 and T4 comprise a compression test and a lateral load test on one pile, and a tension test on another pile. Such an arrangement was made with the envision that the large test loads applied in an attempt to bring the piles to failure would cause large displacement and shear failure along the pile shaft. As such, a pile subjected to compression test would not be suitable for tension test any more, and vice versa, or else the latter test would be testing the residual strength rather than the intact strength of the pile/soil and pile/rock system.

Also shown in Figs. 2a and 2b are the instruments installed in the test piles, consisting of three sections, each with three rebar stressmeters and one extensometer, at three depths for each pile. During test loading, readings of these instruments were taken, in addition to the measurement of pile head movement, before and after each increment of load and at suitable intervals in between.

LOADING TESTS

Table 1 summarizes the pile test loads and associated pile head displacements. Reaction piles of 1.0m diameter linked up by large steel girders in layout as shown in Fig. 3 were adopted to provide reactions to the test loads as high as 17,000 to 20,000 kN. The compression load was applied simultaneously by 3 hydraulic jacks of 10,000 kN capacity each. The loading procedure was modified from the "Standard Loading Procedure" of ASTM D1143. It started with 4 incremental loading steps, each maintained for 2 hours, to half the estimated ultimate test load. At the 4th step, the load was first maintained for 12 hours, unloaded in 2 steps to 0, then reloaded in 8 steps to reach the estimated ultimate load. At that stage, the load-settlement curve and the stability of the load reaction system was closely examined. The test load was then increased gradually at small incremental steps until excessive displacement took place at the pile head, or until it was considered unsuitable to apply more load on the reaction system. Finally, the ultimate test load was released in 4 or 5 steps to 0.

The loading procedure for tension tests was similar to compression test, except that the load was applied by two hydraulic jacks acting against the top of two reaction piles.

TEST RESULTS AND ANALYSIS

Figure 4 presents the load-displacement curves of both compression and tension tests of T3 and T4. Results of the unload-reload processes were omitted to maintain clarity. It can be seen that the tension test of T3 reached the status of failure when the tension load exceeded 4,000 kN, whereas T4 pile received tension load of 10,000 kN with its large diameter(1.5 m) and long socket in rock(5 m). On the other hand, the pile heads of both T3 and T4 settled gradually to as much as 129 mm with increase of compression load, indicating a progressive development of end bearing capacity.

TABLE 1. TEST LOAD & CORRESPONDING PILE HEAD DISPLACEMENT

Test No.		T3		T4	
Load Type		Compression	Tension	Compression	Tension
Pile	Diameter, m	1.0	1.0	1.5	1.5
	Total Length, m	15.0	15.8	16.6	19.3
	Length of Socket in Rock, m	2.5	4.0	1.5	5.0
1st cycle	Test Load, kN	7500	2000	7500	4250
	Head Displacement, mm	25	4	14	3
2nd cycle	Ult. Test Load, kN	17000	4000	20000	10290
	Ult. Head Displacement, mm	111	26	129	24
	Residual Displacement, mm	103		112	10

