Piling design and load test considerations in underground stacked rail depot

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ABSTRACT

The East Coast Integrated Depot is the world's first 4-in-1 mass transit infrastructure project in Singapore which houses three rail depots and a multilevel bus depot on the same site. The three rail depots are stacked together, each serving independent Mass Rapid Transit (MRT) lines at the underground, at-grade and elevated levels.

The stacked rail depot is founded on over 2700 numbers of base grouted piles. With tight control on factors such as pile uplift, grout pressure and volume, base grouting can eliminate soil relaxation beneath pile toe, alleviate the tricky issue of ensuring good base cleaning and enables a more economical design for subsequent piling work.

Total 27 numbers (1% total piles) of ultimate and working load tests are carried out using the conventional Kentledge maintained load and Osterberg cell methods. This paper presents the design and considerations adopted in the piling design and discusses the results and findings, particularly on the pile load test responses of base grouted and non-base grouted piles using the two load test methods under the ultimate and serviceability limit state.

Keywords: pile load tests; base grouting; skin friction; end bearing; deep foundation; pile design

1 INTRODUCTION

The East Coast Integrated Depot is a multi-billion major mass transit infrastructure project. It adopts the new design philosophy from Singapore's Land Transport Authority (LTA) where the depots are stacked to minimize land use in the land scarce and densely populated Singapore. The foundation piles have been completed under an Advance Contract while the excavation and structural works are ongoing.

1.1 Site Location

The project site is located at the eastern part of Singapore, bounded by the Upper Changi Road East on the North, Sungei Ketapang on the South, Xilin Road on the East and Sungei Bedok on the West as shown in Fig. 1.

1.2 Proposed Rail Depot Structure

The underground rail depot stretches over 1.0km with spans varying from 145.0m to 360.0m giving an excavation area in excess of 23.0 hectares.

Diaphragm wall act as the temporary and permanent retaining system for the 15.0m deep excavation. The structural system consists of multi-span reinforced concrete of partial precast elements. These are held by columns generally spaced at 11.4m which are in turn supported by foundation piles and base slab of 1.0m to 1.5m thick.

Over 2700 nos. of bored and barrette piles between 24.0m to 50.0m deep are proposed to transfer the loads to competent ground. The pile size ranges from 1.2m to 2.5m dia for bored piles and 1.5m width x 3.0m to 5.0m length for barrette piles.

Fig. 1. Site Location.

2 GEOLOGY AND SUBSURFACE CONDITION

From published geological literature, the stratigraphy includes Fill material overlying the recent Quaternary deposits of Kallang Formation consisting of Alluvial and Marine members followed by the late tertiary Old Alluvium (OA) deposits.

Based on approximately 210 nos of boreholes carried out during the design and construction stages, sandy Fill material up to 14.0m thick is encountered.

The thinnest Kallang Formation is 5.0m at the eastern site boundary and increases to 20.0m thick towards the canals. The most prominent member of the Kallang Formation which is encountered in almost all the boreholes is the very soft and compressible Marine Clay member. It is at times interbedded between the fine to coarse fluvial sand layer and fluvial clay members. Pockets of estuarine peaty clay is occasionally encountered.

The OA deposit with varying degrees of weathering from fresh to destructured, mainly consists of cemented silty sand and sandy silt with SPT-N values generally increases with depth from 5 to 100 blows per 300mm penetration. Fig. 4 shows a typical cross section of the subsurface profile.

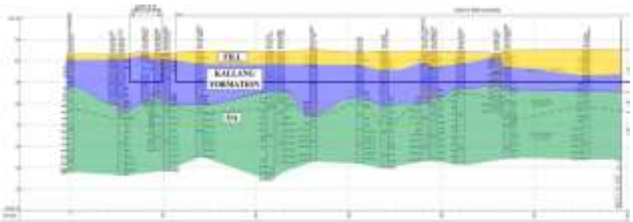

Fig. 4. Typical Subsurface Profile.

3 PILE DESIGN AND CONSIDERATIONS

The axially loaded non-displacement piles are designed based on Eurocode 7 (EC7) and the authorities' requirements in Singapore. The piles are designed with a working load (compression) with a ratio of permanent action (G_k) to variable action (Q_k) in 0.75:0.25.

Should the installation of piles can be carried out as soon as it is practical, the shaft resistance can be easily mobilized with settlement typically 1% pile diameter. However, substantial amount of settlement between 5-10% of the base diameter is required to mobilize the full base resistance. It is an uphill task in ensuring that the base is properly cleaned, free of debris to achieve proper contact to utilize the end bearing.

