International Journal of Earth, Energy and Environmental Sciences
ISSN: 2517-942X
Vol:11, No:9, 2017

Design and Construction Validation of Pile Performance through High Strain Pile Dynamic Tests for both Contiguous Flight Auger and Drilled Displacement Piles

S. Pirrello

Abstract**—**Sydney's booming real estate market has pushed property developers to invest in historically "no-go" areas, which were previously too expensive to develop. These areas are usually near rivers where the sites are underlain by deep alluvial and estuarine sediments. In these ground conditions, conventional bored pile techniques are often not competitive. Contiguous Flight Auger (CFA) and Drilled Displacement (DD) Piles techniques are on the other hand suitable for these ground conditions. This paper deals with the design and construction challenges encountered with these piling techniques for a series of high-rise towers in Sydney's West. The advantages of DD over CFA piles such as reduced overall spoil with substantial cost savings and achievable rock sockets in medium strength bedrock are discussed. Design performances were assessed with PIGLET. Pile performances are validated in two stages, during constructions with the interpretation of real-time data from the piling rigs' on-board computer data, and after construction with analyses of results from high strain pile dynamic testing (PDA). Results are then presented and discussed. High Strain testing data are presented as Case Pile Wave Analysis Program (CAPWAP) analyses.

Keywords**—**Contiguous flight auger, case pile wave analysis, high strain pile, drilled displacement, pile performance.

I. Introduction

THE increasing rise in house pricing within Sydney has driven property developers in investing in what used to be expensive areas to develop. These areas are usually underlain by deep soft soils for which conventional piling technique, such as bored piles, do not perform well. On the other hand, CFA and DD Pile techniques are more appropriate for these ground conditions. Also, DD piles results in a substantial reduced spoil quantity compared to CFA piles. This paper will discuss the design and construction challenges encountered with both piling techniques, their limitations and advantages within the same ground conditions. Estimated design loads settlements curves are validated by means of PDA testing and results are discussed.

II. Geological and Geotechnical Characterization of the Site

The Sydney 1:100 000 Geological Series Sheet shows that the site is underlain by man-placed filling and alluvial/estuarine sediment. The sediment comprises silty to peaty sand, silt and clay. Shell layers are also expected to be present within the soil profile. The Prospect/Parramatta River 1:2500 Acid Sulphate Soil Risk Map shows the site as 'Disturbed Terrain' in which soil investigations are required to assess acid sulphate soil potential. Several Cone Penetration Tests and Boreholes were carried out during the geotechnical investigation. All boreholes were extended to bedrock and NMLC-sized diamond core drilling was used to obtain 50-mm continuous core of the rock, enabling unconfined compression strength testing to be carried out on selected samples. A summary of the ground conditions encountered on this site and inferred from the CPTs can be described as follows:

- FILL (**Unit A**) – concrete, bitumen, sand, gravel and clayey filling to depths between 0.3 m to 1.8 m;
- ALLUVIUM (**Units B and C**) – clayey and sandy soils to depths of between 16m and 23m. The soils became firm to stiff (clays) and dense to medium dense (sands) with depth. Occasionally soft and loose layers were also encountered;
- SANDSTONE BEDROCK (**Unit D**) – encountered from depths of between 16 m and 23 m. The bedrock had typically a weathered layer low strength (LS, defined as having typical UCS values of ≈2 to ≈6 MPa) up to 1m thick over medium strength (MS, UCS ≈6 to ≈20 MPa) to high strength to the base of the cores at 22 m to about 30 m.

Stiff clay/dense sand (i.e. Unit C) variability can be appreciated on the two cross sections of the site given in Figs. 2 and 3, below. The locations of the cross sections along with the borehole locations are shown in Fig. 1.

III. Practical Considerations on the Pile Types Adopted for This Site

CFA piles are a non-displacement form of cast in situ piles. Theoretically, the volume of spoil coincides with the volume of the auger. DD piles on the other hand seek to laterally displace the soil and consequentially improve the latter, with the advantage of having virtually no spoil. Fig. 4 below shows schematically the conceptual difference between CFA and DD penetration. A summary of the most popular displacement heads is presented in Fig. 5.

S. Pirrello is with Keller Australia, Sydney, NSW 2100 Australia (phone: +61 2 8866-1100; e-mail: stefano.pirrello@ keller.com.au).

