A NOVEL GROUND IMPROVEMENT AND DEEP FOUNDATION SYSTEM USING PAILEs

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Abstract

The global demand for infrastructure, including extension of roads, rail or airports requires major investment. Innovations that may provide substantial cost savings are being actively sought through the use of more efficient technology, better risk management and design optimisation. This extends to the development of innovative Ground Improvement techniques and Deep Foundation systems. Techniques that increase the speed of foundation installation, bearing capacity achieved and/or provide greater quality assurance of the installed product have the potential to result in significant benefits in terms of direct cost, materials and time savings.

The PAILE method facilitates the installation of a range of foundation and ground improvement piles with enlarged bases, at a significantly reduced cost compared with current methods. In addition, as the resistance of each PAILE is verified during installation there is reduced risk to all parties. By utilising proven, existing technology in a novel way, PAILEs can be installed using smaller machinery and less material than with traditional piling equipment. The technology thus provides further savings and simplification of onsite operations, as it reduces working platform requirements and associated site preparation works, vibration and ground disturbance.

1 Introduction

The ongoing drive to minimise risk, whilst delivering projects on time and on budget, has resulted in the construction industry becoming particularly cautious about adopting new technology. This has stymied innovation and prevented widespread adoption of new techniques. However, as the construction industry is the largest consumer of raw materials, the source of 50% of solid waste and 15% of greenhouse gas emissions (Bühler et al 2016), it has the potential to make a meaningful impact, both from considerations of sustainability and social responsibility, and from a financial viewpoint.

The potential to realise meaningful savings was the main driver behind the development of the PAILE technology. PAILE combines existing, proven concepts to facilitate the installation of enlarged base concrete piles, in a single, continuous process. Similar to conventional steel helical piles, PAILE technology minimises ground heave, does not result in induced ground vibration and requires a relatively small piling rig to install. This innovative method reduces direct installation costs by employing smaller machinery and less material in comparison to standard piling practice and offers further indirect savings by minimising associated temporary works. For example, the lighter loads associated with the smaller piling rigs reduce the thickness of, or quantity of reinforcement in, the required working platform. Similarly, the streamlined onsite operations associated with PAILE construction potentially further minimise the spatial extents of construction phase ground disturbance.

In addition to the construction benefits, PAILE also lowers project risk by allowing direct measurement of the resistance being achieved during pile construction. The information this measurement provides is similar to that of a static load test. Thus, the technology increases the available QA record and offers the opportunity for design adjustments to be made. Once all PAILEs have been installed, the geotechnical data can be fused with the direct resistance measurements made during installation, to deliver a comprehensive site risk profile and enable further optimisation of the infrastructure design. The applied algorithms are based on the principles of Artificial Intelligence, hence the “AI” in the PAILE product name.

2 Technical considerations

The intention during the development was to find a pile type and installation technique that:

- Had an **enlarged base (**and optionally **enlarged head)**
- Was constructed out of **reinforced concrete**
- Was **installed in a single continuous and cost-effective process**
- Had effectively **preloaded the soil immediately under the pile base to improve load / settlement behavior**
- Used **displacement methods similar to those employed in the installation of steel helical piles;** and
- **Provided pile resistance measurements in real time**

Due to the cost savings, installation efficiencies and improved safety it would offer benefits over established piling techniques and industry practices.

2.1 Pile geometry

The ultimate geotechnical resistance of a compression pile is the sum of pile shaft and base resistance:

$$R_{g,ult} = A_s \cdot f_s + A_b \cdot f_b$$

Where:

- A_s denotes pile shaft area
- f_s is shaft resistance per unit area
- A_b is the pile base area; and
- f_b is base resistance per unit area

For the purpose of this study, the pile ultimate base resistance is defined as the maximum resistance at which steady deep penetration is experienced. It is noted that the pile displacements required to achieve maximum f_s and f_b are significantly different, so the two values may not co-exist. Plunging failure itself may also not be observable in many scenarios, as the base resistance will simply continue to increase within soils that demonstrate a continued strength improvement with depth. Nevertheless, the plunging resistance at any depth can be equated to the CPT tip pressure q_c, weighted for partial embedment and averaged according to the zone of influence of the pile tip as suggested by White and Bolton (2005) and Bogusz (2016).

Various design codes and guidelines (e.g. British Standard BS 8004:2015, French Standard NF P94-262:2012 or Dutch Standard NEN 9997-1:2012) recommend that the ultimate pile bearing resistance be reduced to $f_b = k_c\, q_c$, where k_c is between 0.25 to 0.90 and is dependent on the soil type and pile installation method employed. It is noted that this reduction factor is aimed at limiting pile head settlement to no greater than 10% of the nominal pile diameter under the ultimate design load (Tomlinson and Woodward, 2007).

Considering the above definition of base resistance, and q_c/f_s ratios typically recommended by national codes or design recommendations (e.g. Bustamante and Gianeselli, 1982), it is evident that the ratio of f_b/f_s may range between 30 and 300. Thus, a unit area of pile base may be 30 to 300 times more effective in resisting applied load than the same unit area of its shaft. Consequently, as shown in Figure 1, it is possible that two piles installed immediately adjacent to each other could offer the same ultimate geotechnical resistance yet occupy significantly different volumes. Such varied geometry would not only affect the volume of replacement (pile) material required, it would also significantly alter the volume of spoil produced (for bored piles) and energy required to achieve pile installation.

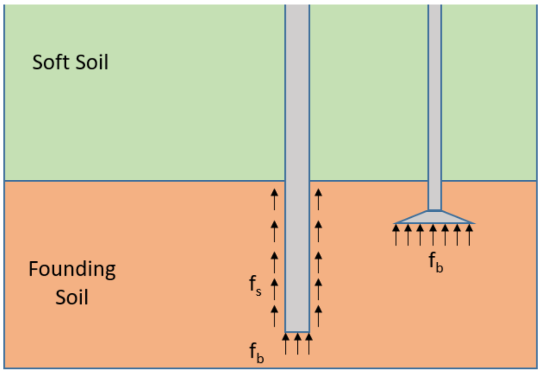

Figure 1: Straight shaft pile (left) vs pile with an enlarged base (right) – both piles would offer same ultimate geotechnical resistance

The two most common methods of forming enlarged bases currently are under-reaming using special boring tools or tamping of the pile base using hammers (e.g. Franki piles). Both processes are relatively time consuming compared to straight shafted piles, which makes them less cost effective and therefore not commonly employed. The above considerations led to the conclusion that, if the process of forming the enlarged base of a pile could become as quick as, or faster than, the installation of a straight shafted pile of a considerably longer length, then piles with enlarged bases should be very competitive due to their optimal material use and superior sustainability outcomes.