3.1 Piling Geotechnical Zoning

In the design stage, 15 preliminary subzones are established for the piling design based on the subsurface condition. These are further refined to 36 subzones upon obtaining the additional boreholes information during the construction stage for design optimization. The zoning is presented in Fig. 5.

3.2 Design Approach Based on EC7

Geotechnical Capacity

The geotechnical design is carried out based on Limit State Design approach in accordance to SS EN 1997-1:2010 following the recommended Design Approach 1 (DA1). Partial factors as stipulated in the Singapore National Annex will be applied to the characteristic values for shaft and base resistance for Combination 1 and 2 under DA1.

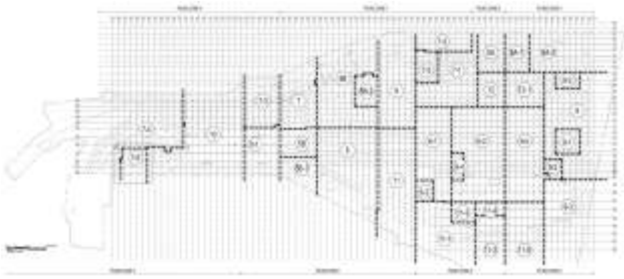

Fig. 5. Subzones for Piling Design.

Ultimate Limit State (ULS) in terms of compressive and tensile resistance and Serviceability Limit State (SLS) in the form of settlement, heave, lateral movement and vibration are the two limit states to be considered for axially loaded piles to ensure failure or severe damage does not occur and the pile displacements are within permissible amount.

Ground displacements like downdrag (or negative skin friction) to be treated as an action shall also be considered. However, it should be noted that with the unloading effect of soil removal from the 15.0m deep excavation, the soil settling more than the pile is unlikely to happen.

The geotechnical capacity of an axially loaded piles in compression can be obtained using the 'Alternative Method' where the design is based on ground test results. It shall also satisfy the equilibrium equation as follows:

$$R_{c;d} = \frac{R_{s;d}}{\gamma_s \gamma_{R;d}} + \frac{R_{b;d}}{\gamma_b \gamma_{R;d}} \quad (1)$$

$$F_{c;d} \leq R_{c;d} \quad (2)$$

where $R_{c;d}$ and $F_{c;d}$ are the ultimate compressive design resistance and design axial compression load respectively, $R_{s;d}$ and $R_{b;d}$ are the shaft and base resistance respectively, γ_s and γ_b are the partial base and shaft resistance factors and $\gamma_{R;d}$ is the model factor specified in the Singapore National Annex SS EN 1997-1:2010.

Structural Capacity

For piles subjected to pure compressive axial load, the average compressible stress across the whole section of the pile shall not exceed 0.3 times the characteristic cylinder strength of concrete at any point along the pile.

3.3 Base Grouting

As mentioned, it must be recognized that thorough cleaning of the pile base especially for piles as large as 2.5m diameter with pile toe as deep as 60.0m below ground level is extremely difficult. Although the issue is more apparent for large diameter piles, base grouting (or post-grouting) is adopted for all the piles to overcome the problem.

The intention is to eliminate two primary construction related mechanisms that affect end bearing resistance, i.e. soil relaxation beneath the pile toe and possible soft toe issue where debris remains after base cleanout. It also aids in mobilizing higher base capacity where can be developed within SLS limit.

Typical base grouting circuits' configuration in bored piles and barrette piles are shown in Fig. 5 where each circuit comprises injection and exit tubes linked by 'tube-a-manchette' across the pile toe.

Base grouting is carried out less than 24 hours after casting and well before the pile concrete gains significant strength. Each circuit is pressurized with water to 'crack' the grout valves and base concrete. After 3 days, cementitious grout is injected under pressure at pile base in a controlled process that compress the pile shaft in an upward fashion. Should the target pressure on the uplift specified is not achieved, the pile shall be re-grouted within 24 hours.

The base grouting effect is controlled using the base uplift criteria where the uplift measured at the pile head shall achieve at least 0.2mm and maximum uplift to be 2.0mm (cumulative of all the grouting stages). The grout pressure is also limited to 30.0bar.

The effect of base grouting is two-fold. It enables consolidating and compaction of sand which is loosened by boring action and pre-compression of shaft. It thus gives a stiff response when the pile is loaded, enhance the load-settlement behavior which can then results in an economical pile design. Sufficient shaft capacity is also required to resist the uplift caused by grouting.

