

Verification of pile design with bi-directional load testing

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ABSTRACT

A large-scale pile testing programme, in-part comprising nineteen bi-directional load tests and forty-seven thermal integrity profile tests, was undertaken on 0.9m and 1.5m diameter bored piles up to 35m in length for the Victoria Park to Canning Level Crossing Removal Project, Western Australia. The bi-directional load tests were undertaken using *Ougan Supercells*, an assembly of cone-shaped hydraulic jacks, placed within each test pile at a level that would generate near equal and opposite upward and downward directed forces, facilitating shaft friction and end bearing measurements. The supercells were typically located within the bottom third of the test piles allowing pile behaviour at depth and at the pile base to be investigated. This paper discusses the interpretation of bi-directional load test data, pile-displacement behaviour and pile load distribution obtained in the tests carried out. Full mobilisation of pile capacity was observed in the lower test segment of a number of tests. Large displacement of the lower test segment allowed ultimate pile shaft friction and end bearing to be measured in very low strength Osborne Formation Siltstone and Shale. The significant displacement of the lower test segments was attributed to pile base disturbance during construction even though thorough base cleaning of the piles was undertaken. Pile constructability factors resulted in lower anticipated pile shaft friction as well as end bearing. Comparison is made between measured and estimated ultimate values from conventional rock uniaxial compressive strength-based methods for the Osborne Formation, and from cone penetration test-based method for the superficial formation.

Keywords: Bi-directional load tests, Supercells, Osborne Formation

1 INTRODUCTION

The Victoria Park to Canning Level Crossing Removal Project (LXR) forms part of the wider METRONET program, which collectively comprises 70km of new heavy passenger rail and sixteen new rail stations across the Perth metropolitan area. The LXR involves all design and construction of the new Armadale Line, including commissioning, interconnection with the existing passenger rail line network and final asset acceptance of those works.

The project involves the removal of six level crossings on the inner section of the Armadale Line from approximately Miller Street in East Victoria Park to William Street in Beckenham. The level crossings will be replaced by 4.6km of elevated viaduct. The viaduct carrying the rail over Mint Street and Oat Street is referred to as Viaduct 1, and the viaduct over Welshpool Road and supporting the proposed new Welshpool Road is referred to as the Viaduct 2. Viaduct 3 carries the railway over the Wharf Street Group crossings and Viaduct 4 carries the railway and Beckenham Station over William Street.

Viaducts 1 and 2 are supported by 1.05m and 1.2m diameter continuous flight auger (CFA) piles up to 25m in length; and Viaducts 3 and 4 by 0.9m and 1.5m diameter bored piles up to 35m in length. The abutments on all viaducts are supported by bored piles.

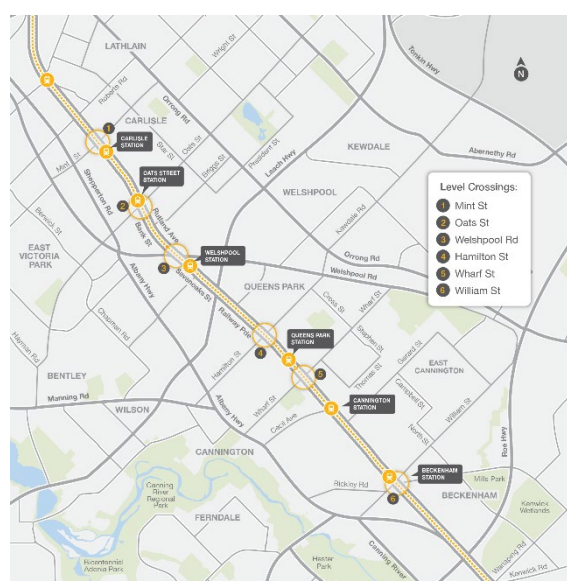


Figure 1. Regional Geology of North-West portion of alignment (Gozzard, 1986)

As part of the wider pile testing programme, pile tests comprising nineteen bi-directional load tests of bored piles and fourteen top-down static load tests of CFA piles were undertaken. All load tests were carried out on sacrificial piles. The pile locations were selected to be representative of the ground conditions applicable to the production piles, out of the way of the production piles and so as to not present an obstruction for future construction works.

This paper discusses the bi-directional load tests (BDLTs) undertaken in the project. BDLT comprises an assembly of hydraulic jacks (*Ougan Supercells*),

embedded within each test pile, that forces the pile apart for shaft friction and end bearing measurements. The objectives of BDLTs are to investigate pile behaviour at depth, quantify constructability factors of piles in low strength Osborne Formation siltstone, and to verify pile design; with the use of supercells located within the bottom third of the piles.

2 SUBSURFACE CONDITIONS

The overall elevation of the ground surface along the LXR alignment gradually decreases along the alignment from north-west to south-east from around 15m AHD to 7m AHD.

The geology along the LXR alignment is provided in two Geological Survey of Western Australia (GSWA) 1:50,000 Environmental Geology Series maps, including:

- “Perth” (Gozzard, 1986a) - Sheet 2034 II and Part of 2034 III 2134 III (Figure 1); and
- “Armadale” (Gozzard, 1986b) - Part Sheets 2033 I and 2133 IV (Figure 2).

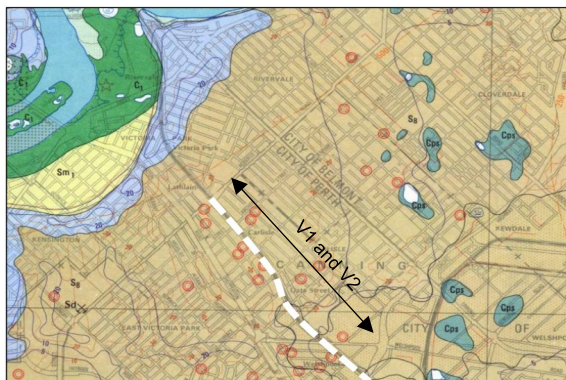


Figure 2a. Regional Geology of North-West portion of alignment (Gozzard, 1986)

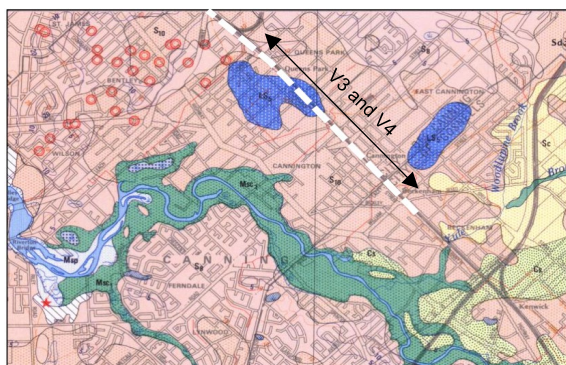


Figure 2b. Regional Geology of South-East portion of alignment (Gozzard, 1986)

In the subsurface of Viaducts 1, 2 and northeastern part of Viaduct 3, the superficial formation predominately comprises medium dense to dense silica Bassendean Sand overlying dense to very dense Gngangara Sand with infrequent cohesive layers. Underlying Gngangara Sand is the Ascot Formation that is variably cemented and ranges from loose sand to medium strength silicious calcarenite with thinly interbedded sand

horizons, containing shell fragments, glauconite and phosphatic nodules near the base.

