The paper was published in the proceedings of the 11th Australia New Zealand Conference on Geomechanics and was edited by Prof. Guillermo Narsilio, Prof. Arul Arulrajah and Prof. Jayantha Kodikara. The conference was held in Melbourne, Australia, 15-18 July 2012.

Comparison of Pile Shaft Capacity Predictions vs Test Results for Different Construction Methods and Different Construction Time

T. D. McWilliam, BEng (Hons) MEng.Sc

Coffey Geotechnics Pty Ltd, 8/12 Mars Road, Lane Cove West NSW 2066; PH (02) 9911-1000; FAX (02) 9911-1001;
email: tristan_mcwilliam@coffey.com

ABSTRACT

This paper presents a comparison of pile shaft capacity predictions and dynamic load test results for cast-in-situ bored piles constructed with temporary casing, permanent casing, with bentonite support or with no support. Pile shaft capacity predictions were based on parameters derived from borehole information, insitu and laboratory test results. Ground conditions comprised alluvium and residual soils overlying argillite bedrock. The piles were designed to be self supporting open bored piles with temporary casing, as necessary. However to overcome problems with side wall collapse during construction, bentonite support or permanent casing was proposed. This lead to concerns regarding the appropriateness of the design parameters from bentonite cake on pile sidewalls. Pile load testing was carried out using high strain dynamic test methods on piles constructed using casing and bentonite. The results of the tests for the various support conditions and construction methodology are discussed with reference to the adopted design values. The case study illustrates that where constructed with appropriate controls, bentonite support does not reduce the mobilised skin friction when compared to other methods of construction. Where poor construction techniques were employed and where construction time was excessive, poorer than expected performance may result.

Keywords: Pile, Support, Bentonite, Casing, Testing, Construction Time.

1 INTRODUCTION

This work was completed as part of a road upgrade project within New South Wales. The project included the construction of bridges at 20 locations, predominantly with bored cast-in-situ reinforced concrete piles at the abutments and piers. The piles were 0.9m to 1.8m in diameter with length/diameter ratios typically between 15 and 30. Initially, the piles were proposed to either be open bored, or use temporary casing through soils likely to undergo sidewall collapse during construction.

As part of the design verification process, dynamic load testing of the piles was initially proposed to assess the ultimate load carrying capacity of the piles and to verify the design process adopted for assigning shaft friction and end bearing values. During construction of some of the early bridge foundations, difficulties were encountered with construction of some of the piles. A significant amount of slumping of the sidewall material was occurring along the pile shaft, in residual clays. The initial expectation was that only minimal slumping of material into the pile bore would occur in such material. Alternative construction techniques were proposed, including using permanent casing and using bentonite to provide temporary support during construction. As a result of the alternative construction methodologies adopted, additional pile testing was undertaken to verify that the parameter assessment was still valid. This paper presents the findings of the testing undertaken at three bridge locations, focussing on pile shaft friction.

2 GEOLOGICAL AND GROUNDWATER CONDITIONS

The ground conditions at the three sites studied generally comprised highly over-consolidated stiff and very stiff residual soils overlying extremely and distinctly weathered argillite bedrock. The consistency of the soil generally improved with depth. The argillite was typically very low and low strength with competent bedrock generally lying 25m to 30m below surface level. The approximate depth below surface where groundwater was encountered was at 8m for Site 1, 14m for Site 2 and 1m below base of fill for Site 3. Table 1 provides a summary of the ground conditions.

Table 1: Summary of Ground Conditions – Depth Range Where Material was Encountered

Material Description	Site 1		Site 2	Site 3	
	Abutment A	Pier 1	Abutment A	Pier 1	Abutment B
FILL: Stiff and Very Stiff Clay	0m to 3m	0m to 2m	-	0m to 1m	0m to 5m
CLAY: Firm	-	-	-	-	5m to 7m
CLAY: Stiff	3m to 5m	2m to 13m	-	-	-
CLAY: Very Stiff and Hard	5m to 11m	13m to 22m	0m to 10m	1m to 8m	7m to 12m
BEDROCK: Extremely and Distinctly Weathered Argillite, Extremely and Very Low Strength	From 11m, base not proven	From 22m, base not proven	From 10m, base not proven	From 8m, base not proven	From 12m, base not proven