Fig. 1 Test locations

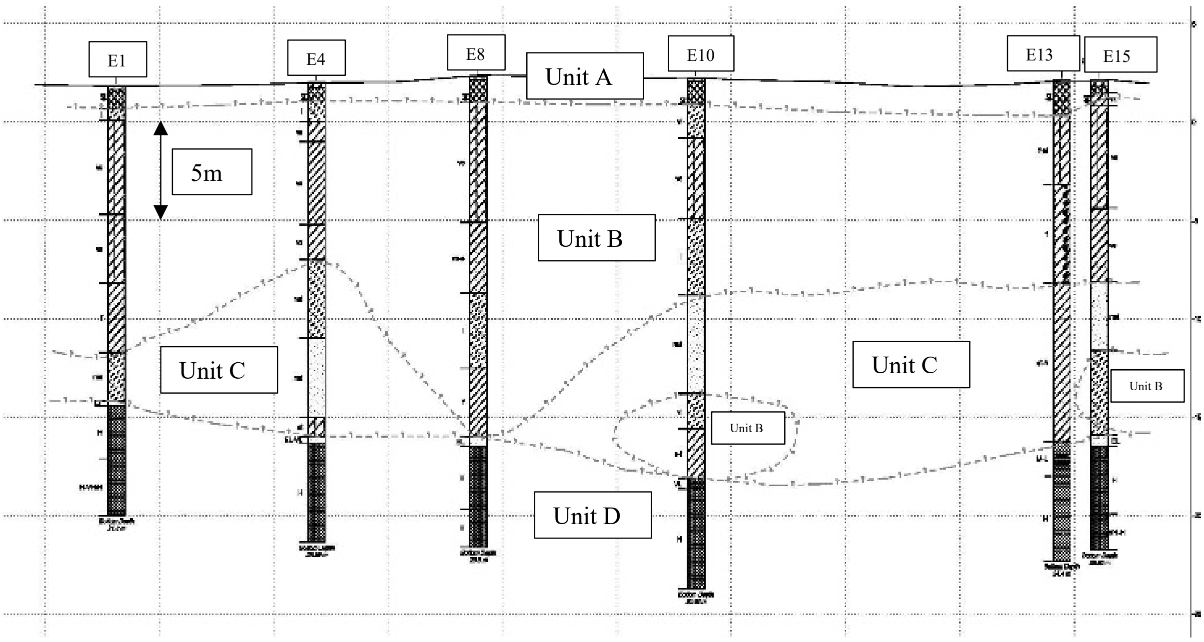

Fig. 2 Cross-section A-A'