The superficial formation of the southeastern part of Viaduct 3 and Viaduct 4 mainly consists of Guildford Formation (Q_{pa}) medium dense to dense clayey sand and stiff to hard sandy clay, overlying a variable thin layer (< 5m) of Gngangara Sand. In a broader sense, and for the purposes of the current project, the Guildford Formation can be considered a mostly cohesive formation with less frequent pure granular layers.

The basement formation underlying the superficial formation described above is very low to low rock strength, laminated, glauconitic Osborne Formation Siltstone and Shale with interbeds of mostly fine-grained sandstone at Viaducts 3 and 4. The elevation of the top of Osborne Formation along the alignment typically ranges between RL-15m AHD at the northwestern part to RL-10m AHD at the southwestern part. Note: one of the BDLT piles is located at the southern abutment of Viaduct 2 where the basement formation is Kings Park Formation described as calcareous, glauconitic siltstone and shale of shallow marine to estuarine origin. The Kings Park and Osborne formations have been differentiated in the project area by the presence or absence of marine fauna, respectively, as well as by differences in grain size distribution, colour and logged or measured strength/ consistency.

Detailed geotechnical investigations were carried out in several phases throughout the project to provide a level of geotechnical information suitable to enable geotechnical and structural designs to be completed. The site investigations were undertaken in accordance with the Public Transport Authority (PTA) standards and included boreholes and/ or CPTs at each pier location, extending to depths beneath the proposed pile foundation toe levels. Other investigation techniques utilised included seismic CPT, dilatometer, and down-hole pressuremeter, test pits, hand augers, and monitoring bore installation.

An extensive laboratory testing program was undertaken that included standard classification tests, UCS, triaxial and consolidation testing.

Geological conditions along BDLT test alignment have been separated into fifteen ground models (GMs) based on ground variability. The characteristic design parameters of each GM were established using the statistical methods described in A.J. Bond 2011. For the cone penetration tests (CPTs) in the superficial deposits, the 95% confidence cone tip resistance (q_c), skin friction (f_s) and pore pressure (u_s) values were evaluated. Similarly, for Osborne Formation, the 95% confidence uniaxial compressive strength (UCS) values were determined. These values were used as the basis for the correlation of design parameters.

The 95% confidence q_c profiles of the superficial formation for ground models relevant to BDLTs are shown in Figure 3. The q_c profiles indicate that the superficial formation is variable in the horizontal and vertical extents as the strata transitioned.

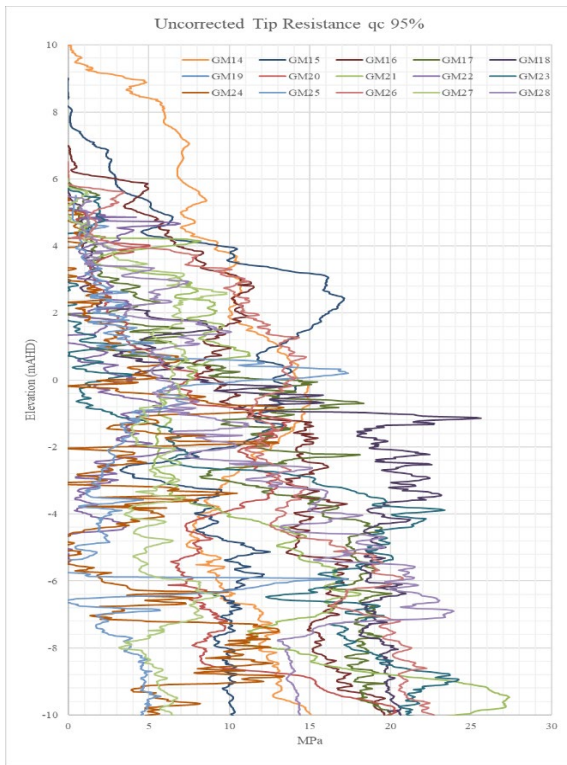


Figure 3. 95% confidence q_c profiles of superficial formation for ground models along BDLT alignment

Figure 4 shows that the Osborne Formation is a very low-strength siltstone (95% confidence UCS of <0.7MPa). The strength increases marginally with decrease in elevation to -3 5mAHD, noting that the bored piles were extended to between RL-20m and RL-33mAHD. Note: the 95% confidence UCS of Kings Park Formation is 3.5MPa.

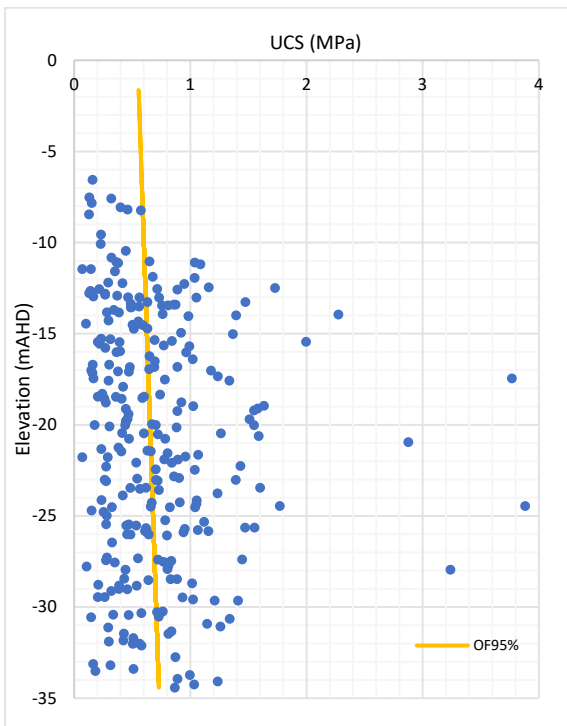


Figure 4. UCS profile of Osborne Formation along BDLT alignment

The groundwater level in the area has been assessed through hydrogeological modelling by the project team and generally ranges from approximately 12mAHD at the northern end of the alignment to 5mAHD at the southern end. For pile design purposes, the groundwater was generally considered to be the ground level.

