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Drilled Micropiles Installed Through Reclamation Fill

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Summary Micropiles 250mm in diameter were installed through reclamation fill consisting of hydraulically placed sand. Beneath the fill was soft marine clay overlying overconsolidated alluvial deposits. The founding depths of the micropiles were up to 38.5m below the ground surface. Two load tests were carried out to study the performance of the micropiles with respect to the frictional resistance, end bearing resistance and displacement in both compression and tension. Chin's stability plot was used to determine the ultimate capacities of the micropiles tested. The importance of displacement assessment for slender micropiles and factor of safety in design is discussed.

1. INTRODUCTION

The Changi International Airport Extension Project in Singapore involved the construction of two additional finger piers (Pier C and Pier D) at Terminal 1. The extension will provide for increased passenger load and improved efficiency in movement of passenger traffic. Throughout the construction of the extension, normal airline ground operations had to be maintained. This meant that all works had to be carried out under the strict control of the Airport Authority.

The foundations for the main part of the new works were supported on large diameter bored piles. At the areas where the new structures were designed to abut against the existing Terminal buildings, micropiles had to be used due to space constraints. The micropiles were designed to act in groups varying from two to ten piles.

The new columns at the Terminal buildings were located eccentric to the centroid of the new micropile groups. In addition the new first storey floor slabs were designed to act in cantilever overhanging the new columns. Therefore unbalanced forces and moments caused by the permanent eccentric loading conditions were imposed on the micropile groups. The micropiles had to be designed to take loads either in compression or in tension to react against these unbalanced loads within each group.

The combination of column loads and overturning moments to be designed for are summarized below :-

Category	Column Load	Overturning Moment
1	2250 kN	4700 kNm
2	2500 kN	4700 kNm
3	2750 kN	4700 kNm
4	3250 kN	4700 kNm
5	3000 kN	7000 kNm
6	3250 kN	7000 kNm
7	3750 kN	7000 kNm
8	4750 kN	7000 kNm

The foundation for the passenger loading bridge at one of the new gates (C23) was also changed from bored piles to micropiles due to height restriction for the drilling rigs. As the loading of the bridge was not uniform with most columns supported on single micropiles in compression, tie beams were introduced to provide the necessary lateral bracing and structural stiffness in the horizontal plane.

2. SOIL PROPERTIES

The ground at the airport site was part of a massive reclamation programme completed almost 20 years ago (Choa, 1980). The reclamation fill consisted of hydraulically placed sand dredged from the sea. Beneath the fill was soft marine clay overlying overconsolidated alluvial deposits commonly referred to as the Old Alluvium. Fig.1 shows the typical soil profile at the present site based on the nearest

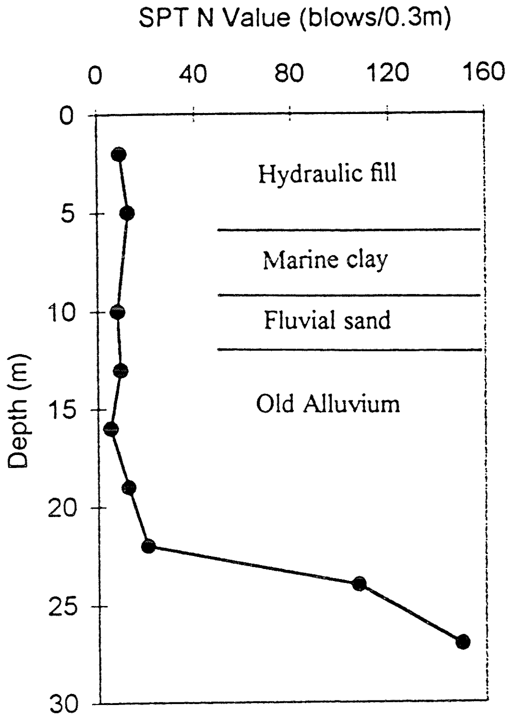

Figure 1. Soil profile at borehole BH8.

borehole BH8. The properties of the soils are summarized in Table 1.