3 PARAMETER ASSESSMENT

The ultimate skin friction values for soils and extremely weathered rock were assessed using a number of different correlations. Equation 1 proposed by Decourt (1995) was adopted for cohesive soils with SPT data. For cohesive soils where no SPT data was available, equation (2) proposed by Woodward and Boitono (1961) was adopted. Where the foundation comprised a matrix of distinctly weathered argillite and soil strength material, a higher limiting value of ultimate skin friction was adopted. Typically, a value of 75kPa or 100kPa was adopted depending on the proportion of distinctly weathered argillite within the clay matrix. Where the foundation comprised predominantly distinctly weathered or better argillite, equation (3) proposed by Zhang and Einstein (1998) was adopted.

$$f_s = a[2.8N_{60}+10] \leq f_{sl} \text{ kPa} \qquad (1)$$

f_s= ultimate skin friction, N_{60} = corrected SPT N value
a = 1 for displacement piles in all soils, and non-displacement piles in clays;
= 0.5 to 0.6 for non-displacement piles in granular soils
f_{sl} = limiting value of ultimate skin friction = 60kPa for bored piles in clay

$$f_s = \alpha c_u \text{ kPa} \qquad (2)$$

f_s = ultimate skin friction α = reduction factor between 1.0 and 0.4
c_u = undrained shear strength.

$$f_s = a_s[q_u]^b \text{ MPa} \qquad (3)$$

f_s = ultimate skin friction, a_s = 0.2 and b = 0.5
q_u= rock unconfined compressive strength (UCS) MPa

4 CONSTRUCTION

4.1 Summary

Table 2 provides a summary of the construction details for the piles at each of the sites. It presents the type and depth of casing used at each location, whether or not bentonite was used for temporary support and the construction time taken from commencement of boring to placement of concrete. The piles analysed at these locations were designed with a diameter of 0.9m.

Table 2: Summary of Pile Construction Details

Site	Location	Pile Reference	Pile Length	Casing Used	Casing Depth (m)	Bentonite Used	Construction Time (hr)
Site 1	Abutment A	NA-T	15m	Temp.	9	No	47
		NA-T2	14m	Temp.	8	No	6
	Pier 1	N1-T	23m	Temp.	No Record	No	20
		N1-5	24m	Temp.	15[a]	No	51
Site 2	Abutment A	A-1	27m	Temp.	6	Yes	9
		A-2	26m	Perm.	26[b]	No	57
Site 3	Pier 1	P-1	35m	Temp.	12	Yes	31
	Abutment B	B-3	28m	Perm.	13[c]	Yes	51

[a] casing initially installed to a depth of 6.5m. Following collapse of the sidewall material during excavation, the casing was subsequently installed to a depth of 14.5m.
[b] casing initially installed to a depth of 4.2m. Following collapse the sidewall material during excavation, the casing was subsequently installed to a depth of 26m. Due to difficulties removing the 26m length casing, it was left as permanent casing.
[c] loss of bentonite at a depth within the uppermost 10m resulted in increasing the length of the casing from 7m to 13m.

4.2 Use of Bentonite

Table 3 presents the acceptance criteria adopted for the use of bentonite for temporary support.

Table 3: Properties for Bentonite suspensions

Property	Stages		
	Fresh Bentonite	Working Bentonite	Concreting Bentonite
Density (g/ml)	≤ 1.10	≤ 1.25	≤ 1.15
Marsh value (s)	32 to 50	32 to 60	32 to 50
Fluid loss (ml)	≤ 30	≤ 50	N/A
pH	7 to 11	7 to 12	N/A
Sand Content (%)	N/A	N/A	≤ 2
Filter cake (mm)	≤ 3	≤ 6	N/A
Frequency of testing	Before use or at least once a day	Before use and once during pile excavation	Before concreting pile

5 PILE TESTING

High strain dynamic pile testing was performed on each of the piles listed in Table 2 using a Pile Driving Analyser (PDA). The interpretation of the PDA data was undertaken by others. The interpreted mobilised skin friction resistance values are presented in Section 6 for comparison with the design skin friction values.