3 PILE DESIGN APPROACH AND METHODOLOGY

Piled foundations of LXR were designed in accordance with AS 2159:2009 where the design geotechnical strength is shown by equation (1).

$$R_{dg} = \phi_g \times R_{d,ug} \geq E_d \tag{1}$$

R_{dg} : design geotechnical strength of pile.
 $R_{d,ug}$: ultimate geotechnical strength of pile.
 ϕ_g : geotechnical strength reduction factor, adopted as 0.74 given the extent of site investigation and pile testing undertaken.
 E_d : design action effect

3.1 Design in Superficial Formation

Shaft friction (f_s) for the superficial formations was assessed using the CPT-based *Laboratoire Central des Ponts et Chaussées* (i.e. LCPC) method (Bustamante and Ghaneselli 1982) modified by Lehane and 95% confidence q_c profiles of each GM. In this method, the f_s is obtained from correlations with the CPT data locally collected in Perth as shown in equation (2). Additionally, f_s is limited to a maximum of 100kPa for the bored pile design (refer to Poulos & Ameratunga 2022).

$$f_s = \frac{q_c}{(175 \pm 30)} \tag{2}$$

3.2 Design in Osborne Formation

Pile capacity in the Osborne Formation comprising shaft friction (f_s) and end bearing (q_b) was assessed based on the relationship proposed by Zhang and Einstein (1998) for low strength, very weak rock as shown by equations (3) and (4).

$$f_s = a_s q_u^{b_s} \tag{3}$$

q_u : Rock Uniaxial Compressive Strength (MPa)
 a_s : 0.2 to 0.3
 b_s : 0.5

The use of $a_s=0.2$ is considered conservative, as data presented by other authors can range up to 0.6.

$$q_b = a_b q_u^{b_b} \tag{4}$$

q_b : ultimate end bearing pressure
 q_u = Rock Uniaxial Compressive Strength (MPa)
 a_b = 4.8
 b_b = 0.5

The use of $b_b=0.5$ is considered conservative, as presented in Zhang and Einstein (1998).

4 PILE CONSTRUCTION

All bored test and production piles were constructed using the conventional bored pile method comprising auger drilling through temporary casing to around 10 m depth followed by bucket drilling supported by polymer to the design toe depth. On reaching the design depth in the Osborne Formation (or Kings Park Formation), the pile toe was cleaned using a cleaning bucket and the pile toe checked using a weighted tape. The pile depth was recorded at time of cleaning, then after reinforcement cage installation and just prior to concreting. If excessive sediment was present, the cage was removed and the pile base recleaned. The polymer was also monitored for viscosity and sand content throughout the construction process.

In general, sand content in the polymer was maintained below 2% and sediment build-up at the pile base was minimal. It was noted by the site engineers during construction that the pile bases within the Kings Park Formation felt noticeably “harder” through the weighted tape and the impact of the weight on the pile base was more noticeable than in Osborne Formation. This is likely attributed to the higher strength rock.

Concrete was placed in the pile hole using a tremie pipe, ensuring the tremie was always below top of concrete level to ensure concrete column integrity.

Construction of the test and production piles was carried out under the supervision of a suitably experienced geotechnical engineer, who was independent of the piling contractor. The geotechnical engineer monitored the construction process to ensure the agreed quality standards and construction methods were adhered to. The engineer also logged the materials excavated from the boreholes to ensure the ground conditions encountered were as expected from the geotechnical investigations.

5 PILE TESTING PROGRAMME

5.1 Bi-directional Static Load Testing

BDLT was carried out at a frequency of 4% of all bored piles installed at Viaducts 3, 4; and at all abutment locations of Viaducts 1-4. This amounted to 19 no. tests. Each of the tests were identified based on the Viaduct number followed by the Pier number. For example, V3-P27 test is located at Viaduct 3 and near Pier 27.

Further discussion of BDLT is presented in Section 6.

5.2 Integrity Testing

Pile integrity of both the sacrificial test and production piles was monitored using Thermal Integrity Profiling (TIP) which utilises the heat generated through the curing of the concrete to evaluate the pile shaft profile and concrete cover. Temperature sensors are placed in groups of 4 at 300mm depth intervals around the reinforcement cage with readings at each level averaged together, and along with the concrete volume records, used to assess the overall shape of

the pile. Adjustments were made to the profile to account for external influences such as the presence of steel casing, groundwater, and the load test cells.

TIP testing is becoming a preferred pile integrity test method as a profile may be developed along the full pile length. On the LXR project, TIP testing was carried out at a frequency of 10% of all the bored piles installed (i.e. 47 no. tests carried out) and was conducted in lieu of other methods, including low strain testing, coring, and sonic logging.

TIP testing results on test piles tested indicated a reduction in pile diameter at toe level in tests V3-P27 and V4-P13, and ‘necking’ where supercells were located. Interpreted pile radius and concrete cover profiles of tests V3-P27 and V4-P13 are presented in Figures 5 and 6. Pile radius reduction in the profiles is further discussed in Section 7.3. No significant defects were recorded in any of the piles tested.

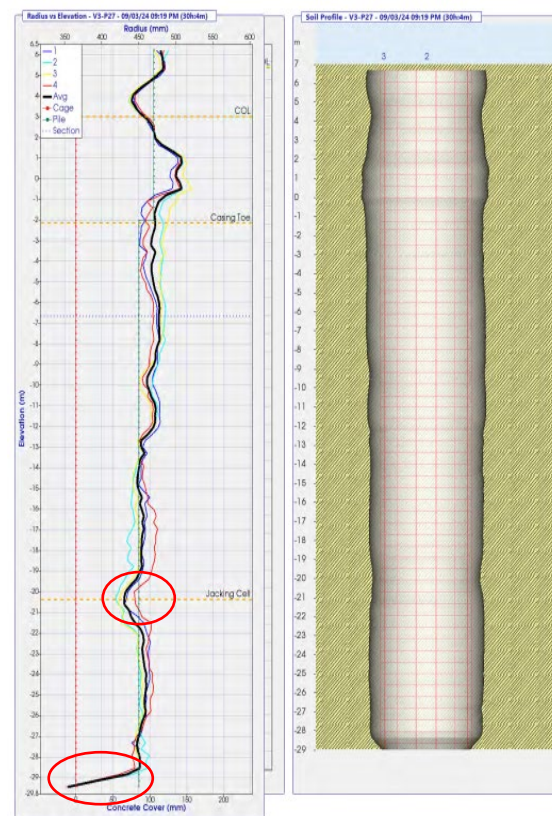


Figure 5. Pile radius and concrete cover profiles (test pile V3-P27)