2.2 Load / settlement curve

The objective of a pile is to provide adequate load resistance and to limit settlement to an acceptable level under a predetermined load. The act of drilling holes and removal of soil for the purpose of installing piles typically leads to loosening of the soil around the pile shaft and immediately under the pile base, for example by stress relief. On application of a load, a pile must settle in order to recompact the soil and mobilize resistance. Pile shaft resistance is mobilised at relatively small differential movements, typically less than 5 mm. Conversely, the full base resistance of a pile requires much larger settlements to occur, typically 10 to 25% of the base diameter.

When a pile is reloaded the load/settlement response is generally several times stiffer than during the initial loading cycle, as shown in Figure 2. This is because the soil under the pile base has achieved plastic deformation during the first loading cycle. Accordingly, as annotated in Figure 2, if the design maximum allowable settlement of a pile was $\boldsymbol{s_1}$, then preloading the base to a predetermined load $\boldsymbol{P_2}$ prior to unloading and subsequent reloading of the pile, means that the applied load to cause a future settlement of $\boldsymbol{s_1}$ would be increased from $\boldsymbol{P_1}$ to $\boldsymbol{P_2}$.

Pressure grouting of a pile base is one typical method of preloading. As reported by Fleming (1993) and Dapp et al (2006), the use of pressure grouting at the pile base is considered an effective method to increase the base stiffness of a single pile. Pressure grouting is not commonly used in Australia due to the additional costs and time associated with the technique.

Compaction of the soil under the base of a pile may also be achieved by the method employed by Franki piles, and the technique produces an enlarged pile base which, in combination with compaction of the soil, leads to the improved load/settlement response of the pile. However, the disadvantages associated with such a dynamic compaction technique, including slow productivity, uncertainty regarding the size of the oversized base achieved, the introduction of significant ground vibrations and high noise levels, have precluded this method from widespread adoption within Australia. Therefore, any new method of achieving this outcome during the construction process could be highly valuable, if it could both (i) be implemented without requiring significant additional expenditure or time; and (ii) allow verification of the achieved condition.

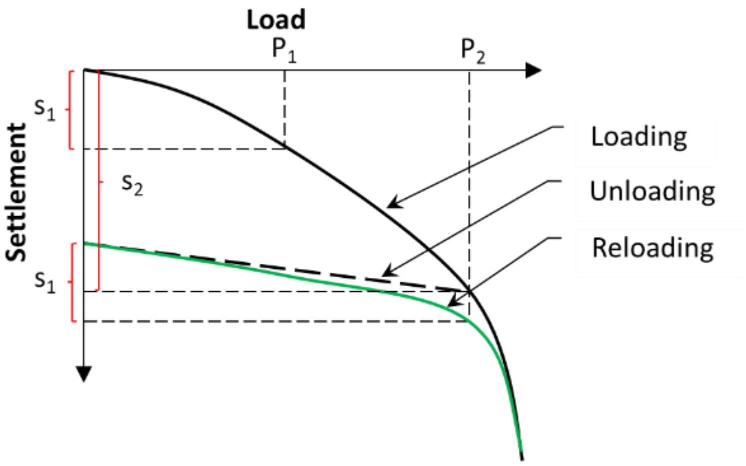

Figure 2: Typical load/settlement curve of a pile base

2.3 Material selection

Concrete is a cost-effective material for construction of axially loaded piles due to its excellent performance in compression. However, when unreinforced, concrete design capacity is typically reduced by 50%. Comparatively, steel offers excellent mechanical properties and high ductility in both tension and compression, yet it is relatively expensive and has a high carbon footprint. The cost of steel piles is further increased if wall thicknesses are increased in order to accommodate sacrificial steel requirements to ensure pile design life in corrosive environments, such as acid sulphate soils or in soils with high chloride contents (e.g. marine applications).

When selecting a suitable pile option for any given application, a reliable capital cost comparison must be conducted in conjunction with assessment of the likely performance of the pile. For example, for compression piles a suitable comparison could be based on the material cost required to form a 1 m long non-bearing section of a pile of a standardised (unit) cross section divided by its design structural capacity. This indicator, termed the Material Performance Unit Cost (MPUC), would then be expressed in units of cost (e.g. Dollar) per unit force (e.g. kN) per unit length (e.g. metre).

Demonstrating the procedure, the MPUC has been calculated for an axially (compressive) loaded pile using typical concrete and steel costs in Brisbane, Australia (as of 2019). The calculated MPUC is plotted in Figure 3 below for five cases: (i) an unreinforced concrete pile (ρ_s = 0%); (ii) a reinforced concrete pile with minimum steel (ρ_s = 0.5%); (iii) a reinforced concrete pile with typical steel rate (ρ_s = 1.5%); (iv) a reinforced pile with maximum steel rate (ρ_s = 4%); and (v) a steel pile (ρ_s = 100%).

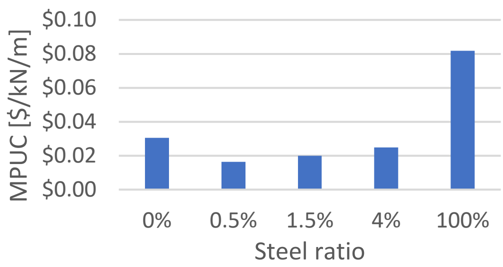

Figure 3: Material performance unit cost comparison (Brisbane, Australia, 2019)

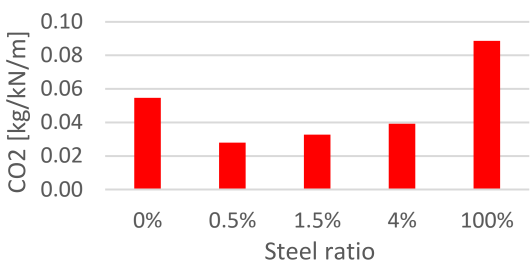

Figure 4: Ratio of embodied CO_2 to material performance

The results of the MPUC analysis indicate that lightly reinforced concrete is, as expected, the most cost-effective material for compression piles. It may also be observed that the optimum steel reinforcement ratio is equal to the minimum steel ratio required as per AS 2159:2009, which is 0.5% for piles fully embedded in ground and without bending.

