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Performance of large diameter bored piles in varying degree of weathered sedimentary and metamorphic rock formations

Performance des pieux forés de grand diamètre dans des degrés variables de formations rocheuses sédimentaires et métamorphiques altérées

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ABSTRACT: Large diameter cast in place bored piles of up to 1,800mm diameter with working load up to 12,000kN were successfully designed and constructed for a railway viaduct in Segamat, Johor, Malaysia. The general geological formations of the constructed bored piles are Semantan formation and Gemas formation. The lithologies of these sedimentary formations generally consist of shale, sandstone, conglomerate and metamorphic rocks. This paper aims to present the performance of preliminary test piles that were subjected to compressive axial loading via maintained load tests using kentledge or reaction pile systems. All preliminary test piles were fully instrumented using proprietary Global Strain Extensometer (GLOSTREXT) system to determine the mobilised shaft friction in overburden soil, shaft friction in varying degree of weathered rock, as well as the mobilised end bearing resistance which are later compared to common correlations and outcomes in published literatures.

RÉSUMÉ : Des pieux forés de grand diamètre coulés sur place jusqu'à 1 800 mm de diamètre avec une charge de travail jusqu'à 12 000 kN ont été conçus et construits avec succès pour un viaduc ferroviaire à Segamat, Johor, Malaisie. Les formations géologiques générales des pieux forés construits sont la formation Semantan et la formation Gemas. Les lithologies de ces formations sédimentaires sont généralement constituées de schistes, de grès, de conglomérats et de roches métamorphiques. Cet article vise à présenter les performances des pieux d'essai préliminaires qui ont été soumis à une charge axiale de compression via des essais de charge maintenue utilisant des systèmes de pieux kentledge ou de réaction. Tous les pieux d'essai préliminaires ont été entièrement instrumentés à l'aide du système exclusif Global Strain Extensometer (GLOSTREXT) pour déterminer le frottement de l'arbre mobilisé dans le sol de mort-terrain, le frottement de l'arbre à divers degrés de roche altérée, ainsi que la résistance du palier d'extrémité mobilisée qui sont ensuite comparées aux corrélations courantes. et les résultats dans les littératures publiées.

KEYWORDS: Bored pile, instrumented, rock friction

1 INTRODUCTION

In geotechnical engineering, due to the nature of soil or rock as a construction material, designers are never one hundred percent sure what they are dealing with. Especially in sedimentary formations, although rock is encountered during the subsurface investigation (SI) works, but due to the powerful drilling capacity of the machine, rock may not be encountered or recovered during the bored pile construction. As such, understanding of rock core recovery ratio (CRR) and Rock Quality Designation (RQD) will play a major role in the bored pile design.

In addition, the shaft friction resistance of the weathered rock (with low RQD and low CRR) and intact rock are other important considerations in bored pile design. Therefore, the results of four instrumented preliminary test piles are presented in this paper to compile the above-mentioned design parameters for ease of reference in future.

2 DESIGN METHODOLOGY

2.1 Subsurface information

The location of the proposed railway viaduct is underlain by Semantan and Gemas formations. The Semantan and Gemas formations are formed during the Middle to Late Triassic age and mainly consists of sedimentary rocks. The lithologies of the said formations are generally shale interbedded with pyroclastic (acidic tuff), siltstone and sandstone, conglomerate and metamorphic rocks. The overburden soil consists of silty SAND

and sandy SILT which was derived from the weathered sandstone and siltstone.

During the subsurface investigation (SI) stage, a borehole was sunk at each instrumented test pile location prior to the construction of the pile. Table 1 shows the borehole information while the simplified borelogs are presented in Section 4. The rock core photos are as shown in Figures 1 to 4. Generally, the rock core samples are slightly weathered to highly weathered.

Table 1. Subsoil and rock condition for each instrumented test pile

Instrumented Test Pile	Overburden soil thickness	Rock condition
PTP-V1A-01	21.8m	Highly weathered sandstone with RQD = 0% (21.8m to 25.6m)
PTP-V1-01	11.25m	Highly weathered quartzite with RQD = 0% (11.25m to 16.75m) and slightly to moderately weathered quartzite with RQD = 30% to 50% for the subsequent depth
PTP-V1-02	12.15m	Highly weathered quartzite with RQD = 0% (12.15m to 16.8m) and highly weathered quartzite with RQD = 14% to 18% for the subsequent depth

V2-P56

13.5m

Highly weathered siltstone
boulder with RQD = 0% (6.0m
to 11.0m) and slightly
weathered quartzite with RQD
= 72% to 93% for the
subsequent depth



Figure 1. Rock core samples for PTP-V1A-01; borehole 1DBH-101.