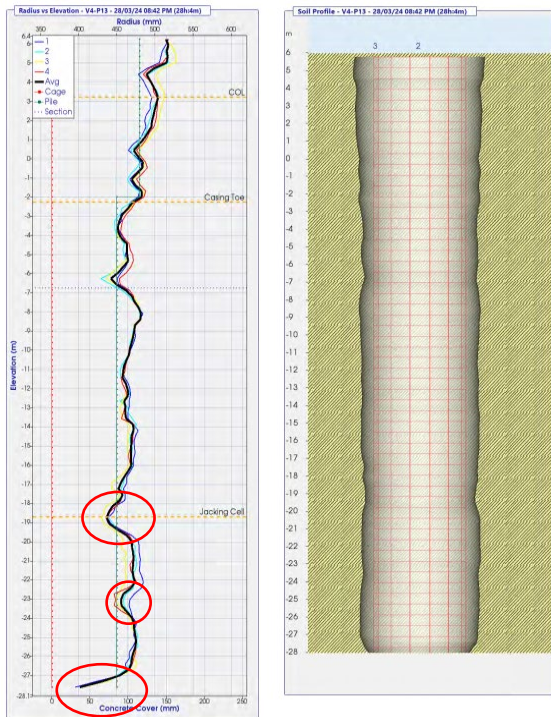


Figure 6. Pile radius and concrete cover profiles (test pile V4-P13)

6 BI-DIRECTIONAL LOAD TESTING

BDLT is a high-capacity static load test that allows investigation and quantification of constructability factors of piles at depth, by applying the test load within the bottom third of the pile or near to the pile base.

Given that the majority of the piles were 24m to 35m in length and are founded in the Osborne Formation, a stratum characterised by low strength rock and some variability, a relatively large number of BDLTs (i.e., 4% of the bored piles installed) were undertaken for the design verification of ϕ_g , f_s and q_b adopting the equations (1) to (4) for bored piles design. Note: Test V2-A2 is located close to the abutment of Viaduct 2 and was founded in Kings Park Formation.

BDLT was the preferred test option, as dynamic testing of the piles was not possible on this site due to the proximity of sensitive utilities along the entirety of the alignment. Further advantages of BDLT are discussed in Section 8.

6.1 Test Load

Two test types were considered in BDLTs based on the magnitude of test load:

- Type A - Seven of the test piles were tested to the design ultimate geotechnical strength of the piles ($R_{d,ug}$). Test piles were loaded at 10% serviceability load (P_s) increment intervals to 100% P_s and held for a minimum of 4 hours, followed by 20% P_s increment intervals to a maximum test load of 100% $R_{d,ug}$ for a 1-hour hold before unloading.

- Type B - Twelve piles to 1.2 times the serviceability load (P_s). Test piles were loaded at 10% P_s increment intervals to maximum test load.

Note: Unload-reloading was avoided to allow for more accurate pile stiffness assessment, as discussed in Section 7.3.

6.2 Test Arrangement and Instrumentation

The BDLTs were undertaken using *Ougan Supercells*, an assembly of hydraulic jacks, arranged in a ring shape as shown in Figure 7. The supercells were manufactured in different shapes (i.e. cylindrical shape for the 0.9m diameter piles and cone shape for the 1.5m diameter piles) for ease of concrete flow between the upper and lower test segments and to allow debris from the pile base to be flushed out through pile top during concreting.



Figure 7. Photo of supercell arrangements for BDLT

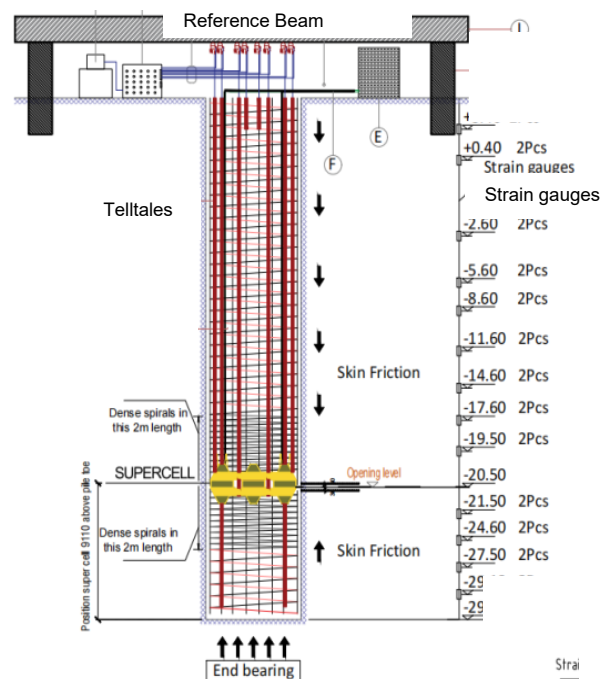


Figure 8. Instrumentation of BDLT Pile

The ring of cells was placed at a level that was assessed to generate near equal and opposite upward and downward directed forces for pile f_s and q_b measurements. The supercells were located within the bottom third of the test piles, apart from test V2-A2 where the cells were located at the pile base as the calculated end bearing in founding unit (Kings Park Formation) nearly equals the shaft resistance from the pile above.

Telltails in pairs were placed at four levels in all test piles: at the top and base of test piles, and at the top and base of supercells for pile displacement measurements at different levels.

The load distribution along seven Type A test piles was measured with strain gauges, placed in pairs at approximately 3m intervals or where there is a change in strata. A schematic drawing of the instrumentation is presented in Figure 8.

7 BI-DIRECTIONAL LOAD TEST RESULTS

Eighteen of the nineteen 0.9m and 1.5m diameter bored piles, 24m to 35m long were installed through the superficial formation (Bassendean Sand, Guildford Formation, Gnangara Sand, Ascot Formation) and very low strength Osborne Formation rock (UCS of around 0.6MPa). Test pile V2-A2 was the only test pile founded on Kings Park Formation rock that is of higher strength than Osborne Formation (UCS of around 3.5kPa).