A similar analysis was completed to evaluate the comparative ‘environmental friendliness’ of each pile type, by calculating the embodied CO_2 per unit length of pile and per unit of compression resistance. As presented in Figure 4, the reinforced concrete piles demonstrated lower embodied CO_2 per standardised pile resistance than either the unreinforced concrete or steel piles. As evidenced by both the MPUC and sustainability criteria, any innovative piling technique would offer the greatest benefit if it utilised reinforced concrete as the construction material.

2.4 Installation method

Pile installation methods can be broadly divided into replacement methods and displacement methods. Pile installation via displacement methods can be further sub-divided based on the nature of actions used in the process, into dynamic methods (such as vibration or hammering of the pile into the ground) or static methods (such as screwing or pressing the pile into the ground). Displacement methods for pile installation are generally regarded as offering greater cost effectiveness in comparison to replacement methods, due to the higher speed of pile installation, improved pile to soil load transfer performance, little to no spoil production and higher degree of quality control offered by these techniques.

In particular, the screwing method of pile installation, such as that associated with steel helical piles, appears to be the most attractive as it offers the following benefits:

- *Low installation energy* – the energy needed to install a displacement pile depends directly on the volume and consistency of soil that needs to be displaced. Because helical piles have an optimal geometry (i.e. slender shaft and large base) the ratio of installation energy to pile bearing capacity is much lower than for full displacement piles.
- *Minimal thrust* – during the screwing in process, steel helical plates advance themselves into the soil, which reduces the need for a high pull-down force to be exerted by the piling rig.
- *Small equipment* – because the installation energy and pull-down requirements are minimised, the size of the construction plant required for the installation of steel helical piles is smaller than for other pile types.
- *Working platform* – because the size of required equipment is minimised, the thickness and quality (e.g. geo-reinforcement) of the working platform may be significantly reduced.
- *Vibrations* – there are no ground vibrations induced by the screwing pile installation technique.

- *Quality control* – although not universally accepted, the torque required to install helical piles is indicative of pile capacity, and is used for pile quality control on a regular basis (Perko, 2009).
- *Limited ground heave and lateral movements* – the small displacement to pile capacity ratio associated with helical piles minimises risks related to site surface heave and soil lateral movements.

Based on the benefits offered by the static screwing pile installation method, it is considered to be superior to any other displacement pile installation method.

2.5 Verification testing

All soils and rocks are spatially variable, and site investigations are aimed at providing information for geotechnical engineers to determine characteristic values from which to calculate pile shaft and base capacities. However, in order to accommodate the variability of ground conditions, design engineers further downgrade the calculated pile capacities using strength or resistance reduction factors. As specified in AS 2159:2009, the geotechnical resistance reduction factor may range from 0.4 to 0.9, and is dependent on the assessed risks arising from, amongst other items, (i) the site geology; (ii) the quality and extent of the ground investigation completed; (iii) the method of deriving design parameters; (iv) structure sensitivity; (v) level of quality control during pile construction; and (vi) type and amount of pile verification testing to be undertaken.

In terms of pile verification testing, the most reliable verification test is a static load test on a completed pile. Although such a full-scale test provides the best quality information for assessing achieved pile resistance, the high cost and time required for full test completion makes it impractical to undertake such a test on every pile. In addition, unless such tests are performed on pre-production trial piles and completed well in advance of final pile design, the results are rarely used to alter the design, but rather solely as a QA assessment in order for the Client to accept the completed piles.

As displacement pile installation methods, such as screwing or hammering, are sensitive to the ground consistency, monitoring parameters available to the operator on piling rigs allow the estimation of ground resistance and thus pile resistance. Although this allows an operator to immediately adjust pile installation depths to account for any observed significant variation in ground conditions, the relation of these plant measured parameters to final pile performance is via generic correlations only. Empirical relationships are derived from past projects and vary widely based on the selected technical reference, in addition to piling rig type and size, piling rig operator and change in drilling tools. Accordingly, verification of pile resistance via reliance on parameters recorded by the piling rig during pile installation is not considered sufficient for QA purposes.

Since the majority of piling works in Australia are let under design and construct contracts, it is the responsibility of contractors to balance the amount, and thus cost, of pre-production testing, design risk reduction factors and verification testing to come up with the best tender offer. Due to high costs and long time required to physically test piles, the amount of pile testing is typically minimal and the geotechnical reduction factors are most often assumed in the range between 0.45 and 0.65. Without pile testing, AS 2159:2009 actually limits the geotechnical reduction factor to 0.4 (Clause 8.2.4).

Based on the fact that the magnitude of the geotechnical reduction factor (ϕ_g) is directly related to the percentage of piles tested within a project, an innovative pile installation method that could complete the equivalent of a static load test on every pile during the execution of a project would be highly valuable, especially if such *in situ* testing could be undertaken with minimal impact on productivity. This could yield significant direct savings to a project by reducing the length of piles, since the design resistance of each pile could be increased with a ϕ_g reduction factor closer to unity, as per AS 2159:2009.

3 Ideal piling system

PAILE technology combines all the desirable concepts outlined in Section 2, to provide a pile installation technique that creates reinforced concrete piles with an enlarged base, in a single, continuous process. This highly desirable outcome is achieved by a specialised PAILE auger arrangement, with the main components being an external tube with one or more helical plates, an internal tube with a single helical plate at the bottom, and a mechanism that provides the thrust necessary to extend the tubes telescopically, displace and prestress the soil and form an enlarged base. The construction sequence is conceptually shown in Figure 5 and Figure 6. In the first phase of the construction sequence, as shown in Figure 5(a), the PAILE auger is screwed into the ground with the helix closed.

