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BARRETTES DESIGNED AS FRICTION FOUNDATIONS:
A CASE HISTORY

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ABSTRACT

This paper describes the performance of a fully instrumented test barrette subjected to an ultimate loading of 30932 kN. The load transfer characteristics were measured by vibrating wire strain gauges. Rod extensometers recorded the displacements at several locations along the barrette shaft. Test results indicated that substantial loads were carried in shaft resistance. The end bearing component was reduced by poor toe conditions caused by debris accumulation at the trench base. The load-displacement behaviour and factor of safety of barrette foundation design is discussed. The load test results of a subsequent working barrette confirmed that the performance of the barrettes designed as friction foundations in the Old Alluvium is satisfactory.

KEYWORDS

barrettes, shaft resistance, end bearing resistance, displacement, factor of safety, instrumentation, Old Alluvium

INTRODUCTION

It is common practice to design barrettes as end bearing foundation elements founded in bedrock. This is due to a lack of confidence in the shaft resistance that can be mobilised at the shaft-soil interface as a result of the use of stabilising slurries. However, experience gained from instrumented barrettes tested in more recent projects in Singapore has shown that substantial shaft resistance could still be mobilised and that end bearing resistance was less reliable due to deposition of debris at the barrette toe (Ho, 1993; Ho and Tan, 1996). This paper presents data from full scale load tests at the Singapore Post Centre site in Singapore which opted for friction barrettes as part of the foundation system.

The Singapore Post Centre project comprised a 15-storey tower block, a 5-storey podium and a 3-level basement. The basement was constructed within retaining walls formed by diaphragm wall panels surrounding the site. The foundations of the building included both compression and tension bored piles, as well as barrettes. The barrettes were either rectangular panels or T-panels forming part of the diaphragm wall, or singular panels in groups. The barrettes were constructed through soft clays and loose sands and embedded into the Old Alluvium which is a dense to very dense overconsolidated sandy deposit. For the respective design working loads (WL), the barrette founding depths were determined on the basis of maintaining a minimum factor of safety (Fs) of 1.5 against the ultimate shaft resistance (Qsu), where Fs = Qsu/WL.

INSTRUMENTED ULTIMATE LOAD TEST

An instrumented barrette test panel TB1 was load tested to 30,932 kN to observe its performance at the ultimate failure condition. The size of the test panel was 2.8m x 0.6m. The toe level was 47.4m below ground. The test panel was excavated under bentonite slurry support. The barrette was cast with G40 concrete and overbreak was about 10.4%. The total time taken to complete the test barrette was about 4 days and 14 hours.

A total of 24 nos. vibrating wire strain gauges were fixed to the reinforcement cage at 8 levels along the barrette shaft, either in groups of two or four at each level. Where a group of two strain gauges was adopted, they were placed each at the middle of the panel length. For a group of four gauges, they were placed one at each face of the panel. Each strain gauge was installed on a T13 reinforcement bar 1m long using a microbond spot welder and pickup sensors were placed on top of the gauge before potting the assembly with epoxy in a
rubber hose. The reinforcement bar was then tied to the cage and the signal cables were routed to the top of the barrette and terminated in a junction box. The readings were displayed using a LCD (Liquid Crystal Display) indicator. Rod extensometers were also provided to measure the displacements at three different levels along the shaft. These consisted of a 8mm diameter stainless steel rod placed within a protective 13mm diameter mild steel pipe fixed with an anchor point. The lowering of the reinforcement cage into the excavated trench, together with the strain gauges and extensometers took 6 hours to complete.

The settlement of the barrette top was read using dial gauges and levelling survey. The displacement of the extensometers were measured by linear displacement transducers connected to a data logger. The loads were applied through four 1000t jacks spaced evenly in a row at the top of the barrette. Twelve strain gauge load cells were placed above the jacks to indicate the loads. Fig. 1 shows the details of the test barrette.