7.1 Measured Load-Displacement

The load-displacement measurements of the Type A and Type B BDLTs are presented in Figures 9 and 10, respectively. Reference to both figures shows that the lower test segments in Osborne Formation displaced large movements (up to 160mm), whilst the associated upper test segments displaced no more than 10mm at maximum test load. Upper and lower test segments of pile V2-A2 in Kings Park Formation displaced around 20mm in both directions.

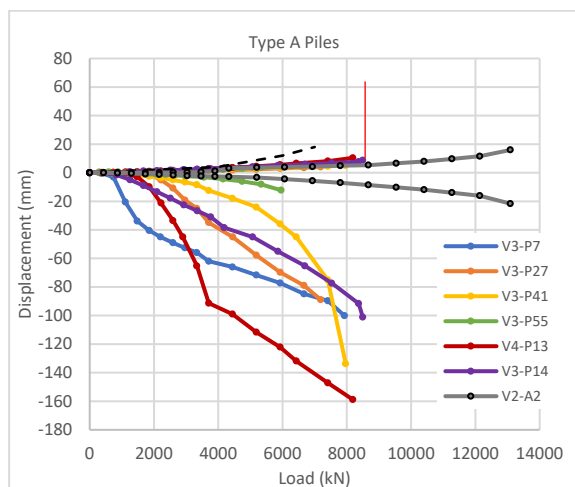


Figure 9. Measured load-displacement curves of Type A test piles

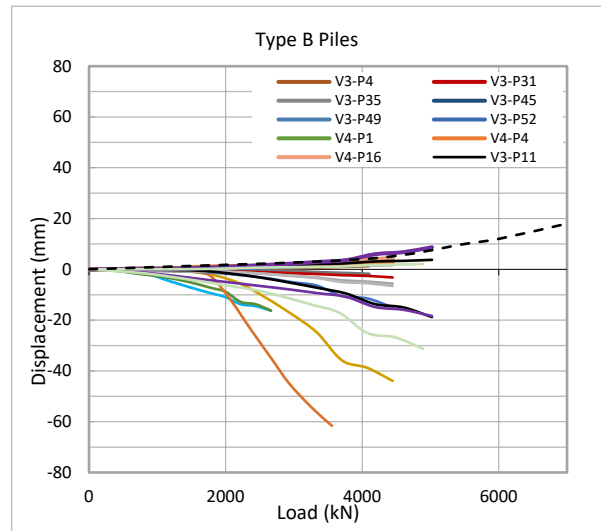


Figure 10. Measured load-displacement curves of Type B test piles

Despite construction records indicating a clean pile base with minimal sediment build-up, the large displacement of the lower test segment in Osborne Formation is likely attributed to some sort of pile base disturbance of the very low strength rock. It is not known exactly what the cause of the soft toe is, but it could be partly attributed to basal heave of the newly unconfined pile base; or relaxing of the rock following removal of the overburden. It is also likely that the sand content falling out of solution over time taken between the last cleaning and concreting. During geotechnical investigations, a loss of strength in the recovered core was evident between point load testing on site and UCS laboratory testing, which at the time was attributed to loss of moisture in the sample as the core became friable and difficult to handle without breaking.

7.2 Equivalent Top-Down Load-Displacement

The measured upward and downward movements of the BDLT tests may be converted into equivalent top-down load-displacement graphs, noting that the mobilisation of f_s and q_b of BDLTs initiates over the bottom third of the pile where load is applied. This is different to load being applied at the pile top as occurs in a conventional static load test. The graphs were constructed for the assessment of serviceability and ultimate loads against allowable pile head deflections limits determined from equations (5) and (6), as per AS2519.

$$P_s \text{ limit} = (P_s L/AE) + (\text{Max } 0.01 \cdot \text{dia. or } 5\text{mm}) \quad (5)$$

$$P_g \text{ limit} = (P_g L/AE) + 10 + (0.05 \cdot \text{dia.}) \quad (6)$$

P_s : Maximum test load for assessment of pile performance at serviceability limit state = E_{ds}

P_g : Maximum test load for assessment of geotechnical ultimate limit state = E_d/Φ_g

A: Cross sectional area of test pile

E: Elastic modulus of test pile

dia.: Test pile diameter

The equivalent top-down displacement curves, including elastic compression, were constructed by summing the upward and downward load for common displacements having subtracted the buoyant weight of the upper test segment. One drawback of the inference of an equivalent top-down displacement curve is that it is limited to the maximum measured displacement of either the upper or lower test segment, and then reliance is placed on extrapolation to project the element with the lesser movement in order to give combined behaviour up to a nominal load.

In all tests undertaken, the lesser movement occurred in the upper segments. Therefore, the upward movement of Type B tests was projected, shown as a black dotted line in Figure 10, as the behaviour was normally dominated by f_s only, and a single hyperbolic curve was made to match the recorded data. Comparing the projected line to measured displacement of Type A piles in Figure 9, the projected line conservatively predicts a higher displacement of the upper test segment than measured. For Type A tests, full mobilisation of the upper test segment at max test load (i.e. displacement to 80mm occurring at max load represented by a straight projected line as shown by the red line in Figure 9) has been conservatively assumed to construct load-displacement curves beyond 10mm displacement. Note: no extrapolation of curves was undertaken for test V2-A2.

The equivalent top-down curves of Type A and Type B piles are presented in Figures 11 and 12, respectively. Figure 11 shows the test piles typically displaced at a constant rate with load to around 15mm, where the rate of displacement abruptly increased. This abrupt change is due to the conservative assumption that full mobilisation of the upper test segment was achieved at maximum test load as discussed above and is not considered to be the actual load-displacement behaviour of the test piles.