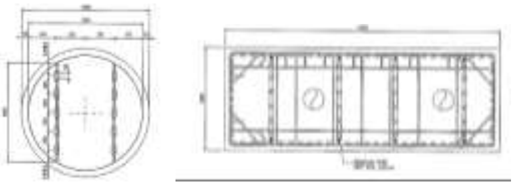

Fig. 5. Base Grouting for Bored and Barrette Piles.

3.4 Pile Design Parameters

For the evaluation of single pile axial load capacity, the empirical correlation of SPT N values to shaft and base resistance is based on the modified Meyerhof (1976) approach.

$$f_s = K_s N \tag{3}$$

$$q_b = K_b(40N) \tag{4}$$

where f_s and q_b are the unit skin friction and unit base resistance respectively, K_i is the value of coef ficient.

Based on published literature and past project experience on similar ground, K_s of 2.0 for clayey OA and 2.5 for sandy OA is adopted. The skin friction is limited to 200.0kPa and 250.0kPa for clayey and sandy OA respectively. K_b of 1.0 and 1.5 is used for cases without and with base grouting.

The design parameters will induce settlement of less than the required 25.0mm under working load conditions.

4 PILE LOAD TESTING

In Singapore, it is a statutory requirement to carry out pile load test. Other than for construction and quality verification, the empirical correlation and design parameters q_b, q_s and pile spring k_s can be substantiated. The uncertainty associated with the load-settlement prediction can also be minimized.

Due to the huge number of piles involved, 1.0% of the total number of piles (i.e. 21 nos. of Ultimate Load Test (ULT) and 6 nos. of Working Load Test (WLT)) are carried out to take advantage of the lower model and partial factors for pile length optimization. 16 out of the 21 nos. of ULT are carried out using Kentledge test, the rest are via bi-directional Osterberg-cell (O-cell) test method. The test piles are 1.0 and 1.2m diameter with working load ranging from 365.0 tons to 967.0 tons.

5 VERIFICATION OF DESIGN PARAMETERS AND FINDINGS

Besides verifying the SLS settlement criteria, combinations of various parameters (higher f_s of 3.0N, 4.0N or debonding and f_b of 70.0N, 90.0N, 100.0N or no base grouting) can be assumed when estimating the pile length to achieve different testing intentions.

As the load tests shows favorable results (elaborated as follows), the subsequent piling design is optimized to f_s of 3.0N and f_b of 70.0N and these parameters are verified in the working load test.

5.1 Findings from Different Test Methods

The findings from ULT are summarized in Table 2 based on the different test methods. It is observed that for O-cell tests, very high friction resistance is mobilized in proximity (1.0m to 2.0m below and above) to jack location; 9.5N to 16.0N and 4.4N to 6.5N below and above jack respectively. When it is well above the jack (>2.0m), the mobilized skin friction resistance is in the range of 2.6N to 5.8N.

The base grouting and the shaft friction for the ULT are plotted in Fig. 6 and 7 with the adopted parameters of 3.0N and 70.0N for f_s and f_b respectively. The end bearing for all the ULT are not mobilized. It ranges from 1.2 to 7.3%D and 0.2 to 5.2%D for Kentledge and O-cell test methods respectively. More than 95% of the mobilized values of ULT fulfils the adopted criteria of 3.0N for f_s.

Table 2. Summary of ULT results.

Test Method	Frictional Resistance (kPa)	End Bearing* (kPa)
Kentledge	2.8N to 5.2N	4673 to 11,739 (62.0N to 100.0N)

O-Cell	2.6N to 15.8N†	4896 to 9051 (61.0N to 90.0N)

* not mobilized
† below the jack; due to base grouting action

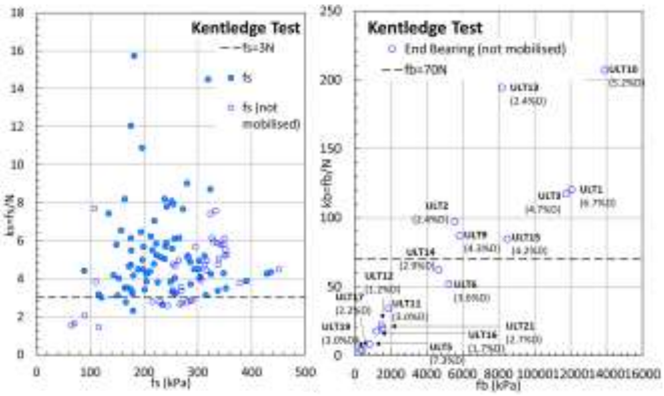

Fig. 6. Shaft Friction and End Bearing (Kentledge Test).