Load was applied to the test barrette 51 days after casting using the Maintained Load Method in three cycles. Each load increment and decrement was held for one hour. The maximum load in the first, second and third cycles were observed for 20 hours, 12.5 hours and 22 hours respectively, whilst rebound at zero load was observed for 3 hours, 7 hours and 3 hours respectively.

BARRETTE DISPLACEMENT

The load-displacement curves obtained from the ultimate load test is depicted in Fig. 2. At maximum load, the displacement was 43.0mm for the barrette top, 36.3mm, 27.8mm and 22.2mm for extensometers e1, e2 and e3 respectively. It was observed that the response of the barrette top remained virtually linear up to 20,025 kN and a top displacement of 12.3mm. This implied that the load carrying capacity was dominated by the shaft resistance up to this point. Beyond this, yielding started to occur and imminent failure was observed at about 30,932 kN. The maximum top displacement of 43.0mm reached at the maximum test load was 7.2% of the nominal barrette width of 600mm.

It was noted that there was a large difference in the residual top displacements at the end of the third cycle (24.8mm) compared with that at the end of the second cycle (3.7mm). This indicated that substantial plastic deformation of the soil had occurred at the maximum test load. The final residual displacement of 19.4mm registered by extensometer e3 for the barrette toe after the test load was removed indicated a rebound of only 2.8mm. This ‘soft toe’ response indicated that
Fig. 2  Applied load versus displacements for barrette TB1
there was debris trapped at the trench base during concreting.

LOAD TRANSFER CHARACTERISTICS

The measured strains in the barrette was used to obtain the load distribution along the shaft. The elastic modulus of the concrete varied with different levels of loading and was determined from the correlation between the measured strains at level A and the corresponding average applied stress at the barrette top. The maximum shaft resistance, $Q_{s\text{max}}$ and base resistance, $Q_{b\text{max}}$ obtained from the strain gauges were 24,729 kN and 6,203 kN respectively. The contribution of the base resistance was 20% of the maximum test load of 30,932 kN. The load transfer down the barrette shaft at the various stages of loading is depicted in Fig.1.

Fig.3 shows the development of skin friction resistance, $f_s$ calculated from the strain gauges. The measured values of skin friction resistance are summarised in Table 1. The skin friction ratios ($f_s/N$) obtained for barrettes and bored piles founded in Old Alluvium at other test sites (Ho and Tan, 1994; Ho and Lim, 1994; Ho and Tan, 1996) are shown in Fig.4 for comparison. It is observed that the skin friction ratios obtained for TBI follow closely the general trend for the other sites, and are all greater than the value of $f_s = 1N$ traditionally given by Meyerhof (1976) for the design of bored piles.

Fig.5 shows the development of end bearing resistance with the displacement of the barrette toe as represented by extensometer e3. The maximum stress mobilised at the base

![Fig.3 Mobilised skin friction resistance ($f_s$) versus shaft displacement ($d_s$) for each soil stratum](image)

<table>
<thead>
<tr>
<th>Strain gauge</th>
<th>Depth (m)</th>
<th>Soil Description</th>
<th>SPT value Ave. N (blows/0.3m)</th>
<th>Ave. SPT value N (blows/0.3m)</th>
<th>$f_s$ (kPa)</th>
<th>$f_s/N$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B - C</td>
<td>3.27 - 14.77</td>
<td>Med. dense sand / v. soft marine clay</td>
<td>0 - 16</td>
<td>-</td>
<td>27.1</td>
<td>-</td>
</tr>
<tr>
<td>C - D</td>
<td>14.77 - 19.27</td>
<td>Very soft marine clay</td>
<td>0</td>
<td>-</td>
<td>56.5</td>
<td>-</td>
</tr>
<tr>
<td>D - E</td>
<td>19.27 - 24.27</td>
<td>Stiff dessicated clay / med. stiff marine clay</td>
<td>4 - 10</td>
<td>6</td>
<td>101.2</td>
<td>16.9</td>
</tr>
<tr>
<td>E - F</td>
<td>24.27 - 31.77</td>
<td>Loose silty sand</td>
<td>5 - 11</td>
<td>7</td>
<td>72.7</td>
<td>10.4</td>
</tr>
<tr>
<td>F - G</td>
<td>31.77 - 38.27</td>
<td>Old Alluvium</td>
<td>18 - 69</td>
<td>41</td>
<td>93.1</td>
<td>2.27</td>
</tr>
<tr>
<td>G - H</td>
<td>38.27 - 46.67</td>
<td>Old Alluvium</td>
<td>85 - 107</td>
<td>96</td>
<td>167.4</td>
<td>1.74</td>
</tr>
</tbody>
</table>
was 3692.8 kPa at a displacement of 22.2 mm. The ultimate end bearing resistance \( (q_w) \) predicted using the method of Meyerhof (1976), \( q_w = 400N_b/3 \) was 13,333 kPa assuming \( N_b = 100 \) blows/0.3m. This implies that the ultimate end bearing resistance can only be mobilised at barrette top displacements much greater than 43 mm.