3. STRUCTURAL LOAD CAPACITY

The allowable compression load (P_a) was calculated by applying a factor of safety of 2.0 on the ultimate structural capacity (P_u) of the grout and steel bars in compression assuming the micropile to be equivalent to a short braced column as defined by BS8110 (1985).

$$P_u = 0.4 f_g A_g + 0.75\, f_y A_{st} \qquad (1)$$

$$P_a = P_u / 2.0 \qquad (2)$$

where

A_g is the net bearing area of the grout (m^2)
A_{st} is the cross-sectional area of the steel bars (m^2)
f_g is the grout strength (N/mm^2)
f_y is the yield strength of the steel bars (N/mm^2)

Yield strength (f_y) of the steel reinforcement bars was 460 N/mm^2. The grout strength (f_g) was specified as 35 N/mm^2. For a micropile of 250mm diameter reinforced with five 40mm diameter high tensile deformed steel bars (A_g = 42,804mm^2 and A_{st}= 6,283 mm^2) the allowable structural capacity was determined to be 1383 kN.

The allowable tension load (T_a) was derived on the basis of restricting the stress in the steel bars to within 0.3 f_y at working load conditions to avoid cracking of the grout.

$$T_a = 0.3 f_y A_{st} \qquad (3)$$

Table 1. Typical Soil Properties (Borehole BH8)

Depth (m)	Soil Sample	Undrained Shear Strength (kPa)	Density (kN/m3)	Liquid Limit (%)	Plastic Limit (%)	Moisture Content (%)
4.0m	Loose to med. dense silty sand	-	18.2	-	-	26
6.0m	Soft to firm silty clay	35	17.6	68	23	33
7.5m	Soft to firm silty clay	-	15.9	70	23	54
9.0m	Loose to med. dense sand	-	21.7	-	-	21
12.0m	Firm to stiff silty clay	38	18.6	63	22	38
15.0m	Firm to stiff silty clay	60	19.9	52	19	29
18.0m	Firm to stiff silty clay	35	21.2	34	13	17
21.0m	Firm to stiff silty clay	45	21.0	35	15	20

On this basis, the allowable tension loads obtained for a 250mm diameter micropile with the various combination of reinforcements used were :-

for	5T40 bars	T_a = 867 kN
for	4T32 bars	T_a = 444 kN
for	3T32 bars	T_a = 333 kN

4. TESTING PROGRAMME

At the onset of the micropiling works, two load tests were conducted in the vicinity of BH8 to determine the behaviour of the micropiles in both compression and tension. Two separate micropiles (2F/WY12 and 2E/WY10) were installed for the compression and uplift tests respectively. Both micropiles were 250mm in diameter and reinforced with five 40mm diameter high tensile deformed steel bars. The test piles were located within 12.5m of each other.

Drilling was carried out under bentonite slurry support using roller button bits. Permanent steel casings were left in the ground due to the collapsible ground that was encountered in the upper weak soils. Pile penetration was 38.4m below ground for the compression pile and 38.5m below ground for the tension pile. The permanent casing for the compression pile was installed to a 15m depth, whereas that for the tension pile was installed to a 8m depth. In both cases, the boring logs of the test piles indicated that the soils up to 10.5m depth consisted of soft clays and loose sands (Fig.2). Below this was Old Alluvium comprising very stiff to very hard silty clays with abundance of sand. The soil from 27m and below was described as being very hard with SPT N value estimated to be greater than 100 blows/0.3m.

The Maintained Load Method was adopted for the load tests. A kentledge consisting of concerete blocks with steel beams was used to provide the required reaction loading of at least 2100kN in the compression load test. A hydraulic jack was placed directly over the pilehead together with a tilting saddle for load application. The applied test loads were measured by calibrated pressure gauges. The displacements of the pilehead were monitored by dial gauges read against two reference beams. Survey levelling of metric rules fixed to the pile head and reference beams was also implemented to counter check the dial gauge readings. Any displacement of the reference beams was corrected for by reading against a control datum bench mark set on one of the completed micropiles outside the testing area. The loads were applied in a single cycle to the maximum test load of 2000 kN (Fig.3). Each load increment was held for 1 hour except at 1000 kN and 2000 kN, where the applied loads were maintained for a period of 17 hours and 24.5 hours respectively. Rebound at zero load was observed for 3 hours.