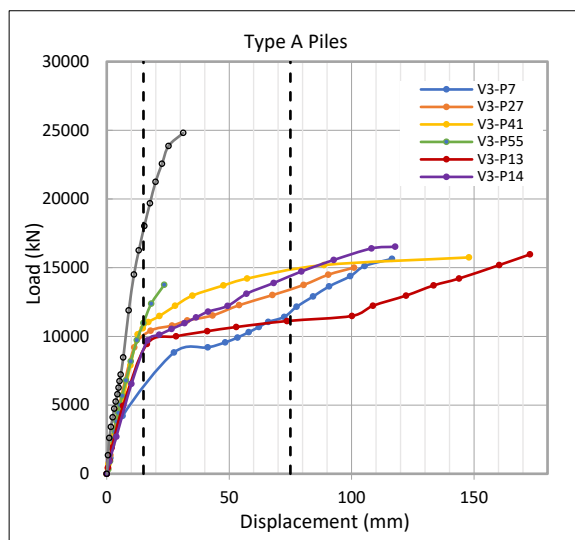


Figure 11. Equivalent top-down load-displacement curves of Type A piles

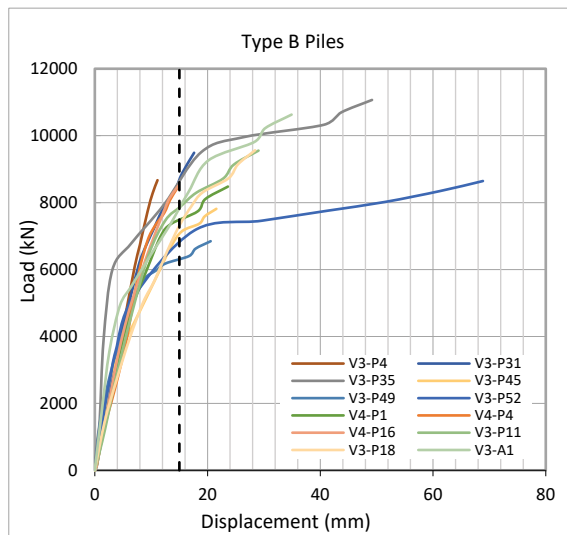


Figure 12. Equivalent top-down load-displacement curves of Type B piles

Based on equations (5) and (6), the acceptance pile head deflections for service loading ranged between 15mm and 2mm; and between 75mm and 90mm for ultimate loading. Note: The lower acceptance deflections are presented in the figures. Allowable service load and ultimate load of each ground model were determined from above figures for the project, to meet AS 2519 criteria.

7.3 Load Distribution

The supercells of all test piles are located within the bottom third of the piles, except for V2-A2 test where the cells were located at the pile base. Type A test piles were instrumented with strain gauges such that the distribution of pile load with depth could be calculated.

The non-linearity of the secant modulus of pile composite material (E_{comp}) vs strain behaviour was considered in the interpretation of pile load distribution. E_{comp} is determined from the tangent modulus shown as:

$$(\delta\sigma / \delta\varepsilon) = A \varepsilon + B \tag{7}$$

Which can be integrated to:

$$\sigma = \left(\frac{A}{2}\right) \varepsilon^2 + B \varepsilon \tag{8}$$

As

$$\sigma = E_{comp} \varepsilon \tag{9}$$

Then

$$E_{comp} = \left(\frac{A}{2}\right) \varepsilon + B \tag{10}$$

$(\delta\sigma / \delta\varepsilon)$: tangent modulus of composite pile material

E_{comp} : secant modulus of composite pile material

σ : stress

ε : measured strain

A, B: constants determined from measured σ and ε .

An example of determination of E_{comp} and stress, from measured strain for test V3-P27 is discussed below. Firstly, the $(\delta\sigma / \delta\varepsilon)$ versus strain curves measured

from test V3-P27 at 6 strain gauge levels nearest to the test cells (Levels U1, U2, U3, L1, L2, L3 – U1 being the strain gauge level just above the test cells and L1 just below) were plotted as shown in Figure 13. The curves show initial high $(\delta\sigma / \delta\varepsilon)$ for strain gauges located further away from the supercells (at Level U3, L2 and L3) due to shaft resistance that developed between the gauges and cells, therefore only readings from gauges near the supercells (Level U1, U2 and L1) were considered for the calculation of E_{comp} for this test. The curves converge to a straight line with irregularities. Large irregularities are usually due to unloading and reloading of the test pile, or when test load intervals are not uniform. For interpretation of E_{comp} , only readings up to 100% P_s (before holding) were considered.

The best-fit line of the $(\delta\sigma / \delta\varepsilon)$ vs ε curves in Figure 13 are represented by the following equation (11), and E_{comp} determined from equation (12):

$$y = -0.1678x + 57.61 \tag{11}$$

$$E_{comp} = -\frac{0.1678}{2}\varepsilon + 57.61 \tag{12}$$

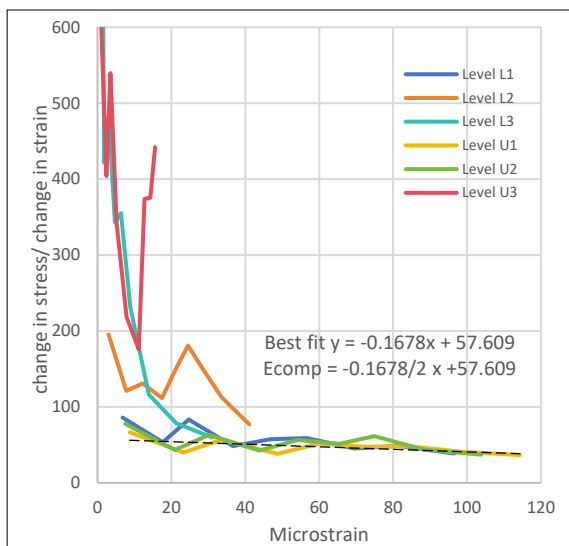


Figure 13. Tangent modulus variation with strain

Equation (12) shows that E_{comp} varies with strain. E_{comp} at zero strain is calculated for each test and is checked against the concrete stiffness measured before pile testing. E_{comp} varied by 30 - 40% from the start of the test to the end of the test. It also varied along the length of the test pile. As a result, pile load-distribution interpreted from test data is sensitive to the non-linear behaviour of E_{comp} with strain.

The stress (hence load) during testing can be determined from measured strain as shown in equation (9). Comparing the load calculated based on a constant E_{comp} and load based on E_{comp} corrected to strain variation, the corrected pile load distribution yields lower shaft friction but higher end bearing.

Load distribution of tests V3-P27 and V4-P13 calculated from equation (9) are presented in Figures

14 and 15. Anomalies observed in the figures are likely due to the following:

- Larger than anticipated load (circled in red in Figures 14 and 15) at pile bases, test cell locations and lower test segment of V3-P27, is likely due to reduced pile diameter at the same locations evident from TIP in Figures 5 and 6.
- Apparent negative skin friction (dotted red circle in Figure 14) is likely due to applied load being transferred to the Osborne Formation, causing the overlying looser sand layer to move more in the direction of loading with respect to the pile.