6 TEST RESULTS

Figures 1 to 4 present a comparison of the mobilised shaft friction with the design line for the piles tested at the different sites.

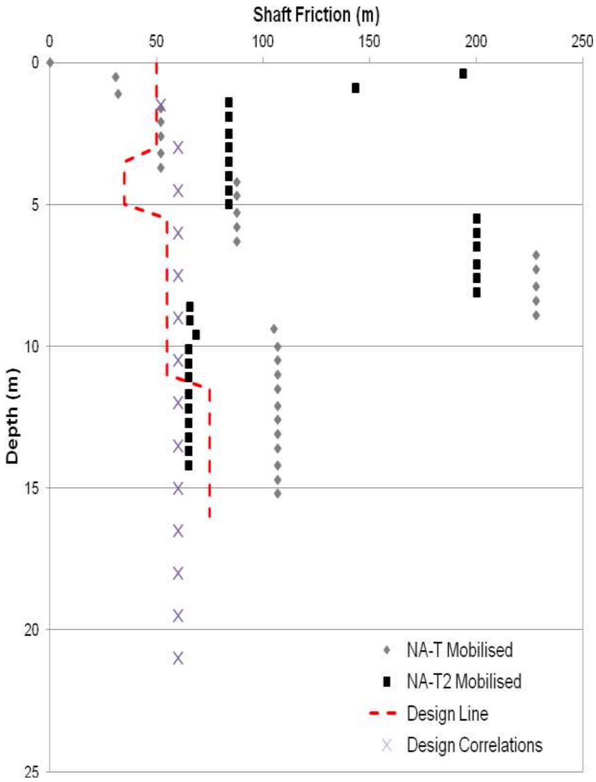

Figure 1: Comparison of mobilised shaft friction with design values at Site 1 Abutment A

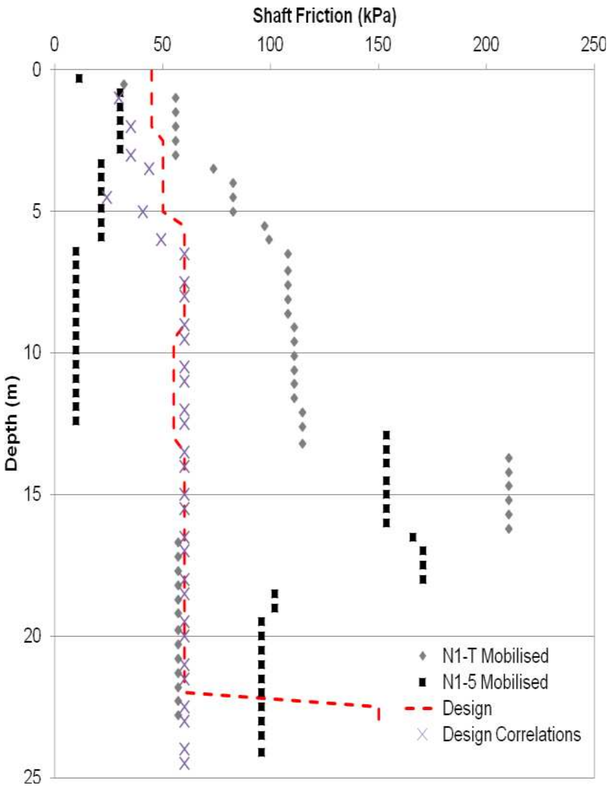

Figure 2: Comparison of mobilised shaft friction with design values at Site 1 Pier 1