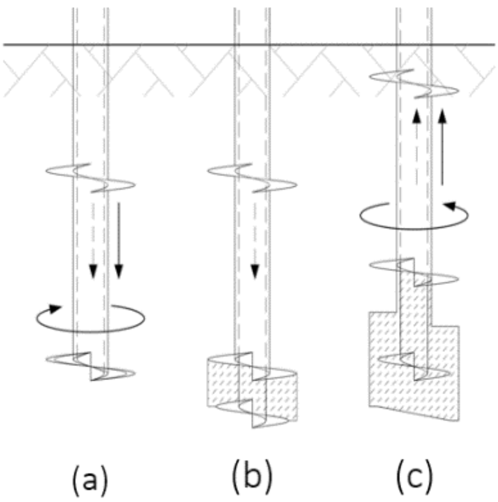

Figure 5: Phases of PAILE installation

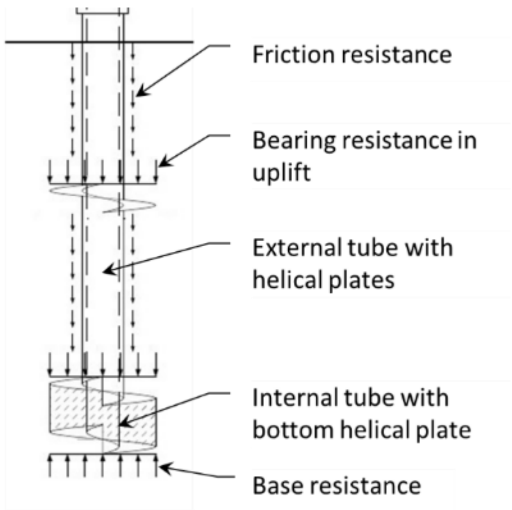

Figure 6: Balance of forces during formation of enlarged base

Once the design depth is reached, the unique construction of the auger allows the creation of an enlarged base by pushing the lower part of the helix down against the reaction of the upper part, as illustrated in Figure 5(b). The diameter and height of the base are defined by the size of the helices and the telescopic movement of the parts, respectively. During the base forming phase, the bottom helix displaces and compacts soil underneath the pile base, which is equivalent to preloading the base of a pile in order to improve subsequent load/settlement performance as previously described.

While the base is formed and the auger is recovered from the ground (Figure 5(c)), concrete is pumped in to fill the void. Once the hole is filled and the auger removed from the ground, reinforcement can be installed in a similar way to CFA piles.

Figure 6 further expands on Figure 5, and annotates the forces acting during the formation of the enlarged base via the PAILE method. It is during this process that the quasi-static load test of the pile base material occurs, whereby a dedicated sensor directly measures the force on the bottom helix required to displace the ground and create the enlarged base. This process may be considered equivalent to a bi-directional static load test and provides immediate assessment/verification of the base resistance. In case the base bearing resistance is insufficient to meet the design requirements, the PAILE helices may be closed up and the auger revert to a drilling phase and penetrate deeper until the minimum design threshold has been achieved during a subsequent pile base formation phase. This process could also be continually repeated to assess the *in situ* resistance available within a targeted material unit or at targeted depths, and used to inform site-specific pile design.

Possible applications of the PAILE technology include:

- Compression or tension piles
- Ground improvement
- Ground anchors
- Geotechnical site investigations

The PAILE method can be also applied to form an enlarged head of the pile by manipulating the helix flights at the ground surface while injecting concrete, during extraction.

4 Field trials

Following successful completion of proof-of-concept trials, a prototype PAILE auger was built and tested in sandy soils.

The main objectives of the field trials were:

- to demonstrate that the process of installation of piles with enlarged bases, called the PAILE method, can be successfully implemented at true scale and using conventional piling equipment;
- to verify the performance of piles under compression and tension loads in sandy soils; and
- to derive a relationship between final static load tests and the Real-Time base resistance measurements recorded during installation

4.1 Ground conditions

The site chosen for the first full scale trials was located in Hope Island, QLD, Australia. Prior to the works, three (3) Cone Penetration Tests (CPTu) were performed to assess the ground conditions (refer to Figure 7).

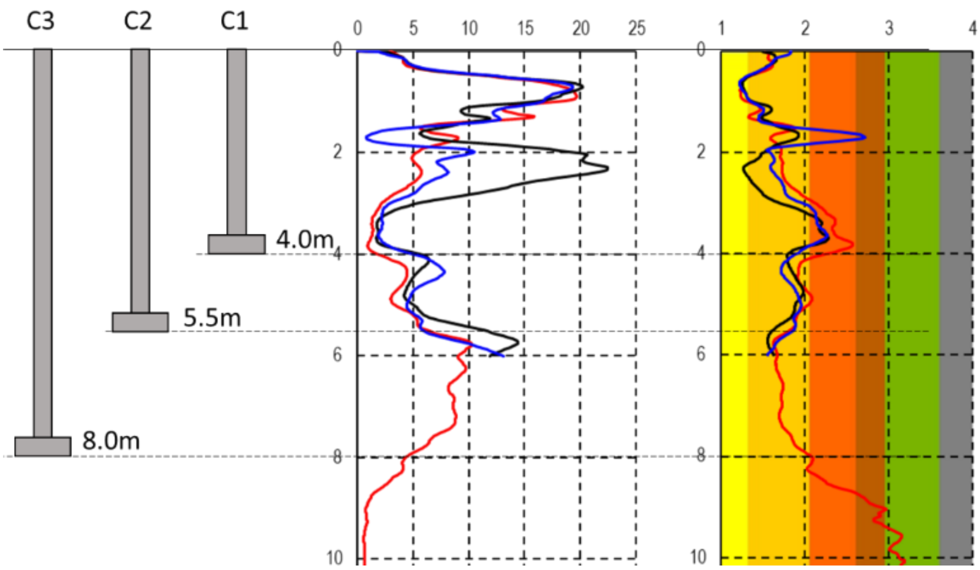

Figure 7: Test piles (left) vs plots of CPT cone resistance q_c in MPa (centre) and SBTn index I_c (right).