For the uplift load test, the reinforcement bars were extended above the test pile through a through-hole jack to which they were coupled. The test load was

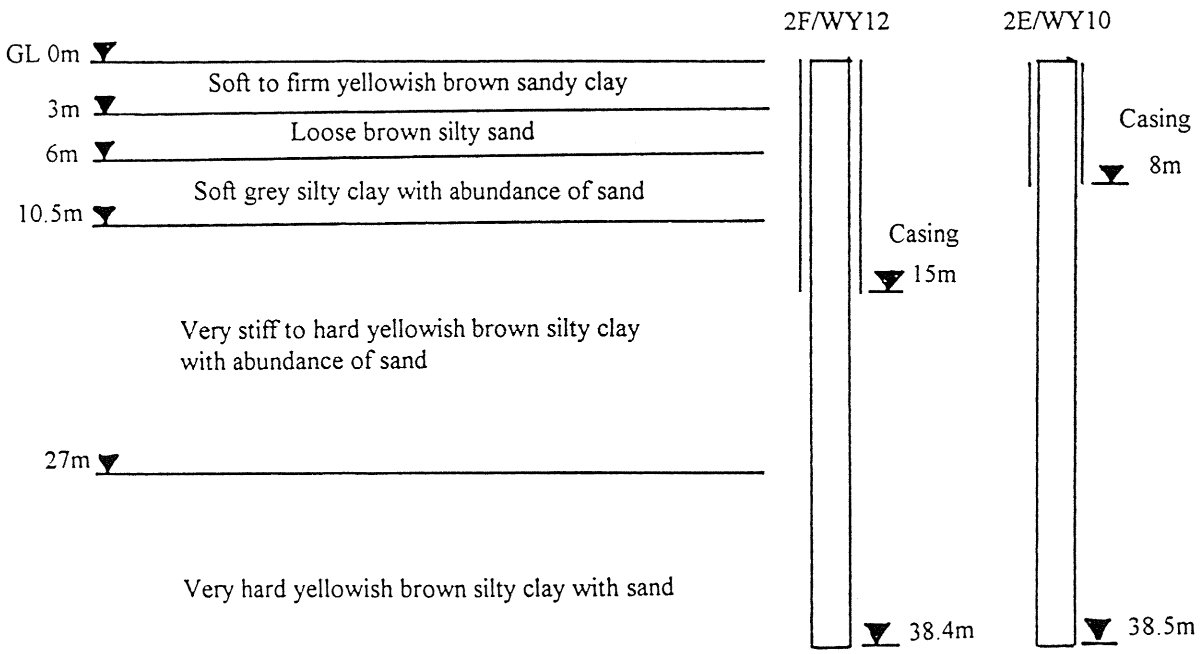

Figure 2. Details of test piles 2F/WY12 and 2E/WY10.

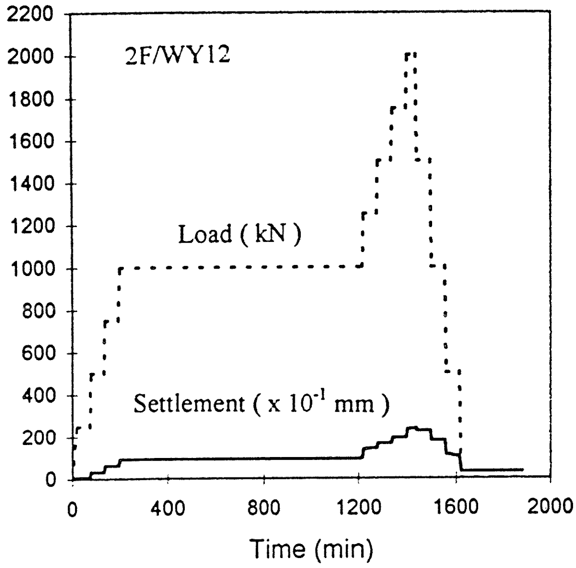

Figure 3. Compression load test.

applied by reacting against a cross beam seated on concrete blocks at each end. The method of measurement of the test loads and pilehead displacements were similar to that for the compression load test. For uplift testing, the loads were applied in one single cycle to the full test load of 860 kN (Fig.4). Load steps were applied at 1 hour intervals. At the maximum load, the load was maintained for a period of 22 hours. Rebound at final unloading to zero load was observed for 15 hours.