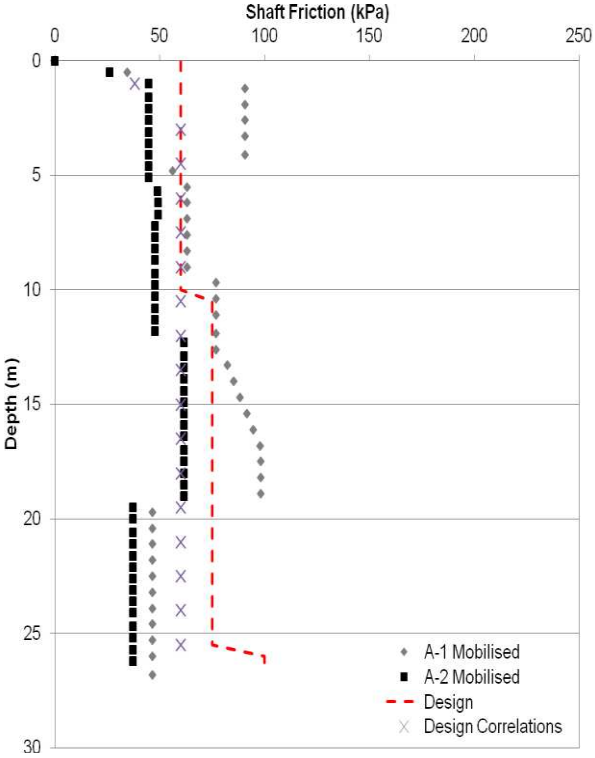

Figure 3: Comparison of Mobilised shaft friction with design values at Site 2 Abutment A

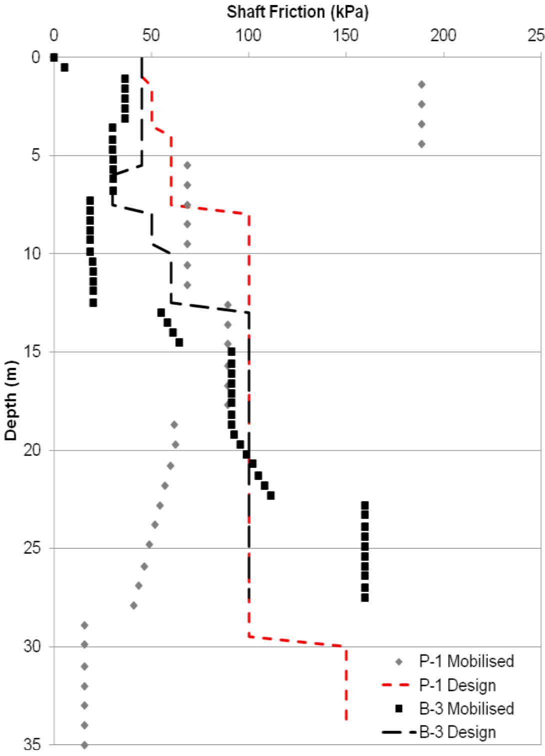

Figure 4: Comparison of Mobilised shaft friction with design values at Site 3 Abutment B and Pier

Figure 1 indicates the mobilised resistance values in both piles significantly exceeded the design values. At a depth of about 5m, there is a significant increase in mobilised resistance, possibly associated with the change from stiff to very stiff clay. Below a depth of 9m, it is inferred that the mobilised resistance is not near the ultimate resistance, as the hammer was not of sufficient capacity to impart the required energy to mobilise the ultimate resistance.

Figure 2 provides a comparison between 2 piles constructed using temporary casing. Pile N1-T was constructed as expected, with no evidence of sidewall collapse during construction. The mobilised resistance in N1-T exceeded the design values. At a depth of 13m, there was a significant increase in resistance, at the inferred transition to very stiff and hard clay (as observed at Abutment A). Below a depth of 17m, it is inferred that sufficient energy has not been imparted on the pile to mobilise the ultimate resistance. Sidewall collapse was observed during excavation of N1-5. Initially, the casing was installed to a depth of 6.5m. Following augering to 14m, sidewall collapse was observed, so the casing was installed to 15m depth. It is inferred that the collapse occurred due to adverse groundwater condtions, the consistency of the material and suction during auger extraction. Along the uppermost 13m of N1-5, where the soil disturbance was greatest, the mobilised friction is significantly lower than that assumed in design. Below this zone, where sidewall collapse was not observed, the mobilised resistance values were of similar magnitude to pile N1-T. Below a depth of 18m, the mobilised resistance is not near the ultimate value due to the capacity of the hammer. A further point of comparison is that the poorer performing pile, N1-5, took an additional 30hr to construct compared with the 20hr required for N1-T.