The following soil profile was inferred from the CPTs:

- Dredged sand fill, approximately 1.5m thick layer of medium dense to dense sand, underlain by

- Medium Dense Sands, extending to approximately 8.5m depth, with a layer of Loose Sand/Silt mixture at between 3.5m to 4m depth, underlain by
- Firm to Stiff clay extending to the CPT termination depth at 20m.

4.2 Details of piles

The diameter of the PAILE tool displacement body was 273mm and the diameter of helical plates was 650mm, which equates to a base to shaft diameter ratio D_b / D_s = 2.4.

The tool was fitted to a conventional 40t piling rig (refer to Figure 8 below) with a 165kNm rotary head torque capacity.

Figure 8: PAILE tool fitted to piling rig Comacchio CH450 with helical plates closed (left) and open (right).

A total of eight (8) piles, were installed during the trials to depths between 2m and 8m.

4.3 Pile C1

Pile C1 was installed to 4m depth (refer to Figure 7) and instrumented with two vibrating wire strain gauges. One gauge was mounted approximately 1m above the base of the pile and the other at about 1m below the top of the pile.

A static compression load test was carried out 43 days after installation. A procedure from AS2159:2009 was followed.

Results of the test have been presented in Figure 9 below, with explanation of respective curves as follows:

- Black line shows mobilised total load applied vs pile head settlement.
- Blue line shows mobilised base resistance vs base settlement.
- Green line shows difference between applied load and mobilised base resistance vs pile head settlement. Note that this would typically correspond to pile shaft resistance.

Figure 10 presents comparison of CPT cone resistance (red line) and soil yield stresses inferred from gathered data. Explanation of respective curves is given below:

- Black line denotes contour of the pile base.
- Red line denotes CPT cone tip stress qc.
- Blue square denotes soil yield stress measured during base formation.
- Green triangle denotes pile base yield stress, as recorded during static load test.
- Purple dotted line denotes predicted pile base resistance, using LCPC1982 method, plotted within the pile zone of influence assumed between 1.5 D_b above to 1.5 D_b below the base, respectively.
- Purple dashed line denotes predicted pile base resistance, using LCPC2012 method, plotted within the pile zone of influence assumed between 0.5 D_b above to 1.5 D_b below the base, respectively.

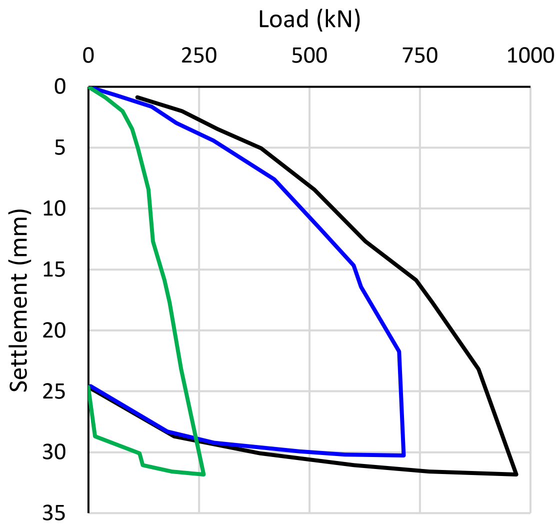

Figure 9: Results of static load test on pile C1.

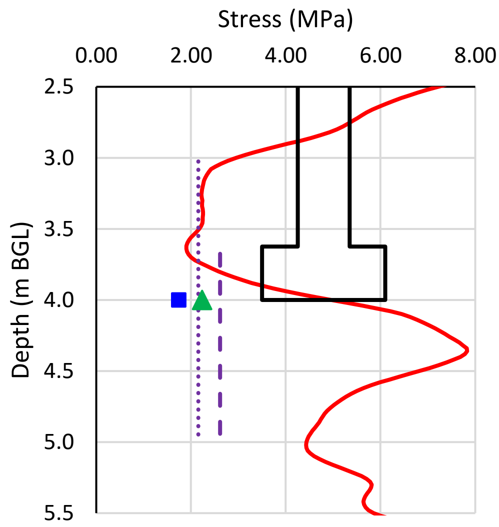

Figure 10: Comparison of yield stresses within pile C1 base zone of influence.

The following conclusions & observations have been drawn from the analysis of the construction records and static test data:

- The pile base ultimate resistance was predicted to be 2154kPa and 2615kPa, using LCPC1982 and LCPC2012 methods, respectively.
- The shaft axial stiffness (EA) calculated from the top strain gauge (SG1) was between 2.3 x 106 to 2.6 x 10^6 kN, which corresponds well with the stiffness calculated using material properties and pile shaft geometry of 2.4 x 10^6 kN.
- The maximum load at the pile base measured during the test was 713kN. Extrapolation of the base load vs settlement curve using method proposed by Chin (1970, 1972), resulted in an inferred ultimate base resistance of 741kN. This load corresponded to a base yield pressure of 2234kPa.
- The measured pile base yield stress was between the predicted values using both LCPC methods. It must be noted that weaker silty sand / sandy silt, with an average CPT qc of 2.16 MPa and located between depths of 3.1m to 3.8m, could potentially be thicker or slightly lower

in the location of the pile, which would explain the lower base resistance value measured during the static test as compared to the base yield stress predicted using LCPC2012 method.

- During formation of the enlarged base a total load of 451kN was measured. It was observed that during the process, the upper helix moved upward and as anticipated considering the location of the base with respect to the CPT qc profile (refer Figure 10). The soil yield stress during base formation was thus 1740kPa (relevant to the helix upper surface), which while indicative of soil strength above the pile base, was only 22% lower than the pile base resistance from the static test.
- The above observation shows that the soil resistance measured during construction of the pile base provides a reasonably good indication of the pile base ultimate resistance while being on the conservative side.
- 95% of the base resistance was mobilised at a settlement corresponding to 3.3% of the base diameter. Typically, full mobilisation of pile base resistance in sandy soils occurs at settlements of 10% to 20% of base diameter. This observation confirms that the process of forming the pile base leads to stiffer load settlement behavior, and as such allows the base resistance to be better utilized at working load conditions.