Figure 2. Rock core samples for PTP-V1-01; borehole 1DBH-A092.



Figure 3. Rock core samples for PTP-V1-02; borehole V1-PTP2-BH2.



Figure 4. Rock core samples for V2-P56; borehole 1DBH-A205.

2.2 Pile shaft friction

Generally, the design of the pile shaft friction in soil is based on the shaft friction factor for various Standard Penetration Tests

(SPT-N) blow counts with a limiting ultimate shaft friction resistance in soil of 250kPa.

In sedimentary and metamorphic rock formations, the definition of rock socket is less apparent due to the varying degree of weathering leading to hard soil or “soft rock” layers before reaching competent rock or “hard rock”. The contribution of shaft friction from “soft rocks” are often overlooked from a design perspective and a conservative approach where the “soft rock” shaft friction is considered similar to the limiting soil shaft friction is frequently adopted. This is understandable as the interfaces between soil, “soft rock” and “hard rock” are especially difficult to define. Moreover, the level at which rock is encountered, is frequently disputed during construction which will have contractual implications. For instance, it is not uncommon for rock levels to differ between SI stage and during the bored pile construction and this is largely due to the vast difference in machine capacities and tools between SI rigs and bored pile rigs.

For the purpose of design, the rock definition is based on the rock core recovery ratio (CRR); of a 1.5m rock core length and the Rock Quality Designation (RQD). A CRR of less than 60% and RQD of 0% is classified as highly weathered / fractured rock considered as “soft rock” and will likely be bored through as soil material during bored pile construction. The term hard soil will be used extensively although the terms hard soil and “soft rock” are interchangeable.

The Unconfined Compressive Strength (UCS) test results of the rock core samples ranges from 5MPa to 134MPa. The large variance in the UCS results is largely due to the varying degree of weathering of the tested samples. Point load tests are also carried out on 50mm diameter rock core samples to obtain the index strength, $I_{s(50)}$ and to determine the conversion factor between UCS and $I_{s(50)}$. Similarly, considerable scatter was observed in the point load tests results whereby the conversion factor was interpreted to range between 10.6 and 15.0. However, for the purpose of design and onward site verification of rock socket, a $I_{s(50)}$ of 2.0MPa was adopted leading to UCS of 21.2MPa to 30.0MPa. According to Williams and Pells (1981) the ultimate rock shaft friction is computed as 900kPa based on rock UCS of 10MPa. However, based on the author’s experience, an ultimate rock shaft friction of 600kPa was adopted.

3 BORED PILE CONSTRUCTION

The piles are constructed by drilling using auger through the overburden soil. Temporary casing ranging from 11.2m to 16.0m length were adopted for the various instrumented test piles and polymer was used as the stabilising fluid.

When the expected rock layer is reached, all three of the following criteria must be fulfilled as supervised by qualified personnel for the material to be deemed as rock:

- Change of tools to rock coring / excavation tools (i.e. core barrel as shown in Figure 5), and
- Recovered rock materials of more than 50% (measurable by weight), and
- Rock materials to achieve minimum $I_{s(50)}$ of 2.0MPa (considering size correction factor).

The stabilising fluid inside the bored hole is tested to ensure the density, viscosity, pH and sand content are within the acceptable limits prior to concreting works (Grade 40) which is carried out via tremie method.



Figure 5. Rock Coring / Excavation tools: Core barrel.

4 INSTRUMENTATION RESULTS

The instrumentation scheme of the test piles utilised the proprietary Global Strain Extensometer (GLOSTREXT) system (Hanifah *et al.*, 2006). Sonic logging access tubes are preinstalled into the pile with the reinforcement cage during casting. The vibrating wire global strain gauges (VWGS) and extensometer anchors are later lowered into the pile body via the access tubes and the setup is prepared for static load tests (i.e. via kentledge or reaction pile systems). The instrumentation levels corresponding to the reference boreholes are presented in Figures 6 to 9.

The results of the static load tests are summarised in Table 2 whereas the mobilised soil and rock shaft friction resistance for each instrumented test piles are summarised in Tables 3 to 6. Figures 10 to 13 show the mobilised soil and rock shaft friction resistance vs pile settlement. Due to the requirements of the project, piles are deemed to have failed if pile top settlement at 1-time pile working load exceeds 8mm, residual settlement after removal of 1-time working load exceeds 6.5mm and pile top settlement at 2-times pile working load exceeds 25mm. It should be noted all instrumented test piles except V2-P56 are tested up to 3-times pile working load in an attempt to achieve the ultimate capacity of the pile. Pile top settlement at 1-time pile working load for PTP-V1-01 exceeded 8mm and this was due to an anomaly in the pile body near the pile head nonetheless, the instrumentation results are unaffected.