5. COMPRESSION SETTLEMENT

The load versus pilehead settlement behaviour of the micropile in compression is given in Fig.5. It can be seen that the settlement was 10.02mm at 1000kN and 23.97mm at 2000kN respectively. The relationship between load and settlement was observed to be relatively linear. The settlement per unit load was 0.01mm/kN at 1000kN and 0.012 mm/kN at 2000kN. Measurements of creep at 1000 kN was 0.72mm over 17 hours and that at 2000 kN was 0.66mm over 24.5 hours, which were insignificant. The residual settlement after the maximum test load was removed was 3.14mm.

Assuming that the length (L_1) of the micropile over which the casing was installed was free from any significant shaft resistance, the elastic shortening (Δ_c) of the pile can be estimated based on the following equation

$$\Delta_c = P/AE\ \{L_1 + 0.5L_2\} \qquad (4)$$

where P is the applied load (kN)
L_1 is the casing length (m)
L_2 is the pile length below the casing (m)
E is the equivalent Young's Modulus (kN/m^2)
A is the cross-sectional area of the pile (m^2)

The equivalent Young's Modulus was computed as

$$E = (E_gA_g + E_{st}A_{st})/A \qquad (5)$$

where

E_g is the Young's Modulus of the grout in compression (kN/m^2)
A_g is the net bearing area of the grout (m^2)
E_{st} is the Young's Modulus of the steel bars (kN/m^2)
A_{st} is the cross-sectional area of the steel bars (m^2)

Based on $E_g = 30 \times 10^6$ kN/m^2 and $E_{st} = 205 \times 10^6$ kN/m^2, the equivalent Young's Modulus of the pile was computed to be 52.4×10^6 kN/m^2. The elastic shortening of the micropile over the "free" length (15m) was estimated to be 11.7mm and that for the remaining portion below the casing was 9.1mm. The total computed elastic shortening was therefore 20.8mm. The difference between the observed pilehead settlement (23.97mm) and the estimated shaft shortening was 3.17mm and represents the pile toe displacement for mobilizing the end bearing resistance. It is interesting to note that this value of pile toe displacement was of the same magnitude as the measured residual pilehead settlement after the test load had been reduced to zero (3.14mm). This implies that the toe condition of the micropile was soft, possibly due to debris accumulation during installation, resulting in irrecoverable yielding of the supporting soil at the pile toe. However, based on the observed linear load - settlement behaviour of the micropile, it is likely that the base resistance had not been significantly mobilized and contribution of load resistance was mainly due to the shaft resistance.

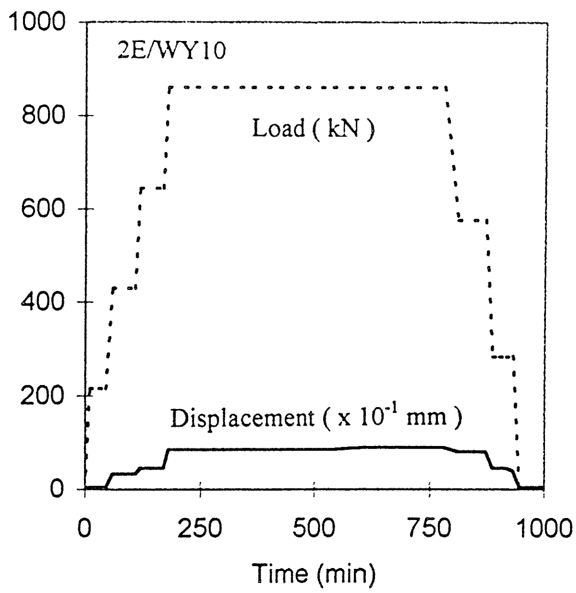

Figure 4. Uplift load test.

6. UPLIFT DISPLACEMENT

The load versus pilehead uplift behaviour of the micropile under tension loading is shown in Fig.7. An uplift displacement of 9.26mm was observed at the maximum test load of 860kN. Observed creep at maximum load was 0.52mm over a 22 hour period. As in the case of the compression load test, the relationship between load and uplift displacement was relatively linear. Upon unloading, the residual displacement was only 0.44mm thus indicating virtually full elastic recovery.

