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Port of Brisbane – Berth 11 Pile Load Tension Test

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ABSTRACT

This paper discusses the behaviour of a tension test pile observed during the full scale static pile load tension test at the Port of Brisbane. During the loading, the pile displacement and extension were measured using displacement transducers to measure the relative movement between top of the pile and three pairs of unstrained telltale rods fixed inside the pile shaft at various depths. Measurement results indicated that the pile skin friction acting in upper part of the test pile already mobilised at an applied tension load of 2,500kN while the lower Pleistocene layers located in the middle – bottom zones of the pile shaft were contributing the major component of tensile resistance. In addition, although the test was designed for the test pile to reach failure at a design ultimate geotechnical capacity in tension of R_{ug} = 5,000kN (with 50% redundancy), the test configuration was not able to measure the ultimate pile tension capacity within the capacity of the loading system. The ultimate geotechnical strength (R_{ug}) of the test pile in tension was subsequently extrapolated using Mazurkiewicz's method deemed as the most appropriate based on the test pile data. The most plausible reason for the fact that the tension test pile did not reach failure is considered to be related to the coefficient α (adhesion reduction factor) of 0.5 used to calculate the design pile skin friction for estimating the design ultimate geotechnical strength compared to an α value of 0.93 derived from back-analysis.

Keywords: static pile load tension test, pile extension, ultimate geotechnical strength

1 INTRODUCTION

During the construction of Wharf 10 at the Port of Brisbane, the Pile Driving Analyser (PDA) test results indicated that the capacity of some piles in the downstream section of Berth 10 was significantly less than the upstream piles and did not achieve the required design capacity in tension.

In order to determine the tension capacity of wharf raker piles under uplift and verify design parameters for detailed design and construction for future wharfs, a pile load tension test was conducted onshore at the upstream end of Berth 11 immediately behind the existing berth rockwall and proposed Wharf 11 alignment.

The site geology at the Port of Brisbane comprises loose sand and soft marine clay Holocene deposits overlying Pleistocene deposits comprising intercalated medium dense sands and stiff clays, overlying Basalt bedrock. The wharf piles principally derive skin friction and hence tension capacity from the Pleistocene deposits.

2 STATIC PILE LOAD TEST PROGRAM

2.1 Pile load test configuration

Figure 1 shows the pile load tension test configuration and layout. The tension test pile was a steel tubular pile 1200mm outer diameter × 16mm wall thickness (Grade 250) including a driving shoe (0.5m long × 32mm) set flush with the outside diameter of the pile. The reaction system consisted of two steel tubular piles 900mm diameter × 12mm thickness acting in compression. The centre to centre spacing between the reaction piles and the test pile was set at 6.0m.

The design founding levels of both the test pile sections (in tension) and the reaction pile sections (in compression) were determined based on static pile capacity analysis using the software package APile Plus 4.0, and adopting the API RP2A method as applied to driven, steel tubular piles. For the purposes of pile capacity assessment for the test pile, a geotechnical design ultimate pile capacity in tension (R_{ug}) of 5,000kN was adopted. For design of the reaction piles, a geotechnical strength reduction factor of $\varnothing_g$ = 0.40 was applied in assessing the corresponding design geotechnical strength (R^*_g) applied for each reaction piles to share support of the design ultimate load for the test pile.