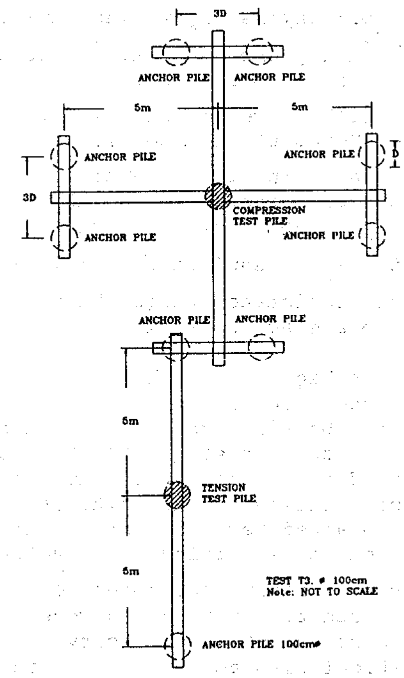


FIG. 3 TYPICAL LAYOUT OF TEST PILES, REACTION PILES AND REACTION BEAMS

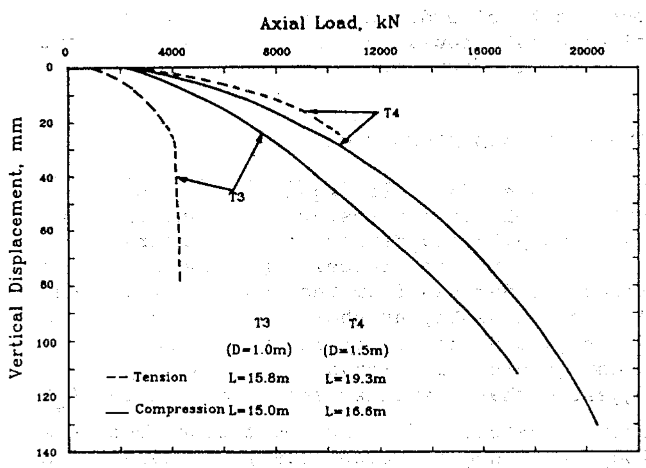


FIG. 4 AXIAL LOAD VERSUS PILE HEAD DISPLACEMENT

As part of the extensometers were out of function, the results of rebar stressmeters were used for calculating the strain of the pile shaft at each instrumented section. Based on the strains, the distribution of applied load along the pile shaft and hence the shaft friction could be calculated. Figure 5 shows, as an example, the load distribution and accumulated shaft resistance along T3 pile under various applied tension load. Figure 6 summarizes the analyzed results of tension tests in terms of load transfer curves for various soil and rock strata. Generally, the ultimate shaft resistance, f_s, increases with the strength of the soil or rock. For soil layers, f_s varies between 40 and 80 kPa, roughly proportional to the SPT N values at a f_s/N ratio of 0.3 to 0.4. In the underlying highly weathered sandy shale layer(N>100, RQD=0_60 per cent), the f_s is about 140 kPa. In the deeper strata of sandstone and sandy shale (N>300, RQD=0_90 per cent), the f_s is about 280 kPa. The critical displacement, δ_{sc}, required to mobilize the ultimate shaft resistance ranges between 3 and 13 mm, with an average of 8 mm.

Presented in Fig. 7 are the load transfer curves obtained from the compression test of T3. The end bearing resistance developed progressively to almost 8,000 kPa at a displacement of more than 100 mm, not showing any sign of yielding. The ultimate shaft resistance obtained here is higher than that in the tension tests, giving f_s values of 100 kPa for soil strata and about 1,000 kPa for the weathered rock strata. However, there is some doubt about these curves which indicate critical displacement, δ_{sc}, to be 30 mm or more, much higher than the commonly accepted range of 4 to 10 mm(e.g. Chang and Broms, 1991; Toh et.al., 1989). For T4, the instruments of compression test pile were found out of order during testing, and no reliable data could be used to establish the load transfer curves.

COMPARISON AND DISCUSSION

Whilst more data are required to establish more reliable load-transfer relationships for designing pile foundation in sedimentary rocks in or near the Taipei metropolis, the parameters derived from the results of T3 and T4 tension tests, as shown in Table 2, could be regarded as a preliminary reference. Investigators like Horvath and Kenney(1979) suggested that a smaller shaft resistance (f_s) would be expected from pull-out test, but the study of Chang and Broms(1991) on cases in Singapore indicates that there is no clear