The elastic extension of the pile shaft (Δ_e) was estimated as follows

$$\Delta_e = T/(A\,E_t)\,\{L_1 + 0.5L_2\} \qquad (6)$$

where T is the applied uplift load (kN)
L_1 is the casing length (m)
L_2 is the pile length below the casing (m)
E_t is the equivalent Young's Modulus in tension (kN/m^2)
A is the cross-sectional area of the pile (m^2)

The equivalent Young's Modulus was computed as

$$E_t = (E_{gt}A_g + E_{st}A_{st})/A \qquad (7)$$

where E_{gt} is the Young's Modulus of the grout in tension (kN/m^2)
A_g is the net area of the grout (m^2)
E_{st} is the Young's Modulus of the steel bars (kN/m^2)
A_{st} is the cross-sectional area of the steel bars (m^2)

The value of E_t is dependent on the condition of cracking in the grouted pile shaft under tension loading. Ho (1994) and Ho and Lim (1998) reported that for bored piles subjected to tension loading, the Young's Modulus of concrete decreased very rapidly in a curvilinear fashion at the higher loading levels to a value less than 2.7% to 8.7% of the initial uncracked values. In the extreme case when the pile shaft at a given location is completely cracked, only the reinforcement bars would be effective in resisting the tension load. Cracking of the concrete would also result in a reduction of the steel-concrete interface bond strength which renders the load transfer between the steel bars to the outer pile shaft interface with the soil to become ineffective. Hence the skin friction resistance of the pile cannot be fully mobilized, and a lower geotechnical uplift capacity would result.

In the present test, the uplift displacement per unit load was computed to be 0.0108mm/kN. This means that the load - displacement relationship was almost identical in both compression and tension loading. It also implies that the micropile shaft was still intact and had not cracked at the maximum uplift test load of

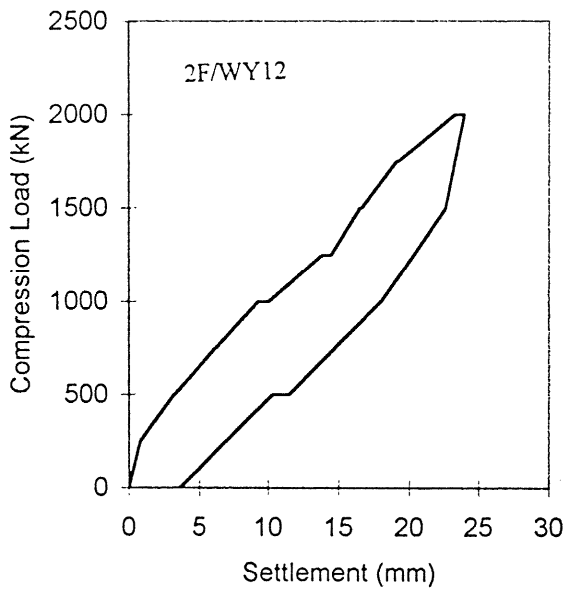

Figure 5. Compression load versus settlement.

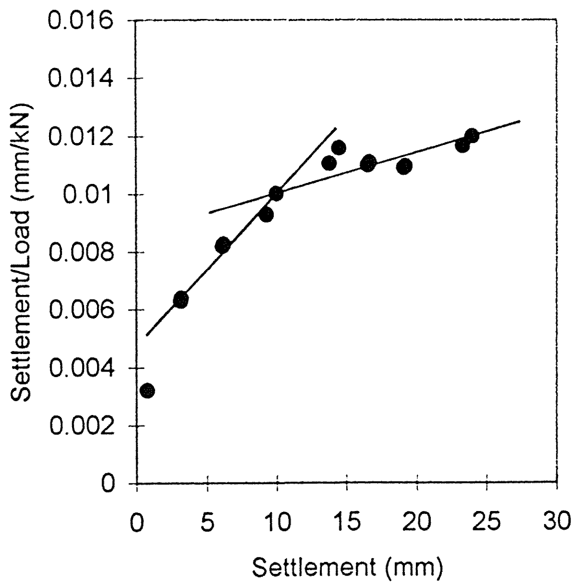

Figure 6. Chin's stability plot for 2F/WY12.