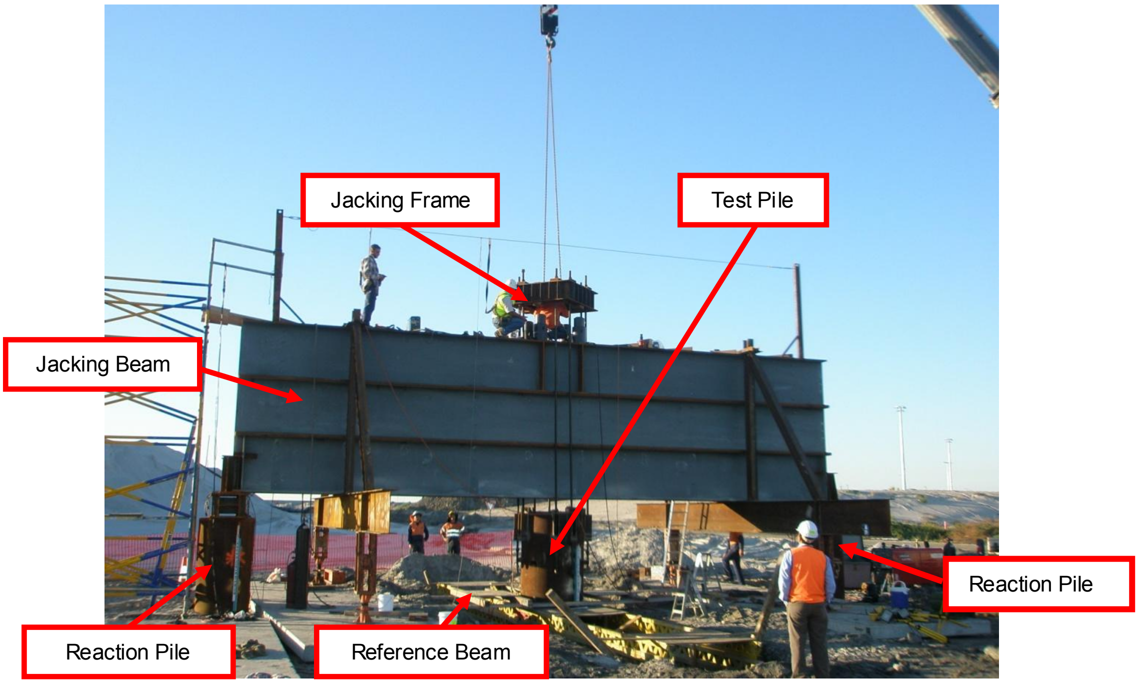

Figure 1. Overview of pile load tension test

2.2 Loading procedure

Tension loads on piles are often intermittent rather than permanent which is anticipated to be the case for the wharf piles (eg. crane and mooring loads). A loading schedule was consequently developed considering the design ultimate geotechnical capacity required for the raker piles and also to reflect the intermittent loading condition of the piles in service. A modified incremental sustained load test schedule which included cycles of loading and unloading up to the approximate design working load was adopted. During each load interval a constant load was maintained for the specified time interval.

2.3 Load measurements

A total of four 300 tonne load cells (total capacity: 4 x 300 = 1,200 tonnes) were used as the primary system to measure load applied during the pile load test. The load cells were placed in series each with a hydraulic jack. The calibrated jack system was also utilised as a secondary system to measure load during the pile load test.

2.4 Displacement measurements

Four displacement transducers were used as the primary system to obtain a measurement of axial movement at the top of the test pile. The transducers were mounted on the reference beam to bear on the pile top at axisymmetric points equidistant from the centre of the test pile, with stems parallel to the longitudinal axis of the pile.

Displacement readings were also obtained by precise levelling using a laser level and were taken from a scale mounted on the test pile, each end of the reference frame, and the reaction piles before and throughout the duration of the pile load tension test. Levels were also referenced to a permanent bench mark located away from the immediate test area. The precise levelling was used as a secondary system for displacement measurement.

2.5 Pile extension measurements

The displacement or extension of the test pile was measured during loading at specified levels down the pile to assist in the evaluation of the distribution of load transfer from the pile to the surrounding soil down the length of the test pile.

The pile displacement was measured using displacement transducers to measure the relative movement between top of the pile and unstrained telltale rods fixed to the pile shaft at various depths. The paired telltales were established within the test pile (in steel tubes welded to the inside of the pile sections) with the rods in each pair oriented diametrically opposite each other and equidistant from and parallel to the pile long axis. Three pairs of telltales were installed at specified depths of the test pile shaft to measure relative displacements between the following horizons and the pile top:

Figure 2. Telltale casings installed on test pile

- Near the pile toe
- Near the middle of Pleistocene layer
- Near the top of Pleistocene layer

The Pleistocene layer was targeted considering that the tension test pile (and the wharf raker piles) predominantly derive skin friction and hence tension (uplift) capacity from the Pleistocene horizon.