By considering an average skin friction along the pile, inaccuracies due to the anomalies may be removed. Where anomalies occurred at the pile base, the values have been disregarded in the calculation of skin friction and end bearing.

Load distribution curves of Type A tests at maximum load are presented in Figure 16, noting that V2-A2 load distribution founded on Kings Park Formation has not been presented as it was at a slightly different elevation. The equivalent top-down load distribution curves (presented as dotted lines) were determined by mirroring the load-distribution of the upper test segment.

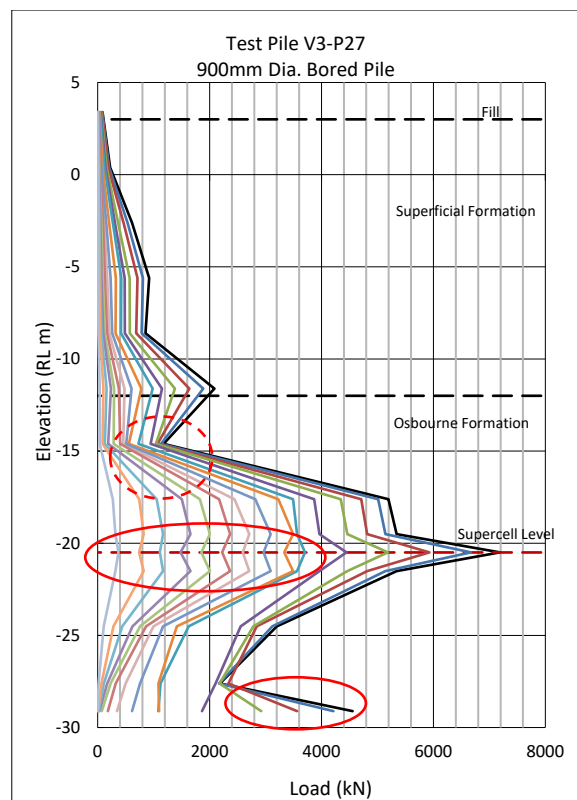


Figure 14. Load-distribution of test pile V3-P27

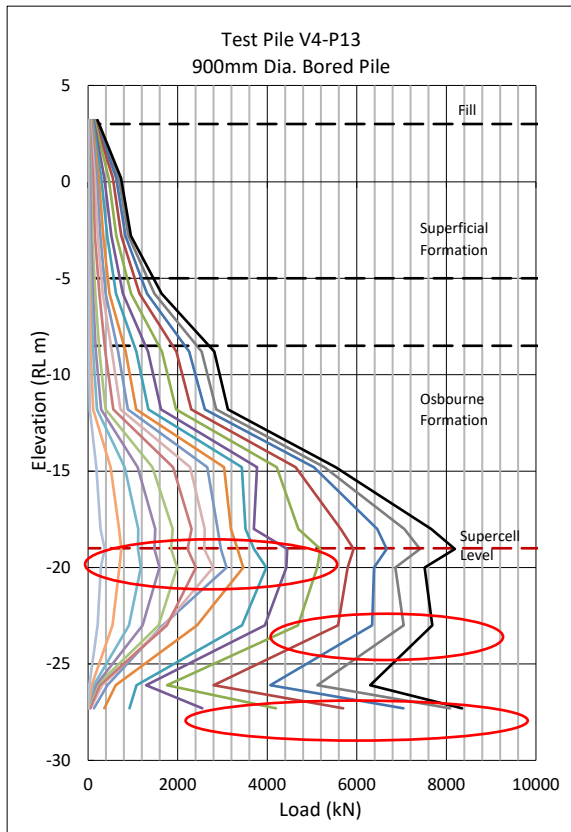


Figure 15. Load-distribution of test pile V4-P13

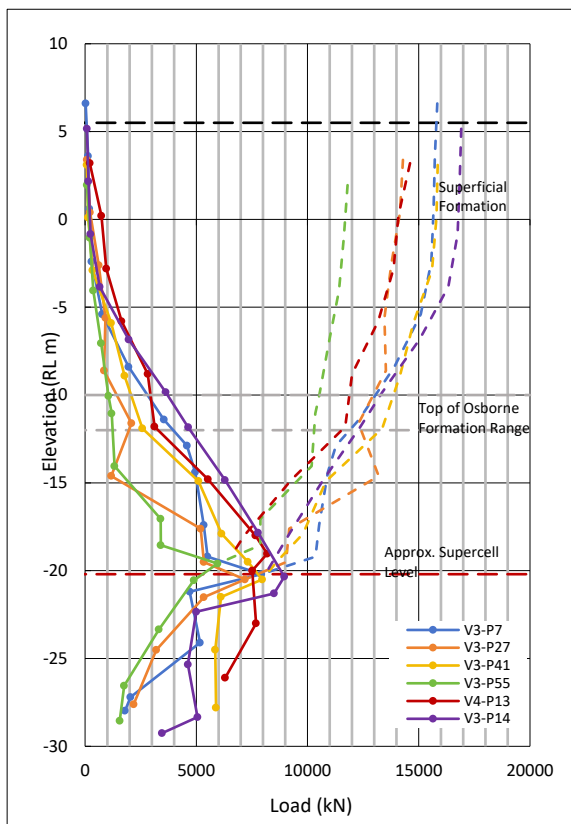


Figure 16. Load distribution along Type A piles at maximum test load (note: V2-A2 curve was not presented as it was at a slightly different elevation)

7.4 Shaft Friction and End Bearing

Measured f_s in the superficial formation, Osborne Formation and q_b values are presented in Figures 17 to 19 (note: V2-A2 was the only test pile founded in Kings Park Formation). The lower test segments of all test piles apart from V3-P55 and V2-A2 have displaced significantly where ultimate f_s and q_b may be determined at allowable deflection limits of 75mm to 90mm.

Figure 17 shows that mobilised f_s in the superficial formation reached values of 60-100kPa where some tests started to plateau (i.e. near ultimate values). The ultimate f_s was similar to values estimated based on LCPC method shown in equation (2).