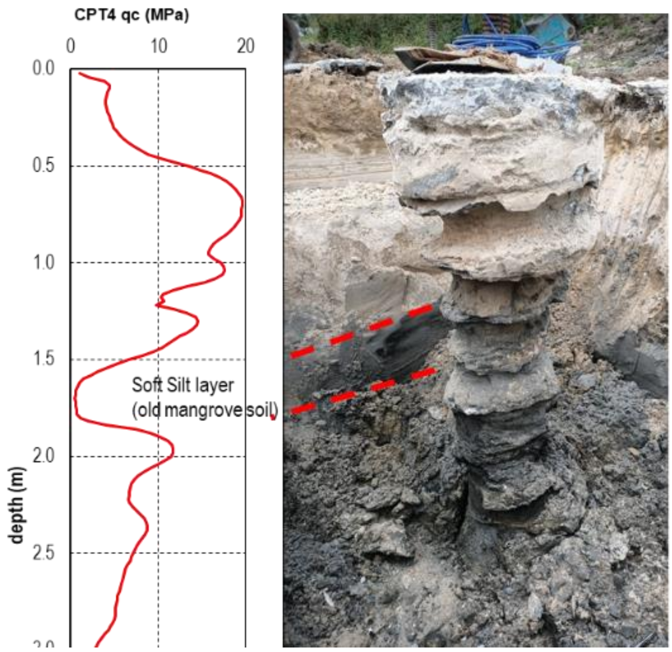

Figure 11: Pile C1 during excavation vs CPT 4 qc profile

- The green line on Figure 9, typically corresponding to pile shaft resistance, after initially softening at settlement of 3mm, maintained constant slope without reaching an ultimate value to the maximum test load. This behavior was deemed unusual, as typically pile shaft resistance is expected to be fully mobilised at settlements less than 5mm. The increasing resistance indicated by constant slope of the green line is speculated to be due to the presence of an enlarged pile head, which was observed after pile excavation (a consequence of lifting the auger without rotation during the final stage of auger recovery). This oversized head, which was founded on medium dense to dense sand (refer Figure 12), would be expected to mobilise some bearing resistance which would add to the shaft resistance inferred from the test at settlements greater than 5mm (approximately 1% of oversized head diameter).

- Upon excavation, the pile shaft was observed to have a spiral flight around the shaft, a clear consequence of concrete filling the voids left by the helical plates during unscrewing (refer to Figure 11 below). This additional spiral flight would also be expected to lead to higher geotechnical shaft resistance R_{gs}.

4.4 Pile C2

Pile C2 was installed to a depth of 5.5m below the ground surface and a compression static load test was carried out 44 days later.

While the pile was also instrumented with vibrating wire strain gauges, similarly to pile C1, the recorded data did not provide meaningful results, possibly due to errors during installation, therefore these results are neither presented nor further discussed.

Figure 12 below presents the load/deflection curve as measured during the test, where:

- black line shows applied load vs pile head settlement, and
- red line shows load deflection curve as predicted using LCPC2012 method.

Figure 10 presents comparison of soil yield stresses measured and predicted, where:

- black line denotes contour of the pile base,
- red line denotes CPT cone tip stress q_c.
- blue square denotes soil yield stress measured during base formation,
- purple dotted line denotes predicted pile base resistance, using LCPC1982 method,
- purple dashed line denotes predicted pile base resistance, using LCPC2012 method.

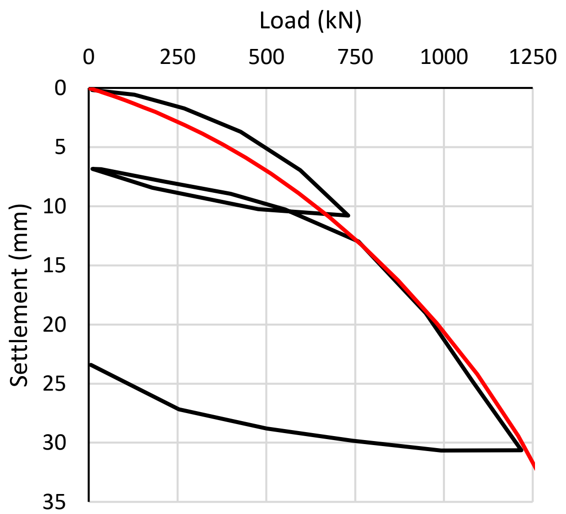

Figure 12: Results of static load test on pile C2.

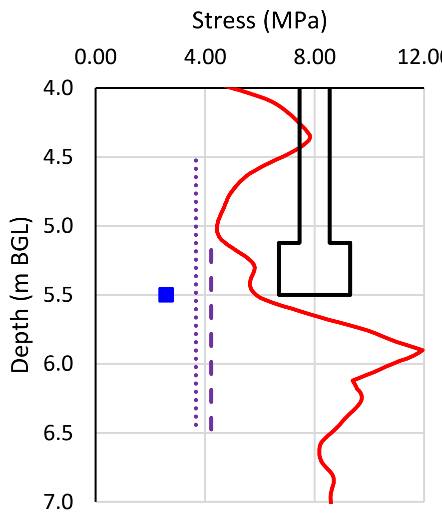

Figure 13: Comparison of yield stresses within pile C2 base zone of influence.

The following conclusions & observations have been drawn from the analysis of the construction records and static test data:

- The pile base ultimate resistance was predicted to be 3664kPa and 4226kPa, using LCPC1982 and LCPC2012 methods, respectively.