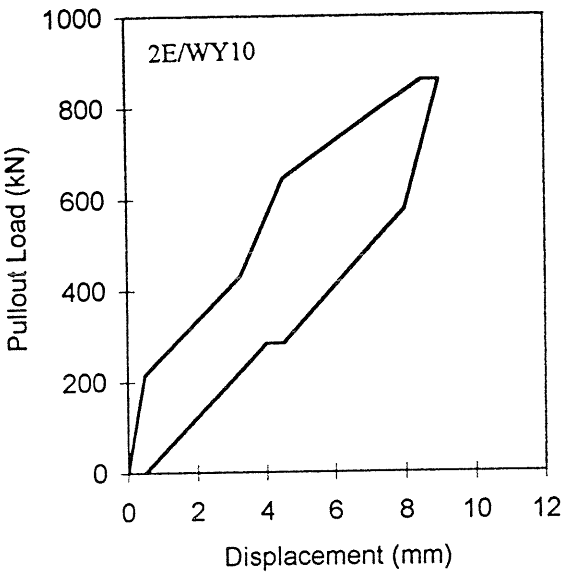

Figure 7. Tension load versus uplift displacement.

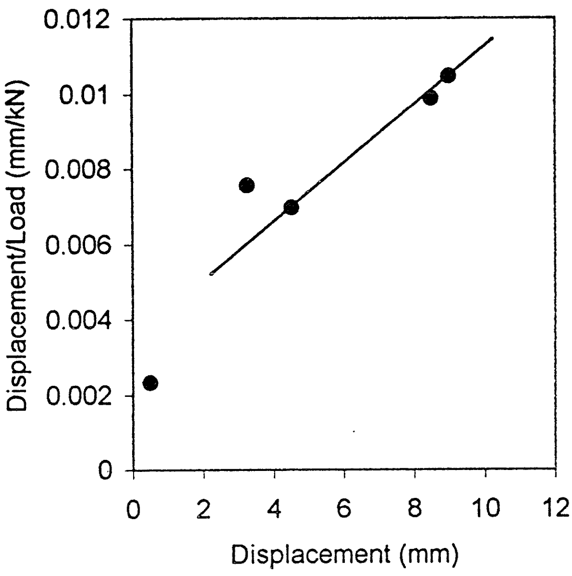

Figure 8. Chin's stability plot for 2E/WY10.

860kN. This condition of non-cracking can be attributed to the fact that the upper 8m of the pile shaft had been cased and the casing had provided additional confinement to the pile shaft, thus preventing the initiation of cracking. Based on the above argument, and assuming no cracking had occurred (i.e. E_t = 52.4 x 10^6 kN/mm^2 as for the compression test pile), the elastic extension of the micropile for the upper shaft length of 8m within the casing was estimated to be 2.7mm and that for the remaining pile length was 5.1mm, giving a total extension of 7.8mm. The theoretical value was comparable to the measured uplift displacement of 9.26mm at the pilehead. The difference represents the displacement in the soil, which was estimated to be of the order of 1.46mm. It can be inferred that the micropile had behaved elastically under tension loading and that a large proportion of the displacement was due to the extension of the pile shaft. Hence for such long slender piles, the amount of displacement contributed by the elastic extension of the micropile alone can be very significant and has to be carefully considered at the design stage.

7. GEOTECHNICAL LOAD CAPACITY

Chin (1970) showed that the behaviour of pile shaft resistance and base resistance in compression loading can be represented by a hyperbolic function with respect to pilehead displacement. A plot of pilehead displacement per unit applied load versus pilehead displacement would result in two typical straight lines. The first straight line is dominated principally by the shaft resistance, Q_s since the influence of the base resistance, Q_b is still small at this level of loading. The second straight line is representative of the total pile resistance, Q_u which is due to the combined effect of both shaft resistance and base resistance. The ultimate values of Q_u, Q_{su} and Q_{bu} can be estimated from the inverse slopes of the two straight lines.

Fig.6 depicts Chin's stability plot for the test micropile in compression. The corresponding values obtained from the inverse slopes were Q_{su} = 1876 kN and Q_u = 7099 kN. Q_{bu} was therefore predicted to be 5223 kN. However it should be noted that the potential base resistance can only be realized provided that the ultimate structural strength of the pile has not been exceeded. For the present test pile, the structural failure load in compression (P_f) was estimated to be 4388.3 kN based on $P_f = f_y A_{st} + f_g A_g$ assuming full lateral confinement. Hence the ultimate base resistance would be limited by this value.

The average ultimate skin friction resistance (f_{su}) over the uncased length of the micropile was determined to be 102.1 kPa. A value of 106,402 kPa was obtained for the ultimate end bearing resistance (q_{bu}). With reference to borehole BH8, the average N value between 15m to 27m was estimated to be 12.3 blows/0.3m and that for depths greater than 27m 128.5 blows/0.3m. The average SPT N value for the uncased length was therefore 68.9 blows/0.3m. On this basis, f_s/N = 1.48 and q_{bu}/N = 828.