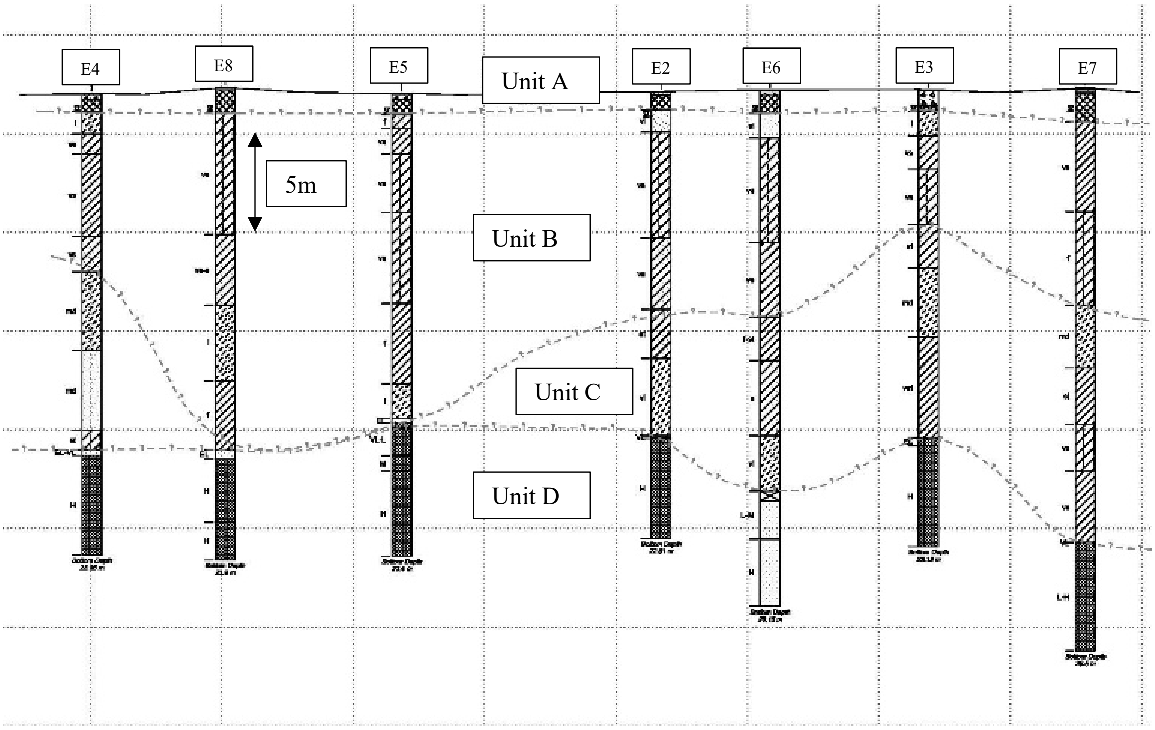

Fig. 3 Cross section B-B'

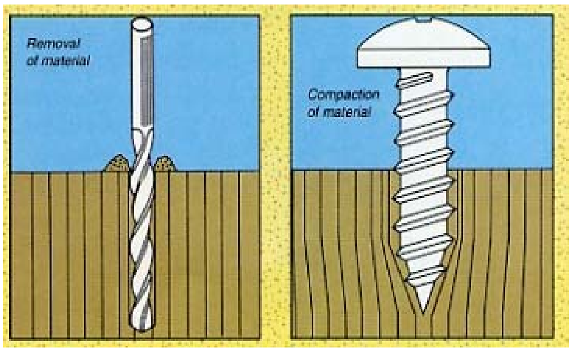

Fig. 4 Wood drill vs screw analogy as in [8], [9]

A detailed explanation of these two piling systems is outside the scope of this paper with the focus being the advantages and disadvantages of the respective pile types. CFA piles can be used in most soil conditions. Modern piling rigs have sufficient power and torque that can deliver a reasonably high production rate while achieving considerable depths with rock socket lengths often sufficient to satisfy design requirements. On the other hand, DD piles have the limitation of not being able to displace the soil when the soils are too stiff/hard or too dense. DD augers can be successfully employed when there is limited thickness of stiff soil over bedrock. For this project, a stiff clay contour map was developed for this site, aiding with the assessment of identifying areas where one piling technology could be used over the other. This assessment was carried out using the available geotechnical data presented in Figs. 2 and 3. The results of the inferred Unit C contour map are presented in Fig. 6.

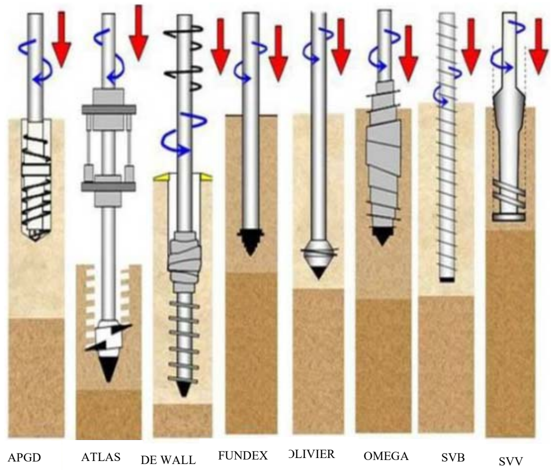

Fig. 5 Most popular full displacement augers [3]

As can be seen from the contour map shown above, the area with no or little stiff clay (zero-meter contour line) is limited to a small area compared to the overall site area. If traditional DD heads presented in Fig. 5 were used, this would result in CFA Piles being used for 80% of the project piles because the DD piles could not achieve the required socket lengths in the rock. The high costs of spoil removal resulting from a predominantly CFA solution on this project would have a detrimental effect on project costs. As a consequence, every

ISSN: 2517-942X
Vol:11, No:9, 2017

effort was made to reduce the volume of spoil generated by piling. A displacement auger capable of drilling through thicker layers of stiff clay and founding in the rock extends the chances of its applications within this site. Fig. 7 shows the modified DD head custom made for this site.