2.6 Subsurface profile condition at test pile location

A geotechnical investigation borehole drilled at the test pile location (prior to the pile load tension test) revealed that the subsurface profile approximately consisted of 5m of sand fill, overlying 8m of Holocene deposits comprising 3m of loose silty sand and 5m of soft marine clay, overlying the Pleistocene deposits comprising approximately 12m of medium dense silty sand, 5.5m of very stiff clay and 3m of medium dense sand to the base level of the test pile.

Atterberg limits, PSD and UU triaxial tests were performed on the samples recovered from the borehole. The UU triaxial test results indicated that Su value of Pleistocene clay is approximately 120kPa.

3 PILE LOAD TENSION TEST RESULTS

The pile load tension test results comprising plots of:

- tension load versus upward displacement of pile head, and
- tension load versus telltale deflection relative to pile head.

are presented in Figures 3 and 4, respectively. Data shown on these figures were obtained from the primary monitoring system (load cells and displacement transducers). In these figures, “load’ is the sum of the four load cell measurements, “uplift of pile head” is the upward displacement value relative to the deflection of the reference beam, and “telltale deflection” is the average displacement of the pairs of telltales (of equal length) relative to the pile head.

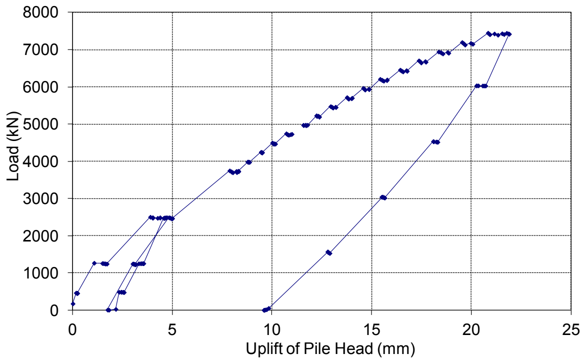

Figure 3. Load versus uplift of pile head

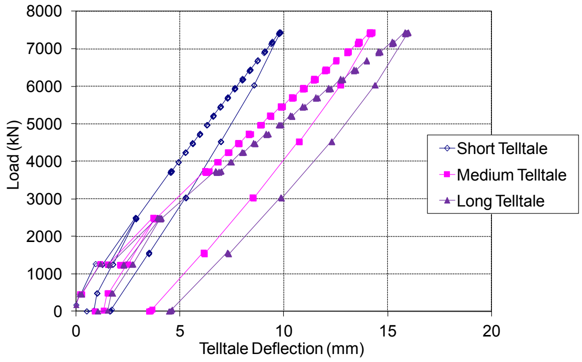

Figure 4. Load versus telltale deflection

4 ULTIMATE GEOTECHNICAL CAPCITY OF TEST PILE IN TENSION

The pile load tension test was designed to achieve geotechnical failure of the test pile at a design ultimate geotechnical strength in tension of R_{ug} = 5,000kN (with 50% redundancy). However the maximum tension load applied during the load test (7,500kN) did not take the test pile to its ultimate geotechnical capacity, as defined by the following failure criteria:

- the load causing movement of 10% of the pile diameter (120mm), or
- the load at which the test pile load cannot be maintained as pile deflection increases.

The ultimate geotechnical strength of the test pile in tension has subsequently been estimated based on the pile load test data by considering various conventional techniques and also PDA test results from re-strike testing.

Based on the pile load tension test data, Mazurkiewicz's method appears to be the most applicable technique for estimating the ultimate capacity of the test pile in tension; and yielded an R_{ug} value of 9,000kN for the test pile. Figure 5 presents a plot of measured load displacement and the plotted line used to assess the ultimate load of the pile in accordance with Mazurkiewicz's method, which consists of the following steps. A series of equal pile head movements is chosen and vertical lines that intersect on the load vs displacement plot. A horizontal line from these intersection points is then drawn on curve to intersect the load axis. From the intersection of each load a 45° line is drawn to intersect with the next load line. These intersections typically fall on a straight line and the point which is obtained by intersection of the extension of the line on the vertical axis is the predicted failure load.