Chin (1972) and Chin and Vail (1973) demonstrated that a pile subjected to pullout testing would display a single straight line since the behaviour of the shaft resistance with respect to pile uplift displacement is also hyperbolic. The inverse slope of this straight line will give the predicted ultimate pullout capacity. Fig.8 depicts Chin's stability plot for the test micropile in tension. Similarly, the ultimate shaft resistance (Q_{su}) was determined to be 1289.9 kN and f_{su} = 53.9 kPa.

Again, relying on SPT values given in BH8, the average SPT N value between 8m to 27m was estimated to be 10.8 blows/0.3m and that for depths greater than 27m, N = 128.5 blows/0.3m. The overall average N value obtained for the uncased length was therefore 55.2 blows/0.3m. On this basis, a value of 0.98 was obtained for the skin friction ratio (f_{su}/N). This f_{su}/N ratio was about 33.8% smaller than that for the compression load test.

8. DESIGN CONSIDERATIONS

It is common to adopt an allowable displacement at working load of 12mm for a single pile to ensure satisfactory performance of the foundation. From Fig.5, the allowable working load (WL) in compression loading to achieve this displacement was 1110.7kN. This value was less than the allowable structural capacity Pa = 1383 kN. The permitted pilehead displacement therefore limited the load that could be applied to the micropile and the full structural capacity of the micropile could not entirely be realized. The computed factor of safety ($F_s = Q_{su}/WL$) against the ultimate shaft resistance (Q_{su}) with respect to this allowable working load was 1.69 and the global factor of safety ($FOS = Q_u/WL$) against the total ultimate bearing capacity ($Q_u = Q_{su} + Q_{bu}$) which includes the base resistance component was 6.39.

The results of the compression load test showed that the ultimate capacity values were potentially very large. However due to the slenderness of the micropiles (L/D = 154) the applied loads have to be limited to a threshold value whereby pile head displacements would be within tolerable magnitudes. This can be achieved by applying a suitable factor of safety to the ultimate pile capacity available. However, the base resistance (Q_{bu}) for slender micropiles would be difficult to mobilize and utilize without incurring large settlements at the pilehead due to difficulty in cleaning the pile toe. Pile performance for a slender micropile would therefore be governed by the available shaft resistance. Hence, the allowable working load (WL) should be restricted to a value less than the ultimate shaft resistance (Q_{su}) as indicated above.

By limiting the stress in the steel to 0.3 times the yield stress (f_y) or a strain of 673$\mu\varepsilon$, an allowable pullout load (T_a) of 860 kN had been specified in the pullout test to avoid cracking of the grout. The corresponding factor of safety ($F_s = Q_{su}/T_a$) with respect to the ultimate shaft resistance in tension was 1.50. At this load level, the measured pilehead displacement was 9.26mm which was well within the 12mm limit. Since Fig.7 indicated that the load - displacement relationship remained linear without any sign of yielding, it was thought that the allowable pullout load could have been higher than 860 kN, as the upper 8m of the micropile was cased and fully confined.

The performance of a micropile in tension is governed by the uplift pile shaft resistance and the condition of cracking of the grout in the pile shaft. The design load not only has to be well within its ultimate shaft resistance (Q_{su}), but also limited to a magnitude such that the tensile strain imposed on the grout does not cause cracking at working load conditions. The maximum allowable pullout load capacity is hence limited by the structural capacity of the micropile rather than the geotechnical capacity.

9. CONCLUSIONS

Test results showed that the performance of long slender micropiles was governed by the elastic shortening or extension of the pile shaft. The relationship between load and displacement in compression and tension were very similar where there was no cracking of the grout in the micropile shaft. For compression loading, the base resistance, although potentially very large, was unreliable due to difficulty in cleaning the pile bases which may result in possible formation of soft toes. Pile performance was therefore dictated by the shaft resistance. It was shown that tolerable pilehead displacements can be achieved by applying a factor of safety (F_s) of 1.69 and 1.5 against the ultimate shaft resistance (Q_{su}) for compression and tension respectively. For tension loading, the allowable working load should be limited to a value such that no cracking of the grout would occur in the micropile shaft.

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