The toe response can be assessed using the coefficient of subgrade reaction \( k_s = q_b/d_b \) where \( q_b \) is the end bearing resistance and \( d_b \) is the toe displacement. The value of \( k_s \) obtained for TBI was 166.3 kN/m, implying that the base stiffness was lower than expected for such a dense to very dense sandy material with SPT N value greater than 100 blows/0.3m.

**DESIGN FACTOR OF SAFETY**

The ultimate bearing capacities of a foundation can be predicted from the inverse slopes of a stability plot, Chin (1972). The values of ultimate shaft resistance \( (Q_{su}) \) and total ultimate resistance \( (Q_u) \) derived for TBI using Chin’s method were \( Q_{su} = 35,780 \) kN and \( Q_u = 41,841 \) kN respectively, (see Fig 6). It is common to specify an allowable foundation displacement of 12 mm at serviceability conditions. For TBI, the limiting load \( (P_L) \) that could be applied in order to achieve 12 mm displacement was 20,025 kN (Fig.2). The factor of safety on the ultimate shaft resistance \( (F_s = Q_{su}/P_L) \) was 1.78 and the associated overall factor of safety which included the base resistance \( (FOS = Q_u/P_L) \) was 2.09.

**Fig. 4 Skin friction ratio (fs/N) versus SPT N value**

**Fig. 5 Mobilised end bearing resistance \( (q_e) \) versus toe displacement \( (d_e) \)**

**Fig. 6 Chin’s stability plot for barrette TBI**
An additional load test was also carried out on a permanent barrette BR85 in three cycles to 31,952 kN. The dimensions of BR85 were 2.8m x 0.8m and was constructed to 50.4m deep. The applied load versus barrette top displacement and the stability plot are given in Fig. 7 and Fig. 8 respectively. The limiting load (PL) at 12mm displacement was 21,689 kN. Based on Chin’s method, Qsu ≈ 30,487 kN and Qu ≈ 48,837 kN. The corresponding factors of safety were Fs ≈ 1.4 and FOS ≈ 2.25.

It can be seen that although the overall factor of safety (FOS) may be less than the conventional values of 2.5 to 3.0, the performance of the barrette remains satisfactory so long as sufficient safety margin (Fs) against the ultimate shaft resistance Qsu is available.

CONCLUSION

The performance of barrette foundations subjected to full scale load tests was discussed in this paper. Test results confirmed that the toe condition of barrettes was likely to be poor due to accumulation of debris at the trench base during construction. However, substantial shaft resistance can be mobilised to carry load satisfactorily with a reasonable factor of safety and within a tolerable displacement of 12mm at working load. The values of skin friction resistance achievable in Old Alluvium was demonstrated to be greater than the value of fs = 1N suggested by Meyerhof (1976) traditionally adopted for design of bored piles. The skin friction ratio (fs/N) was shown to be curvilinear and follows a decreasing trend with increasing SPT N values.

REFERENCES