The most plausible reason for the fact that the test pile did not reach geotechnical failure may be explained by considering the coefficient α (adhesion reduction factor) that was used to calculate the design pile skin friction for estimating the design ultimate geotechnical strength (R_{ug}) in tension:

Skin friction in cohesive soil $f_s = \alpha \times s_u$ (1)

where, s_u = undrained shear strength of clay.

An α value of 0.5 for s_u = 120kPa clay was adopted for the design of the tension test pile as per standard geotechnical practice and the program manual note of the software package APile Plus 4.0 that was used to calculate the ultimate geotechnical strength of the test pile; assuming that tension capacity is 80% of compression capacity for both clay and sand. However a subsequent literature review indicates that the value of α for piles in stiff clays can be highly variable (Vesic, 1976) and our back-analysis based on an ultimate geotechnical strength in tension of 9,000kN indicates an α value as high as 0.93.

Figure 6 presents the ultimate skin friction capacity versus pile penetration. In this figure, the mass of the soil plug (measured length of 28.0m) and a suction of 100 kPa (which may be generated at the base of the pile) were considered. Note that no contribution of the soil plug to the skin friction was taken into account in this plot due to the smaller internal diameter of the driving shoe compared to the pile, and because of the fact that the plug is likely to disconnect from the foundation soil at the base of the pile under tensile loading.

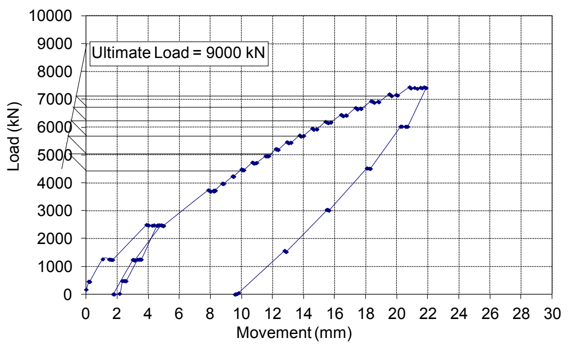

Figure 5. Prediction of ultimate geotechnical strength in tension (Mazurkiewicz's method)

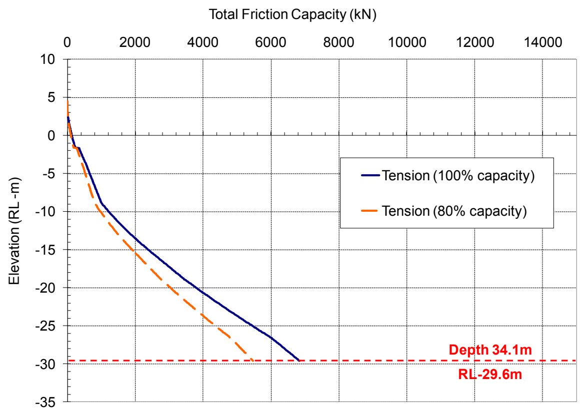

Figure 6. Test pile (with mass for soil plug) total friction capacity

5 PILE EXTENSION AND LOAD DISTRIBUTION

Based on the results shown in Figures 3 and 4, displacements of the test pile at four different elevations (displacement transducer at RL 5.0m and three telltales at RL-11.83m, RL-20.28m and RL-27.01m) are presented in Figure 7 as plots of displacement for various applied pile head loads. It can be seen that the displacement generally becomes smaller with depth irrespective of the load. In addition permanent uplift was observed at the base of the pile after each loading – unloading cycle. Uplift was measured at 0.74mm after the 1st cycle (working load) and 5.14mm after the 2nd cycle (maximum load), respectively.

Figure 8 presents a plot of the calculated strains in the test pile between the various telltale levels. In this figure, the test pile shaft has been divided into three zones according to the depth of the telltales installed. Table 1 presents a summary of these zones, depths and levels.