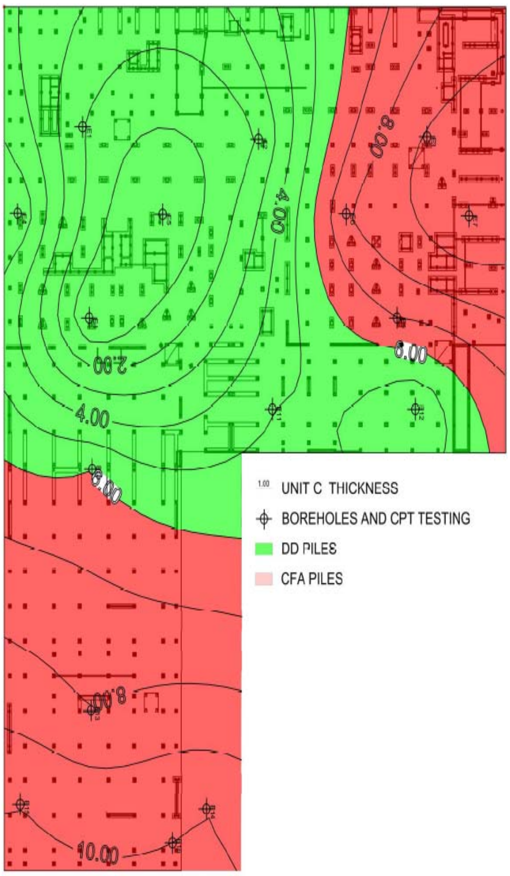

Fig. 6 Unit C thickness contours map

Fig. 7 Keller's modified DD drilling head

The DD head adopted was a hybrid of the Omega and De Waal heads, utilizing an "Omega-type" head with a 3-m to 6-m auger extension. This guaranteed extending the application of the DD piles within layers up to 6 m of Unit C, while still delivering the required designed minimum pile sockets of up to 1-m length in the MS Sydney Sandstone. Pile design and settlement estimate were carried out in accordance with limit state procedures as outlined in the Australian Standard AS2159-2009 [1], which requires piles to fulfil both serviceability and strength requirements. The geotechnical reduction factor adopted for low and high redundancy piles was 0.65 and 0.71, respectively. The foundation design for this project comprised the design and construction of over 800 piles, the final scheme comprising 55% CFA piles and 45% DD. Pile diameters were 600 mm and 750 mm, with all piles being socketed up to 1 m into MS Sandstone to meet design requirements. The proposed development extends to an area greater than 24 football fields. Different design zones were developed, targeting any significant variability of the soil and bedrock profile. Differences such as thicknesses of soft soil (i.e. Unit B), Unit C, and levels of the MS Sandstone had to be assessed and considered in the design. A typical geotechnical soil profile for this site is presented in Table I.

TABLE I
GEOTECHNICAL PARAMETERS

Geotechnical Unit	Unit Thickness (m)	Base of Unit RL(m) AHD	Ultimate Shaft Adhesion (kPa)*	Ultimate End Bearing (kPa)*
B	10	-8	15	-
C	8	-16	40	-
D (LS Sandstone)	2	-18	120	-
D (MS Sandstone)	N/A	N/A	450	30000

Note: (*) LS and MS Sandstone parameters were based as in [4], [5]

As all piles were founded in bedrock, any potential ground improvement in the case of the DD piles was ignored, so technically these piles were designed as CFA piles. Structural capacity of the piles was dictated by Clause 5.3.4 AS2159-2009 [1], which states that for partially reinforced piles the maximum structural load allowed is essentially 0.3f'cAg. Table II shows the maximum design loads for 600-mm and 750-mm piles.

TABLE II
MAXIMUM STRUCTURAL DESIGN FOR PARTIALLY REINFORCED PILES ON THIS SITE

Diameter (mm)	f'c(MPa)	Eds(kN)*	Ed(kN)*
600	65	3040	4140
750	65	6667	9000

Note (*) Eds = serviceability load; Ed = Factored design load. Eds ≈ Ed/1.35

Negative skin friction, assessed to be 20 kPa, resulting from the consolidation of the compressible layer, Unit B, provided an additional load to be considered for both the structural pile design and for serviceability assessment. Pile settlement predictions under ultimate and working load were assessed with aid of the commercial program RATZ, written and developed as in [6].