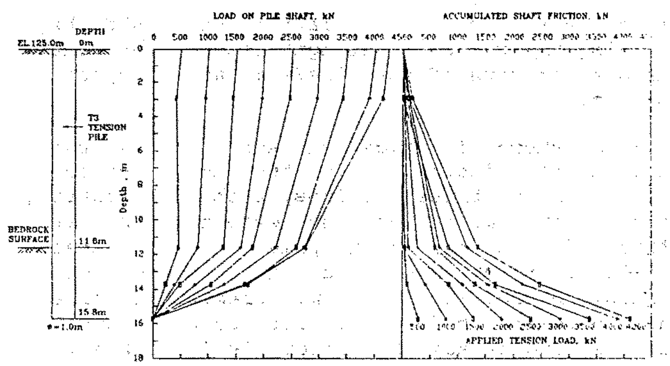


FIG. 5 VARIATION OF SKIN FRICTION ALONG PILE SHAFT DURING TENSION TEST

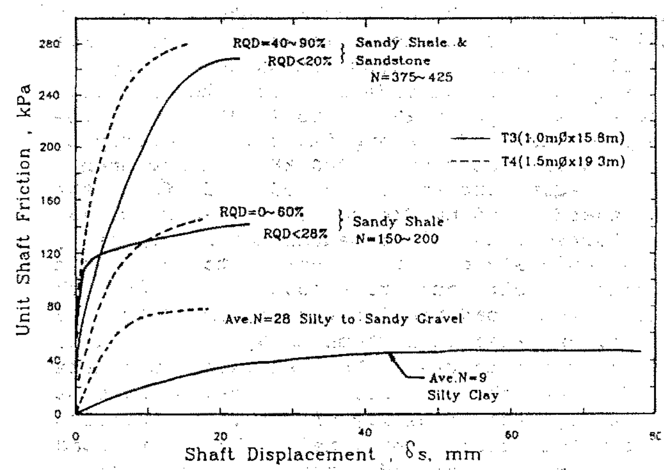


FIG. 6 LOAD TRANSFER CURVES OF T3 & T4 UNDER TENSION

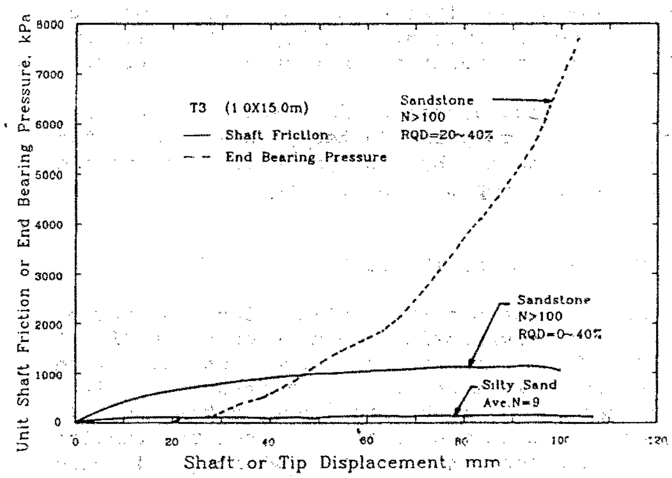


FIG. 7 LOAD TRANSFER CURVES OF T3 UNDER COMPRESSION

difference in the measured f_s values between piles loaded in compression and piles subjected to pull-out. Chang and Broms also suggested a f_s/N ratio of about 2 for augered cast-in-situ piles in the cohesive residual soil in Singapore. On the other hand, Reese and Wright(1977) proposed that f_s/N for sandy soil could be taken as 2.87, with f_s limited to a maximum of 160 kPa for soils with N>60. These f_s/N ratios or f_s values agree reasonably well with those given in Table 2.

Table 3 compiled the shaft friction resistance of bored piles in weathered sedimentary rocks, with SPT N value larger than 100 blows/0.3m, as reported by Buttling (1986), Chang and Wong(1987), Buttling and Lam(1988), Toh et.al.(1989) and Radhakrishnan & Leung(1989). It could be seen that the f_s/N ratio is generally less than 2 except the case reported by Toh et.al. It further shows that for SPT N value larger than 200, there is a decreasing tendency for the f_s/N ratio to become less than 1. The values of 0.65 to 0.8 obtained from T3 and T4 tension tests fall close to the lower bound of the range. It might be attributed to the groundwater inflow accumulated in the drilled hole which could have softened the borehole walls, bearing in mind that most sedimentary rocks like shale, sandstone and siltstone are very susceptible to softening by water. The other possible reason could be the transmission of the applied tension load through the reinforcement cage embedded in concrete, which brought T3 and T4 pile shafts into an elongated status in contrast with the compressed status of test piles in the cases reported by Chang and Broms(1991), where the uplift force was applied through an anchor plate at the pile base.