Average movement = (telltale reading at top of zone and bottom of zone) / thickness of zone) (2)

Table 1: Classification of zones based on telltale installation

	Depth range (mPD)	Mid depth (mPD)	Remarks
Top zone	RL5.97 – RL-11.83 (L=17.80m)	RL-2.93	• Top of Pile – Telltale 1 • Fill and Holocene layers
Middle zone	RL-11.83 – RL-20.28 (L=8.45m)	RL-16.06	• Telltale 1 – Telltale 2 • Near the top of Pleistocene layer – near the middle of Pleistocene layer
Bottom zone	RL-20.28 – RL-27.01 (L=6.73m)	RL-23.64	• Telltale 2 – Telltale 3 • Near the middle of Pleistocene layer – near the pile toe

The data plotted in Figure 8 show that the slopes of data points between those for the middle zone and the top zone are approximately consistent irrespective of the applied tensile load. This implies that the pile skin friction acting in upper part of the test pile installed in fill and Holocene material was nearly or already mobilised at an applied tension load of 2,500kN. The lower Pleistocene layers located in the middle – bottom zones of the pile shaft were contributing the major component of tensile resistance and were increasing in strength and not fully mobilised.

This observation also confirms the pile capacity analysis results which indicate that a 22.3 m penetration (a pile toe level of RL-17.8 m) is required to develop a design ultimate geotechnical strength of 2,500kN assuming that the tension capacity is 80% of the corresponding compression capacity for the pile as shown by the data presented by Figure 5.

In addition, the middle zone of the pile shaft had the largest displacement after each loading cycle (Figure 8). This suggests that some tension load remained locked into this zone after unloading.

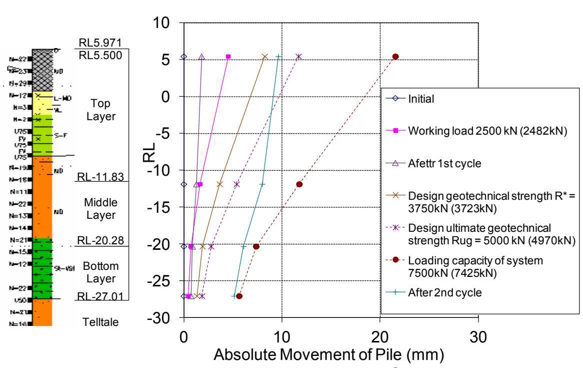

Figure 7. Absolute movement of pile under various load conditions

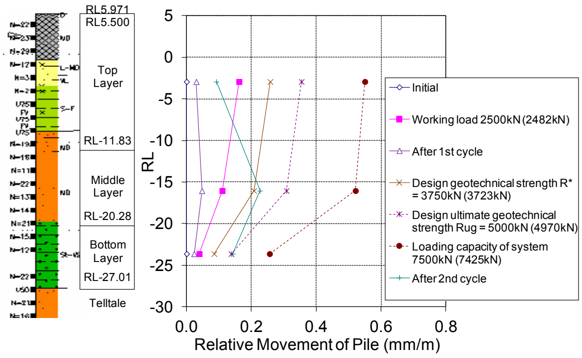

Figure 8. Average movement of pile under various load conditions

6 CONCLUSIONS

In this study, the behaviour of a tubular steel pile observed during a full scale static pile load tension test was examined. In addition, the ultimate geotechnical strength of the test pile in tension of has subsequently been extrapolated considering the tension test load of 7,500kN exceeded the design ultimate capacity by 50% without failure of the pile. From review the following conclusions are drawn:

- Measurement of the pile displacement and extension indicated that the pile skin friction acting in upper part of the test pile already mobilised at an applied tension load of 2,500kN while the lower Pleistocene layers located in the middle – bottom zones of the pile shaft were contributing the major component of tensile resistance.
- The middle zone of the pile shaft had the largest displacement after each loading cycle which suggests some tension load remained locked in this zone after unloading.
- The most plausible reason for the fact that the test pile did not reach geotechnical failure during the pile load tension test may be explained by considering the coefficient α (adhesion reduction factor). The results of back-analysis indicate that the α value could be as high as 0.93 compared to an α value of 0.5 used for design.

7 ACKNOWLEDGEMENTS

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