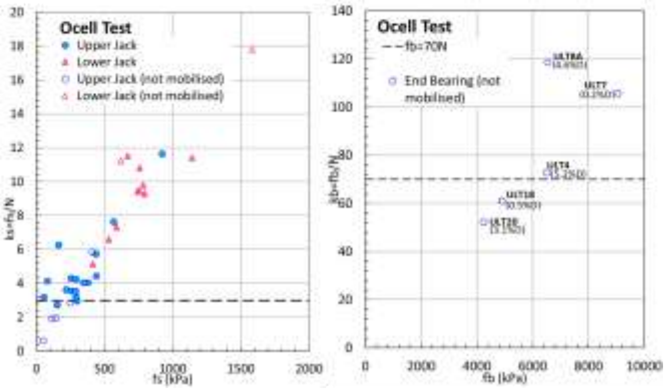

Fig. 7. Shaft Friction and End Bearing (O-cell Test).

5.2 Effect of Base Grouting

It is speculated that the high skin resistance recorded below the jack is likely due to the base grouting. The base grouting could cause a reverse preloading effect and some escape of cement grout to the shaft area might be possible. It might also increase the confining pressure which develops from the bearing shear failure mechanism where the major principal stress shifted to horizontal, illustrated in Fig. 10.

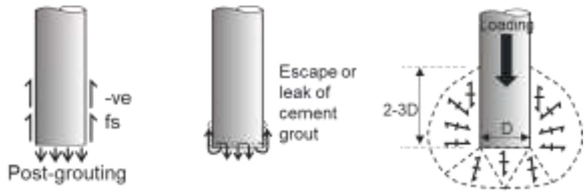

Fig. 10. Shaft Friction and End Bearing (Kentledge Test).

5.3 Verification of Design Parameters

Based on client's requirements, failure is dictated by a maximum settlement of 7.0mm and 14.0mm for 1.0WL and 1.5WL respectively. From the 6 nos of WLT results, the optimized parameters of 3.0N and 70.0N for f_s and f_b evaluated from the ULT satisfies the SLS criterion as shown in Table 3.

Table 3. Summary of WLT results.

Test Pile No	Pile Dia./Size (mm)		Settlement (mm) 1.0WL	1.5WL
WLT-LC31a	1200.0	Pile Head	5.5	7.0
		Residual	1.0	2.0
WLT-T33	1500.0x3000.0	Pile Head	10.3	11.8
		Residual	2.0	2.5
WLT-J18	2500.0	Pile Head	6.3	8.5
		Residual	1.0	1.3
WLT-M19	2200.0	Pile Head	7.3	9.5
		Residual	0.5	0.5
WLT-PEEa84	1800.0	Pile Head	5.3	8.6
		Residual	0.0	0.8
WLT-RB-58	2000.0	Pile Head	5.5	6.8
		Residual	1.5	1.5

6 CONCLUSION

Having sufficient SI information are of utmost importance in the preliminary stage to develop the soil models for piling design. Appropriate testing through instrumented pile load tests are essential particularly for large projects to ensure that the design parameters and settlement are achievable and performed as per expectations prior to actual piling works. Due to the large project site, the pile load tests and its results provides an avenue to optimize the pile design parameters. The huge number of piles involved made the optimization desirable and the savings significant. Base grouting especially dealing with large diameter piles can be beneficial in the assurance that the base is properly founded on good ground.

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REFERENCES

Chang, M.F., and Teo, P. (2004). Load Transfer Characteristics of Bored Piles in Singapore's Old Alluvium. Proc. 15th South East Asian Geotechnical Society Conference, Bangkok, 211-216.

Chin, Y.K., Tan, S.L. and Tan, S.B. (1985). Ultimate Load Tests on Instrumented Bored Piles in Singapore Old Alluvium. Proc. 8th South East Asian Geotechnical Society Conference, Kuala Lumpur, 2-54 – 2-65.

Ho, C.H., and Lim, C.H. (1994). Bearing Capacity and Settlement of Slurry Bored Piles in Singapore Old Alluvium. Proc. of the 3rd International Conf. on Deep Foundation Practice, Singapore, 125-132.

Ong, C.W., Chen, D.C. and Prebaharan, N. (1999). A Summary of Preliminary Pile Load Test Results for North East Line. Proc. of the Rapid Transit Conference, Singapore, 687-702.

Li, W.W., and Wong, K.S. (2001). Geotechnical Properties of Old Alluvium in Singapore. Journal of the Institution of Engineers, Singapore, 41:10-20.