- The pile total ultimate resistance predicted using LCPC2012 method was 1802kN, using the nearest CPT data (located 5.7m away from the pile).
- Extrapolation of the pile total resistance using method proposed by Chin (1970, 1972), resulted in an inferred ultimate pile resistance of 1941kN, which was 8% higher than predicted using LCPC2012 method.
- The predicted load/deflection line (red line) was calculated assuming pile base load transfer law as for precast driven piles, which offer the highest pile base stiffness. While the PAILE method uses screwing process to penetrate the ground, the measured load settlement curve fits well with the prediction. It is speculated that this is due to the compaction of the soil under the pile during the base forming stage.
- During formation of the enlarged base a total load of 703kN was measured. It was observed that during the process, the upper helix moved upward and as anticipated considering the location of the base with respect to the CPT qc profile (refer Figure 13). The soil yield stress during base formation was thus 2574kPa (relevant to the helix upper surface) which, while indicative of soil strength above the pile base, was only 30% and 39% lower than the pile base resistance predicted using LCPC1982 and LCPC2012 methods, respectively.
- It is speculated that the lower yield stress measured during the base formation process was due to disturbance of the soil during the drilling process. Indeed, the installation records show that during the drilling phase, the auger was advanced with over-rotation, i.e. augering was taking place when penetrating through the medium dense sands.
- The above observations show that the soil resistance measured during construction of the pile base provides a conservative estimate of the pile base ultimate resistance.
- The observed load/settlement behavior was slightly stiffer than predicted up to ~730 kN load, which corresponds well with the maximum force measured during the base formation process of 703 kN. This confirms that the process of forming the base leads to a stiffer load/settlement behavior of the completed pile.

4.5 Pile C3

Pile C3 was drilled to depth of 8m, and a static tension load test was carried out 45 days after installation.

During the base formation process, the bottom helical plate moved downward, thus increasing the pile depth to 8.4m.

Figure 14 below presents the load/deflection curve measured during the test.

Figure 15 presents a comparison of CPT cone resistance (red line) and soil yield stresses inferred from gathered data, where:

- black line denotes contour of the pile base,
- red line denotes CPT cone tip stress qc.
- blue square denotes soil yield stress measured during base formation,
- purple dashed line denotes average CPT q_c within 2.66 D_b below the helical plates location at the start of base formation process

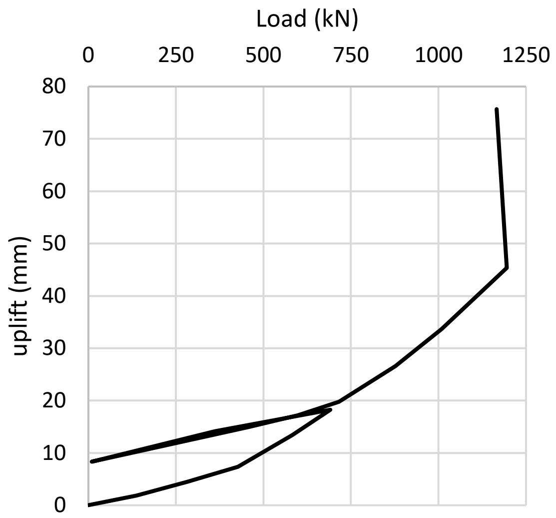

Figure 14: Results of static load test on pile C3.

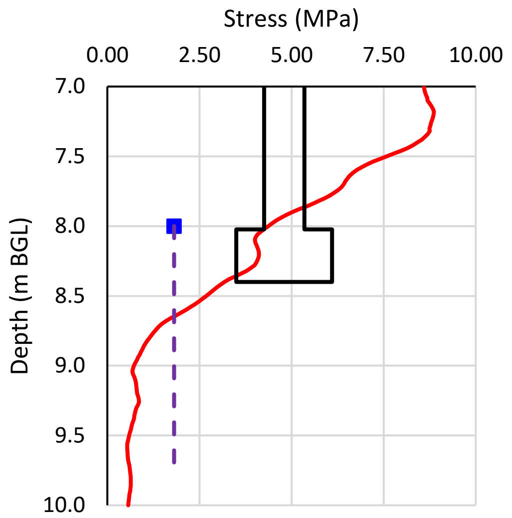

Figure 15: Comparison of yield stresses within pile C3 base zone of influence.

The following conclusions & observations have been drawn from the analysis of the construction records and static test data:

- The pile total ultimate uplift resistance predicted using the nearest CPT data (located 10m away from the pile) was 1394kN.
- The maximum applied load during the test was approximately 1350kN after which geotechnical failure occurred. This load was observed on the load cell indicator gauge immediately prior to the failure; however, the relevant displacement wasn't recorded, unfortunately.
- The maximum sustained load after the failure was 1195kN. Every attempt to increase the load above 1250kN resulted in creeping of the pile.
- During formation of the enlarged base a total load of 600kN was measured. It was observed that during the process, the bottom helix moved downward and as anticipated considering the location of the base with respect to the CPT q_c profile (refer Figure 15). The soil yield stress during base formation was 1809kPa (relevant to the helix bottom surface), which corresponds to an average CPT q_c between depths of 8.0m and 9.7m (refer dashed purple line on Figure 15).
- The measured ultimate uplift resistance was 14% lower than predicted. It is speculated that this was due to the helical plates disturbing the soil during screwing the auger into the ground. Indeed, the installation records show that during the drilling phase, the auger was advanced with over-rotation, i.e. augering was taking place when penetrating through the medium dense sands.
- Even though the measured resistance was slightly less than predicted, the overall performance of the pile was considered very good, considering the disturbance of the soil during the installation process.

4.6 Geometry verification

Upon completion of static load tests, the short piles have been excavated and recovered from the ground for inspection.

Photos of piles fully recovered from the ground are shown in Figures 16 to 18.

Figure 16: Side view of recovered piles

Figure 17: Pile base formed in silt (left) and in sand (right)

Figure 18:Pile shaft after removal of helical thread

The following observations have been made during inspection of the piles:

- the piles were confirmed to have achieved the target geometry, i.e. shaft and base diameters of 273mm and 650mm, respectively. This clearly shows that it is possible to form an enlarged base using the PAILE method in a controlled manner. The resultant base dimensions are driven by the diameter and stroke of the helical plates, and as such, are controllable.
- The pile shaft was solid and without any signs of contamination with soil.
- Additional helical thread was formed around the pile shaft. This is a consequence of concrete filling the voids left by the helical plates during unscrewing.

4.7 Trials summary

- It has been demonstrated in the field that the installation of a full-scale foundation pile with an enlarged and prestressed base using the PAILE method is practical and effective.
- The geometry of the installed piles has been confirmed to be in line with the design assumptions.
- Soil resistance measured during construction of the pile base provides a reasonably good indication of the pile base ultimate resistance while being on the conservative side.
- The capacities measured during static tests have been shown to be slightly (~5%) higher than anticipated from the design.