Under compression load, T3 and T4 piles behaved as partial friction and partial end bearing . Although a generalized conclusion cannot be drawn in quantitative terms, the compression test results shown in Figs. 4 and 7 provide a good demonstration of the progressive development of end bearing resistance with settlement in weathered sedimentary rock strata. The end bearing capacity, taking the sandstone at T3 pile location as an example, is only partially mobilized at a pressure of 7,000 kPa (axial load of 20,000 kN), but the associated tip settlement of 100 mm is in most cases, if not always, beyond practical tolerance. Therefore, for design of pile foundation in weathered sedimentary rocks, it is recommended either to rely mainly on the shaft friction and taking the tip resistance as an extra safety margin, or to take precautionary measures such as base grouting to effectively reduce excessive settlement caused by softening of the weathered rock to a tolerable limit, if tip resistance has to be relied upon.

TABLE 2. LOAD-TRANSFER PARAMETERS DERIVED FROM T3 AND T4 TENSION TESTS

Geologic stratum	SPT N values, blows/0.3m	Shaft resistance, f_s, kPa	Critical displacement, δ_{sc}, mm	f_s/N, kPa
Silty clay	ave.8	20～40	12	2.5～5.0
Silty sand or gravel	ave.30	60～80	8	2.0～2.67
Highly weathered sandy shale	150～200	120～140	8	0.8～0.7
Slightly weathered sandy shale and sandstone	375～430	240～280	8	ave. 0.65

TABLE 3. CASE RECORDS ON SHAFT FRICTION RESISTANCE OF BORED PILES AND SPT N VALUES IN SEDIMENTARY ROCKS

Reference	Geologic Stratum	SPT N Values, blows/0.3m	Shaft resistance, f_s, kPa	f_s/N, kPa
Buttling, 1986	Highly weathered siltstone	230	>195～226	>0.87～1
Chang & Wong, 1987	Highly weathered siltstone, silty sandstone and shale	100～180	100～320	1.0 ～1.8
Buttling & Lam, 1988	Very dence clayey/sandy silt to highly weathered siltstone	110～127	80 ～125	0.63～1.14
	Highly to moderately weathered siltstone	200～375	340	1.7 ～0.9
Toh et.al. 1989	Completely to partly weathered interbedded sandstone, siltstone	100～150	—	1.2 ～3.7
	and shale/mudstone	150～200	—	0.6 ～2.3
Radhakrishnan & Leung, 1989	Highly to moderately fragmented siltstone /shale	400～1000	300～800	0.5 ～0.8

CONCLUSIONS

Based on the tension test results of T3 and T4, a set of load transfer parameters was proposed as a preliminary reference for designing bored piles in the sedimentary rock strata in the outskirt of Taipei City. The parameters as shown in Table 2 agrees reasonably well with reported experience in Singapore and elsewhere in the world. The ratio of ultimate shaft resistance with N values from Standard Penetration Tests in the weathered rock strata concerned is found to be about 0.65 to 0.8 (kPa). More test data are needed to further substantiate or to refine the proposed parameters and correlations.

The test piles behaved as partially friction and partially end bearing under compression. The test results demonstrated that considerable amount of settlement took place with mobilization of end bearing capacity in weathered sedimentary rocks. Appropriate treatment, such as high pressure base grouting, appears to be necessary in order to control pile settlement within tolerable limit if high bearing stress is expected at the tips of a pile foundation.

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