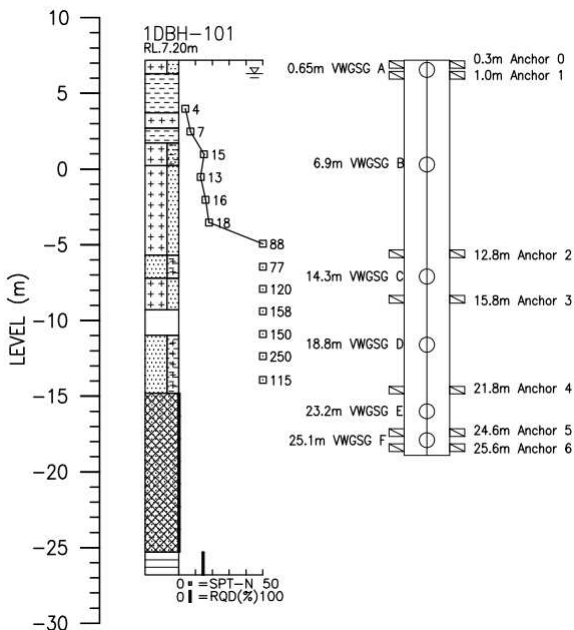


Figure 6. Instrumentation levels for PTP-V1A-01.

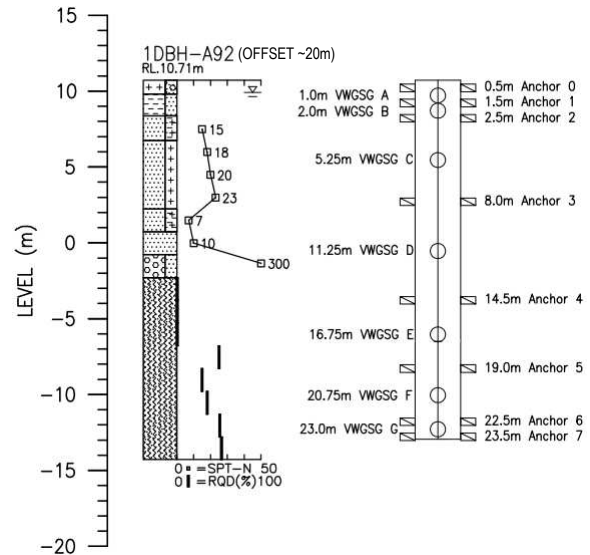


Figure 7. Instrumentation levels for PTP-V1-01.

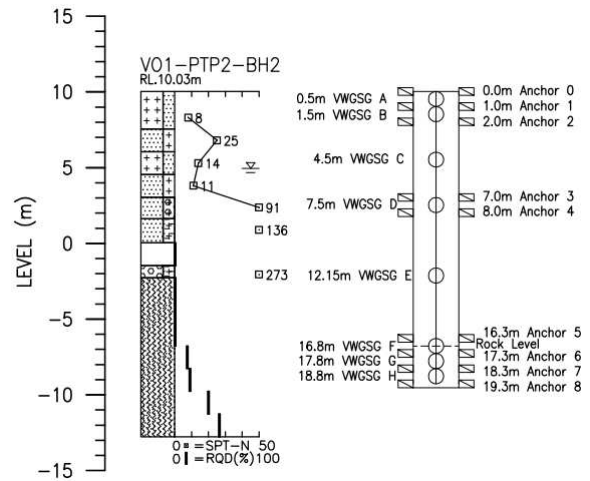
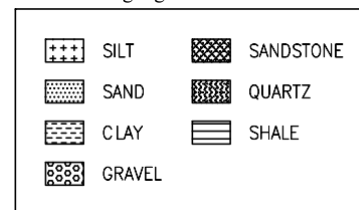


Figure 8. Instrumentation levels for PTP-V1-02.