Figure 3 provides a comparison between a pile constructed using bentonite for support during excavation (A-1) with a pile constructed using permanent casing (A-2). A-1 was bored and concreted within 9hr. The mobilised resistance values were similar to or exceeded the design values adopted. Pile A-2 was initially bored to a depth of 23.5m with about 4m of temporary casing. As a result of continued collapse of material from the sidewalls, the casing was extended to a depth of 26m. Due to the soil disturbance from the observed collapse, and the time taken for construction of the pile (57hr), the mobilised friction values were about 75% of the design ultimate resistance values. It is assessed that below a depth of 19m, the mobilised resistance is not near the ultimate resistance due to insufficient hammer capacity.

Figure 4 provides a comparison of two piles constructed using bentonite for temporary support, at the same bridge location, with their respective design lines. In the uppermost 5m of P-1, the mobilised resistance was about 4 times the design resistance. This may be the result of an enlarged section of pile. Between 5m and 12m, the mobilised resistance of 68kPa is greater than the design value of 60kPa in the very stiff clay, however is lower than the 100kPa design value below a depth of 8m. It is likely that the depth of weathering at this location was greater than expected in the geotechnical model resulting in the extremely and distinctly weathered material being encountered at a greater depth than anticipated. At B-3, construction difficulties occurred during boring of the uppermost 13m of pile, resulting in lower mobilised shaft resistance than design. When the boring reached about 12m depth, a significant loss of bentonite was observed. Review of the construction history of the embankment on which the abutment was to found indicated that a permeable bridging layer had been constructed over the firm ground. The casing was subsequently extended to a depth of about 13m to penetrate the highly permeable bridging layer and prevent further loss of bentonite. It is assessed that the lower than expected mobilisation in the uppermost 13m was the result of auger advancement and soil disturbance in this zone.

7 DISCUSSION

Figure 5 and Figure 6 provide graphs comparing the mobilised shaft friction with the design shaft friction values, classified by support type and time of construction respectively. Where the mobilised resistance is significantly lower than the expected ultimate resistance due to lack of hammer capacity, the values have been excluded. Furthermore, the values in the uppermost 1m have been excluded due to the potential for surface disturbance. In both figures, the dotted line indicates where the mobilised resistance equals the design values. Therefore, data points above the line indicate better than expected performance.

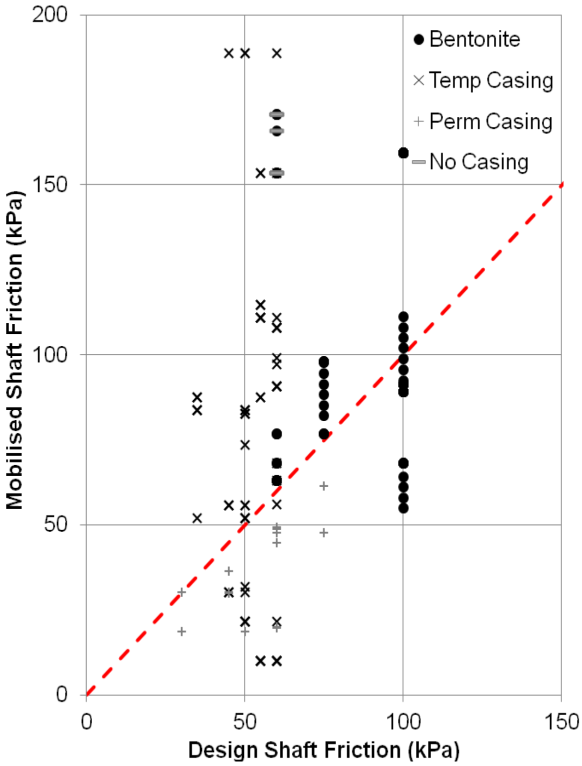

Figure 5: Comparison of mobilised resistance vs design shaft friction values assessed by support type