Table III provides the settlement estimates, including both pile shortening and negative skin friction effects, for the piles loaded in accordance with Table II.

TABLE III
SETTLEMENT ESTIMATE UNDER SERVICEABILITY

Diameter (mm)	f'c(MPa)	Eds(kN)*	d(mm)
600	65	3040	6
750	65	6667	8

IV. CONSTRUCTION ASPECTS QA & QC

Two major construction aspects were concerning the designers while opting for the revised DD head. These were:

1. The uncertainties related to the performance of this new auger drilled through an unusually thicker Unit C layer. In particular, in case of slow penetration, a combined effect of a prolonged penetration and high torque could lead to the "side loading" (also known as "flighting" or "soil decompression") effect for the non-cohesive component of Unit C. This would make the soil surrounding the auger collapse onto the auger.
2. Consideration was also given to the potential within Unit C layer of built up pore pressures due to the displacement effect within the clay potentially causing squeezing of newly formed adjacent pile shafts.
3. Another aspect that demanded care was the possibility of potential necking in the very soft clay layer – UNIT B, where bulges could be generated despite the concrete oversupply. Pile bulging would potentially cause increase in the downdrag forces on the pile [2] within UNIT B.

The CFA and DD piling rigs were all equipped with on-board computers (PL3000) which allow the site personnel and the designer access to continuous monitoring of the all aspects of pile construction in real time. This my information includes:

- Boring rate (m/min);
- Penetration rate(mm/rev);
- Extraction rate (m/min);
- Torque (% of maximum);
- Concrete pressure (bar);
- Concrete oversupply (% above theoretical) and Ideal pile shape

The penetration rates and torque from the PL3000 records served to provide confirmation of rock head levels and socket lengths. This information is essential for a quick assessment of the constructed pile. However, care needs to be taken in the way the data are interpreted especially when a reduced section of the flow profile due to a change in layer's stiffness for example from a low to high stiffness is misinterpreted with a pile necking. This would simply be the case of a larger section back to a nominal section [7]. The analysis of the above information, along with the validation of the rationalised design profile assumed, provided the designers with the confidence required for successful penetration of the Unit C layer in a timely manner.

V. EFFECTS OF DD INSTALLATION

The opportunity was taken to carry out an additional geotechnical investigation involving CPTu probes aimed to assess pore pressure prior and after installation of DD piles. The CPTu's were carried out prior and after construction of piles. The results showed that the water pressure rarely increased due to the installation process and where increases occurred; those increases were modest and always less than the concrete pressure during which the piles were built. Fig. 8 shows the CPTu testing arrangement prior to and after installation of the piles.

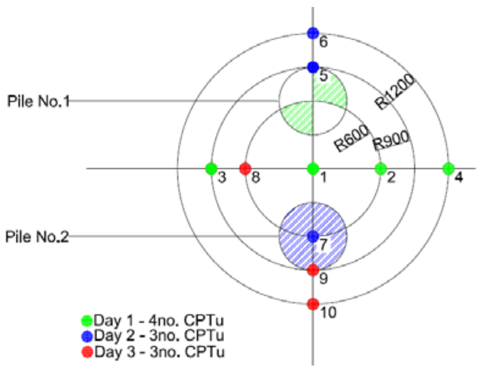

Fig. 8 Keller's CPTu test rational

CPTu tests were positioned at concentric distances of 1d, 2d, and 3d from the middle of the twin pile configuration. The testing sequence was carried out as follow:

- Day 1 – #4 CPTu test were carried out (i.e. 1 to 4) prior installation of the first pile (i.e. green pile);
- Day 2 - #3 CPTu were carried out (i.e. 5 to 7). Then the second pile (i.e. blue pile) was installed;
- Day 3 - # CPTu were carried out (i.e.8 to 10).

Typical CPtu results prior and after the installation are given in Fig. 9.