Mobilised f_s of the upper test segments in Osborne Formation ranged between 150 and 220kPa and could potentially be higher at full mobilisation, as shown in Figure 18. These values were as estimated by the conventional UCS-based correlation shown in equation (4). The ultimate f_s of the lower test segments, however, were lower than design estimates, ranging between 75 and 230kPa at the allowable deflection limit. As the UCS of Osborne Formation slightly increased with depth, the decrease in f_s below the supercells was not anticipated. The low f_s below the supercells is likely attributed to the large displacement of the pile bases as it was evident that the lower segment of test V3-P55 underwent small displacement and yielded f_s similar to the upper test segment. Note: Supercells were located 0.5m from the pile base for test V2-P2, therefore there were no records of shaft friction of the lower test segment even though the pile base displaced small movement. The reverse (where the pile displaced significantly) resulted in a low Poisson effect of concrete, therefore lower shaft friction. The average f_s in Osborne Formation along the piles (i.e. above and below test cells) were between 110 and 220kPa. These values correlate reasonably with lower bound values estimated using equation (4).

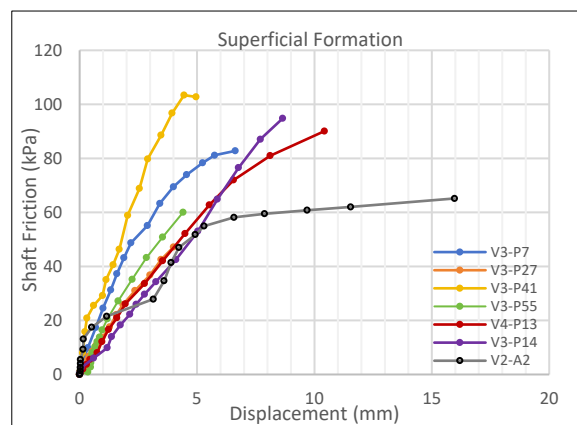


Figure 17. Mobilised shaft friction of bored piles in superficial formation

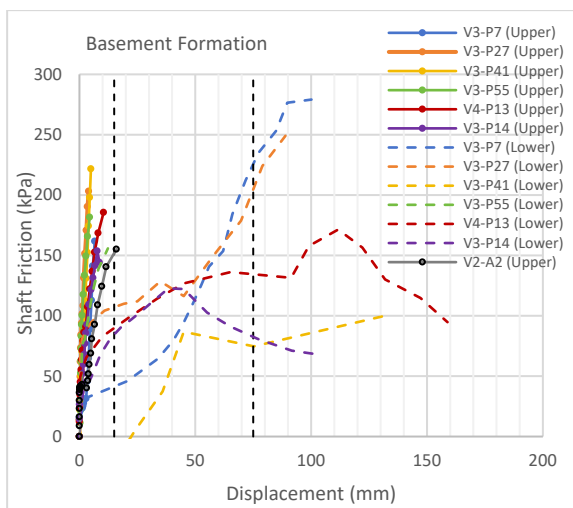


Figure 18. Mobilised shaft friction of bored piles in Osborne

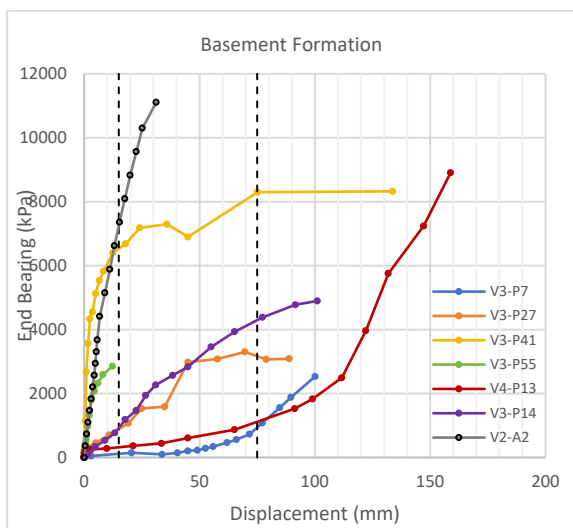


Figure 19. Mobilised end bearing of bored piles

From Figure 19, ultimate q_b at the allowable deflection limit ranged between 1,000 and 9,000kPa. Low ultimate values recorded in V3-P7 and V4-P13 were lower than calculated from equation (5). These low values in the Osborne Formation appear to result from base disturbance resulting in an initial soft load displacement response and high displacement with stiffness of the response increasing with increased load.

Mobilised skin friction in Kings Park Formation (test V2-A2) is around 150kPa, and end bearing 11,110kPa. These values could be higher at full mobilisation.

8 ADVANTAGES OF BDLT

BDLT was employed on this project in order to verify and validate the design of foundations for the high significance rail viaduct. This was a first in Western Australia in terms of both the viaduct structure and the extent of BDLT undertaken.

The advantages of using BDLT include:

- BDLT may be carried out where sensitive services exist that prevent the use of dynamic testing methods
- Only a single pile needs to be installed rather than the requirement for reaction piles for conventional static top-down testing.
- Test equipment is minimal, requiring only a pressure pump, generator, and control box.
- Water can be used in place of oil as the hydraulic fluid to pressurise the jacks, which reduces potential environmental impacts.
- Application of the load close to the pile base may be able to mobilise pile base in soft rock, allowing true verification of base capacity, and where hard rock is present, the full shaft friction may be measured.
- The cost of supercell testing is significantly less than some other forms of testing.
- The cells are custom designed, and jacks may be added or removed to suit the proposed test loads. Similarly, the jack stroke can be tailored to suit.

9 CONCLUSION

Nineteen bi-directional load testing of the 0.9m and 1.5m diameter bored piles at Viaduct 3 and 4 of the LXR project comprised application of test load within the bottom third of the piles founded in Osborne Formation and Kings Park Formation. This allowed observation of pile behaviour at depth (>35m), and in the rock formation.

Full mobilisation of the lower test segments was observed in several test piles, where ultimate skin friction and end bearing were measured. The lower test segments of the bored piles underwent large displacement, which is likely attributed to pile base disturbance during construction in very low strength Osborne Formation even though the base of these piles appeared to be clean with minimal sediment at the time of concreting. Pile base displacement of test pile founded on Kings Park Formation was small, and Kings Park Formation have been differentiated from Osborne Formation by the presence of marine fauna, as well as by differences in grain size distribution, and measured strength/ consistency.

Constructability factors have likely caused large pile base movement, which in-turn resulted in variable ultimate end bearing and lower bound ultimate shaft friction estimated from conventional UCS-based method in the Osborne Formation. In the Kings Park Formation, ultimate capacities were as calculated by UCS-based method; and in the superficial formation, f_s was as calculated by the LCPC method.

10 ACKNOWLEDGEMENTS

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