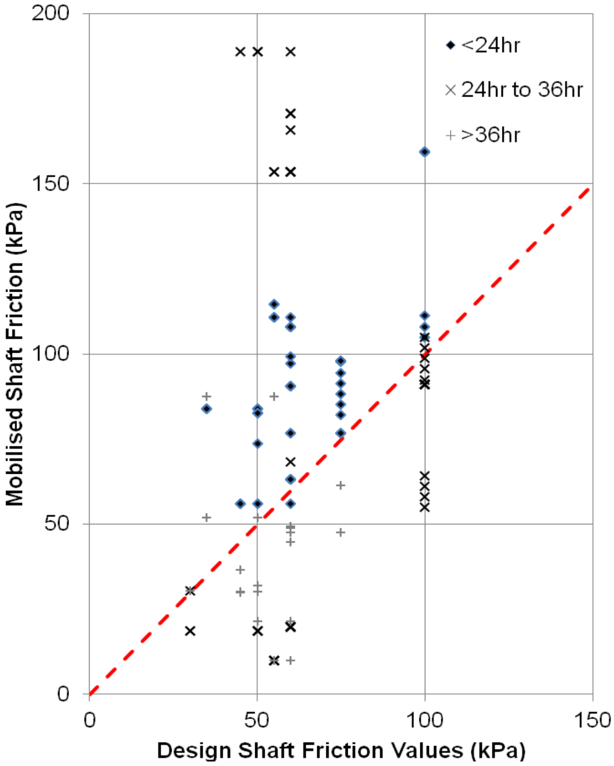

Figure 6: Comparison of mobilised resistance vs design shaft friction values assessed by time of construction

7.1 Performance Assessment by Support Type

Figure 5 shows the mobilised resistance for each of the support types adopted on the project. It indicates the following:

- Piles constructed with bentonite generally resulted in performance better than or close to design. Where performance was significantly less than design, construction difficulties and time of construction were likely to be contributing factors.
- Piles constructed with temporary casing generally performed better than design. Where performance was poor, it was generally associated with sidewall collapse and auger advancement ahead of installation of the casing.
- Piles constructed using permanent casing generally performed poorer than design. This was generally the result of construction challenges requiring the casing to be left in place following loss of bentonite or inability to remove the casing.
- Sections of pile constructed without casing generally performed better than design due to the relatively good ground conditions in which this method was able to be used.

7.2 Performance Assessment by Time of Construction

Figure 6 shows the mobilised resistance classified by time of construction from commencement of boring to completion of concreting. The results indicate that where piles were completed within 24 hours, performance was generally equal to or better than design. When pile completion occurred between 24 hours and 36 hours, performance sometimes met design expectations, but there was an increased likelihood of underperformance. Where construction exceeded 36hrs, the likelihood of performance not meeting design increased significantly.

8 CONCLUSION

This paper has assessed the mobilised resistance at eight piles under dynamic loading conditions relative to their design. The piles were constructed with temporary casing, permanent casing, bentonite or no casing for temporary support. Pile construction time took as little as 6 hours and as long as 57 hours. Key findings from the assessment are as follows:

- The design correlations adopted were generally appropriate for the pile type and ground conditions, irrespective of whether temporary casing, permanent casing or bentonite was used to provide support during excavation.
- The use of bentonite for temporary support does not necessitate the reduction in ultimate shaft friction values, provided appropriate construction control is adopted.
- Very stiff overconsolidated soils may not be self supporting during bored pile construction where adverse groundwater conditions exist.
- Where difficulties were encountered during construction, such as sidewall collapse or auger advancement ahead of casing advancement, the ultimate shaft resistance values reduced below expected design values.
- Time of construction is a key factor in determining the ultimate shaft resistance along the pile. For a pile construction time less than 24hrs, the mobilised resistance generally met or exceeded expectations. Where construction of a pile extened across two days, then the ultimate shaft resistance in some circumstances was affected. Where construction of a pile extended into a third day, the ultimate shaft resistance generally did not meet design expectations.

9 ACKNOWLEDGEMENTS

The author would like to thank Mr Patrick Wong, Professor Harry Poulos and Mr Peter Waddell for their assistance in the preparation of this paper.

REFERENCES

Decourt, L. (1995). “Prediction of load-settlement relationships for foundations on the basis of the SPT-T Ciclo de Conferencias Intern. “Leonardo Zeevaert”, UNAM, Mexico, 85-104.

Zhang, L. and Einstein, H. (1998). “End bearing capacity of drilled shafts in rock.” ASCE Jnl. Geot. Eng., Vol. 124 (7), 574-584.

Woodward, R. and Boitono, J. (1961). “Pile Loading Tests in Stiff Clays”. Proc. 5th Int. Conf. SM&FE, Vol.2, 177)