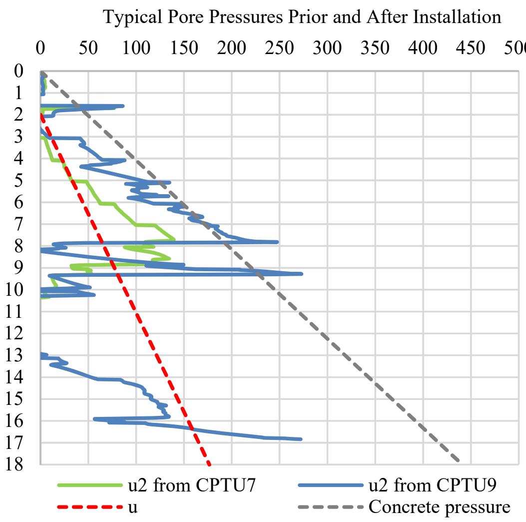

Fig. 9 Pore pressure results from CPTu7 and CPTu9, prior and after installation

The increase of pore pressure after installation can be appreciated in CPTu7 at 8-m and 9.5-m depth. However, this increase is very close to the concrete head pressure at the same depth in the worst case or less, and therefore the integrity of the newly formed pile shafts is not a concern. A substantial increase in pore pressure with depth greater than the concrete head pressure would be a potential indicator of damage to the pile, e.g. pile "necking" at the depths where the pore pressures exceeded concrete pressures.

VI. Pile Testing

Dynamic pile testing was carried out on 18 piles, of which eight of the more heavily loaded piles are presented. Dynamic pile testing is widely accepted in the Australian piling industry as an alternative to conventional static tests. The test is regarded as being sufficiently accurate but at a much lower cost than alternative load tests and regarded as an excellent tool for assessing pile performances economically. The total number of tests performed represented 2.3% of the total number of piles installed, but this was considered satisfactory given previous extensive experience from adjacent sites, supplemented by data from the PL3000 monitoring that confirmed socket length requirements. Unit pile loads and settlement criteria were satisfied in accordance with AS2159-2009 [1]. Settlement due to negative skin friction was also considered in accordance with clause 4.6.3 of AS2159-2009 [1]. Analysis of wave equation did not highlight any necking or bulging of the piles within UNIT B, so no increase on the already estimated negative skin friction on the piles was assessed.

All piles tested were production piles socketed into the sandstone bedrock. A potential disadvantage of dynamic testing of reasonably highly loaded piles is that the hammer drop height must be sufficient to deliver enough energy on the pile to mobilise and prove the required pile resistance whilst not damaging the pile. None of the piles were damaged during the tests as the developed stresses within the piles due to the dynamic impact were continuously monitored. Test signals were recorded on site by means of PDA monitoring and testing devices. Table IV reports the test results. Pile test analyses were carried with CAPWAP, from which the interpreted load displacement curves at construction stage are given in Figs. 10 and 11.

TABLE IV
SUMMARY OF DYNAMIC PILE TEST RESULTS

Pile ID	ϕ (mm)	Pile Type	Pile Depth (m)	N* (kN)	Pg (kN)	Eds (kN)	δ_1	δ_2
4077	750	DD	20	10984	9951	5110	6	15
4052	750	DD	19	11674	9637	4940	6	15
C74	750	CFA	19	12898	12676	6667	8	17
3064	750	DD	20	11447	10098	5150	6	17
2131	750	DD	25	9961	4435	2110	3	17
2111	600	CFA	20	8425	5595	3040	6	18
1093	600	DD	20	7350	3775	1900	3	16
1150	600	DD	24	8220	1929	960	2	20

Note: E_{ds}-Design Serviceability Load. P_g-Maximum Test Load. N*-Total Mobilised Resistance. d_1-Settlement under SLS. d_2-Settlement under N*.