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- Data from the static tests indicate that the load settlement curves of the piles are stiffer up to the value of the load used to form the base. This confirms that the PAILE method of base formation leads to stiffer pile response and hence can offer better utilisation of pile resistance under working conditions.
- The ground disturbance above the base does not negatively impact the performance of compression piles.

5 Cost Example

In order to demonstrate potential economic advantages of the PAILE system, the following construction development is considered:

- Total building footprint 5,000 m^2
- 30kPa average uniformly distributed design load (e.g. heavy industrial warehouse, 3 level residential development or 2 level commercial/light industrial building),
- The total design load of the development is thus 150,000 kN

Further, assume that the subsurface conditions are as follows:

- *FILL* – uncontrolled fill material extending to an average depth of 1m below ground surface (BGL), underlain by
- *Holocene Alluvium* – Soft to Firm Clays (average CPT q_c = 0.7MPa) extending to an average depth of 8m BGL, underlain by
- *Pleistocene Alluvium* – Very Stiff Clays (average CPT q_c = 2.5MPa) extending to an average depth of 25m BGL, underlain by
- Residual Soil / weathered rock – Very Stiff to Hard / dense geomaterial, extending to an average depth of 35m BGL, underlain by
- Fresh rock

A simplified cost comparison has been performed for six (6) different piling techniques that could be considered for the subject development and results presented in the table below:

Pile type	CFA	Drilled Displacement	Timber Driven	Precast Driven	Steel Helical	PAILE
Shaft diameter	600 mm	450 mm	200 mm	300 mm	219 mm	250 mm
Base diameter	600 mm	450 mm	200 mm	300 mm	2x 650mm	750 mm
Piling rig size	35 ton	90 ton	35 ton	60 ton	30 ton	30 ton
Pile depth	16 m	16 m	35 m	35 m	14 m	10 m
Design capacity	667 kN	605 kN	850 kN	1,820 kN	552 kN	548 kN
Number of piles	225	248	176	82	272	274
Pile cost	$3,000	$1,500	$2,200	$5,000	$1,700	$640
Total pile cost	$675,000	$372,000	$388,000	$412,000	$462,000	$175,000
Establishment cost	$20,000	$90,000	$15,000	$60,000	$15,000	$20,000
Working platform cost	$45,000	$120,000	$45,000	$90,000	$30,000	$30,000
Total costs	$740,000	$582,000	$448,000	$562,000	$507,000	$225,000
Ratio to the lowest cost	**3.3**	**2.6**	**2.0**	**2.5**	**2.3**	**1.0**

Table 1: Sample cost comparison. $ = AUD$

The above costs have been estimated based on typical plant hire, human resources and material supply rates for Brisbane, Australia (2019).

6 Conclusions

By evaluating various aspects of pile design, pile types and current pile installation techniques, the features that would make PAILE technology able to provide a cost-effective outcome have been identified.

The PAILE method enables the installation of foundation and ground improvement piles with enlarged bases, of a range of capacities, at significantly reduced cost when compared with currently available piling techniques. Additionally, as the resistance of each pile installed via PAILE is effectively verified during installation, the technique offers the additional advantages of reducing risk to all parties and makes possible on-site pile depth adjustments to reflect design requirements. These quasi-static resistance measurements could be further aggregated across a project footprint and analysed to deliver a comprehensive site risk profile and enable further optimization/verification of the project design.

Some of the advantages offered by the PAILE technology include:

- Optimal material consumption due to incorporation of slender shaft pile with enlarged base
- High load/settlement stiffness due to preloading of the pile base
- Simple and continuous installation process in one pass
- Minimised ground heave due to high capacity to pile volume ratio
- No induced ground vibrations during pile installation
- Use of cost-effective building material (reinforced concrete)
- Requires comparatively small and maneuverable piling equipment
- Instant verification of pile bearing resistance available during pile installation phase

7 References

AS 2159:2009	Australian Standard, A. S. 2159-2009 Piling-Design and Installation.
BS 8004 (2015)	Code of practice for foundations, *BSI*, London.
Bogusz, W (2016)	Deriving a reliable CPT cone resistance value for end-bearing capacity calculation of piles. *Acta Scientarium Polonorum. Architectura* 15, 2.
Bühler, M M. (2016)	White Paper: Shaping the Future of Construction, Insights to Redesign the Industry. http://www3.webforum.org/docs/WEF_Shaping_Future_Construction.pdf.
Bustamante, M, and L Gianeselli. (1982)	Pile Bearing Capacity Prediction by Means of Static Penetrometer CPT. *Proceedings of the 2nd European Symposium on Penetration Testing*, 493–500.
Chin, F. K. (1970)	Estimation of the ultimate load of piles from tests not carried to failure. *Proceedings of the 2nd Southeast Asian Conference on Soil Engineering*, Singapore.
Chin, F. K. (1972)	The inverse slope as a prediction of ultimate bearing capacity of piles. *Proceedings of the 2nd Southeast Asian Conference on Soil Engineering*, 83-91.
Dapp, S D, Muchard,	Experiences with Base Grouted Drilled Shafts in the Southeastern United

M and Brown, D A. (2006)	States. *Proceedings of 10th International Conference on Piling and Deep Foundations*, 1–10.
Fleming, WGK (1993)	The improvement of pile performance by base grouting. Proc. Institution of Civil Engineers, Civil Engineering, 97, May, 88-93
Hammond, G., Jones, C., Lowrie, E.F. and Tse, P. (2011)	*Embodied carbon. The inventory of carbon and energy (ICE)*
NF P94-262 (2012)	Justification of geotechnical work – National application standards for the implementation of Eurocode 7 – Deep foundations, AFNOR, Paris.
NEN 9997-1 (2012)	Geotechnical design of structures, - Part 1: General rules, NEN, Delft
Perko, H A. (2009)	Helical Piles: A Practical Guide to Design and Installation. *John Wiley & Sons.*
Tomlinson, M, and Woodward, J. (2007)	*Pile Design and Construction Practice*. CRC Press.
White, D J and Bolton, M D (2005)	Comparing CPT and Pile Base Resistance in Sand. *Proceedings of the Institution of Civil Engineers-Geotechnical Engineering* **158**, 1, 3–14.