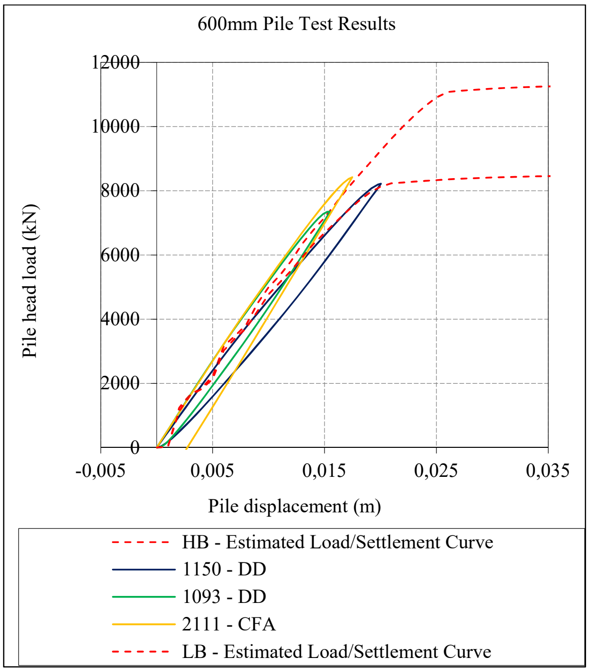

Fig. 90 600-mm estimated vs test load settlement curve

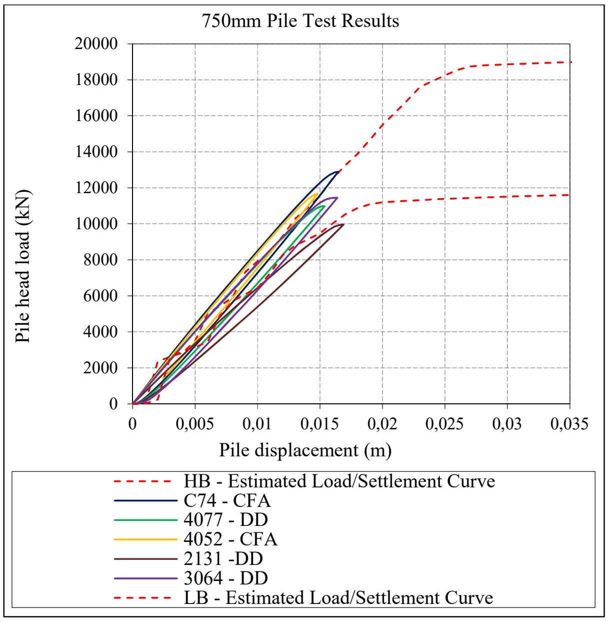

Fig. 101 750-mm estimated vs test load settlement curve

VII. Summary and Conclusions

Technical differences between CFA and DD piles were briefly discussed. The main focus of this paper is to explain the rationale of selecting one method over the other and its application and limitations within different ground conditions.

DD pile augers were adopted to offset construction limitations, so they could be employed in stiff soil layers. The quality control during construction was also discussed focusing on two major aspects such as "side loading" and "pore pressure increase". Design load settlement predictions were validated by means of comparison with PDA testing carried out on the same piles.

The work was completed on time and on budget.

ACKNOWLEDGEMENT

The author would like to thank all colleagues involved in the project from conception to delivery. A special mention goes to the construction team on delivering a state of the art product, which enabled designers to push the design envelope.

REFERENCES

[1] Australian Standard, AS2159-2009. Piling- Design and installation.
[2] Brown, D.A, 2005. Practical considerations in the selection and use of continuous flight auger and drilling displacement piles. American Society of Civil Engineers, ASCE J. Geotechnical special publication No.132, pp.1-11.
[3] Larish, M. D, 2014. Behaviour of stiff, fine-grained soil during the installation of screw auger displacement piles. PhD Thesis, The University of Queensland, Australia.
[4] Pells, P. J. N, Dougals, D. J. Rodway, B., Thorne, C., McMahon, B. K., 1978. Design loadings for foundations on shale and sandstone in the Sydney region. Australian Geomechanics J. Vol.G8, pp. 31-39.
[5] Pells, P. J. N., 1998. State of practice for design of socketed piles in rock. Proceedings of the 8th A.N.Z. Conference on Geomechanics, Hobart, pp. 307-326
[6] Randolph, M. F., 1994. RATZ load transfer analysis of axially loaded piles, program manual.
[7] Slatter, .J, 2006. Recent developments in continuous flight auger (CFA) and cast-in-situ displacement screw piling in Melbourne. Australian Geomechanics J. Vol. 41, No. 3, pp. 17-24.
[8] Van Impe, W.F., 1988. Considerations on auger pile design. Proc. of 1st 2nd Int. Conference on Deep Foundations on Bored and Augered Piles, Ghent, Balkema.
[9] Van Impe, W. F., 1994. Influence of screw pile installati.on parameters on the overall pile behaviour. Workshop 'Piled Foundations: Full Scale Investigations, Analysis and Design', Naples.