NOTE:

The following legend shows the encountered soil/rock types



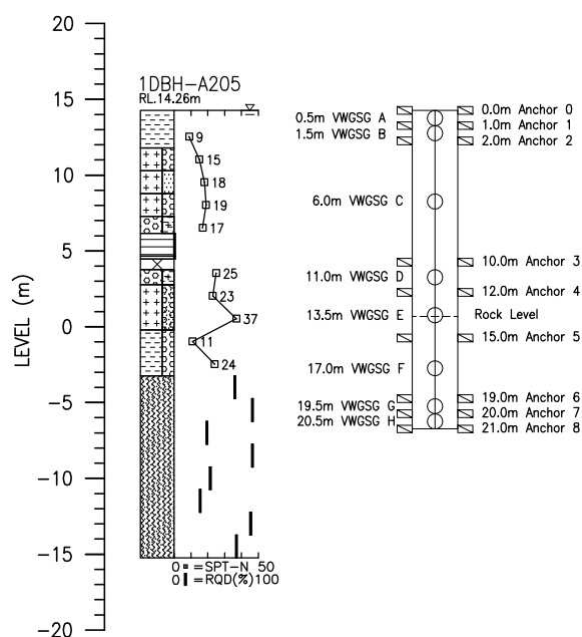


Figure 9. Instrumentation levels for V2-P56.

Table 2. Results of static load tests.

Pile Reference	Pile Diameter (mm)	Test Load (kN)	Pile Top Settlement (mm)
¹ PTP-V1A-01	1000	*5,000 #15,000	7.54 24.21
¹ PTP-V1-01	1800	*10,000 #30,000	13.43 30.58
¹ PTP-V1-02	1000	*5,000 #15,000	5.88 36.52
² V2-P56	1800	*12,000 #24,000	2.93 7.04

¹ denotes reaction static load test system

² denotes kentledge static load test system

* denotes test load at pile working load

denotes maximum test load

Table 3. Mobilised shaft friction resistance for PTP-V1A-01.

VWGSG Level	Soil / Rock condition	Mobilised shaft friction (kPa)	Back analysed shaft resistance factor
1.0m to 12.8m	Soil (average SPT'N of 10)	88.6	8.86
12.8m to 15.8m	Soil (average SPT'N of 80)	212.7	2.66
15.8m to 21.8m	Soil (average SPT'N of 150)	314.9	2.10
21.8m to 24.6m	Hard soil (weathered sandstone, RQD=0%, CRR=55%)	337.9	
24.6m to 25.6m	Hard soil (weathered sandstone, RQD=0%, CRR=55% - 59%)	265.7	

Table 4. Mobilised shaft friction resistance for PTP-V1-01

VWGSG Level	Soil / Rock condition	Mobilised shaft friction (kPa)	Back analysed shaft resistance factor
2.0m to 5.25m	Soil (average SPT'N of 15)	62	4.13
5.25m to 11.25m		72.6	4.84
11.25m to 16.75m *	Hard soil (weathered quartzite, RQD=0%, CRR=100%)	155.2	
16.75m to 20.75m *	Hard soil (weathered quartzite, RQD=30% - 50%, CRR=100%)	510.4	
20.75m to 23.0m *	Hard soil (weathered quartzite, RQD=36% - 52%, CRR=100%)	509.5	

* no rock encountered during bored pile works.

Table 5. Mobilised shaft friction resistance for PTP-V1-02

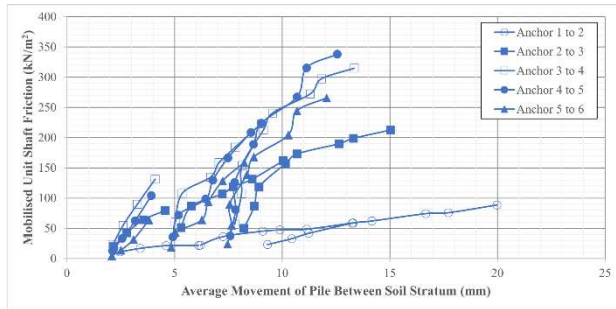
VWGSG Level	Soil / Rock condition	Mobilised shaft friction (kPa)	Back analysed shaft resistance factor
1.5m to 4.5m	Soil (average SPT'N of 16)	58.1	3.63
4.5m to 7.5m	Soil (average SPT'N of 13)	80.8	6.22
7.5m to 12.15m	Soil (average SPT'N of 115)	238.4	2.07
12.15m to 16.8m	Hard soil (weathered quartzite, RQD=0%, CRR=39%)	426.8	
16.8m to 17.8m	Rock (weathered quartzite, RQD=14.7%, CRR=97%)	826.6	
17.8m to 18.8m	Rock (weathered quartzite, RQD=18%, CRR=100%)	172.6	

Table 6. Mobilised shaft friction resistance for V2-P56

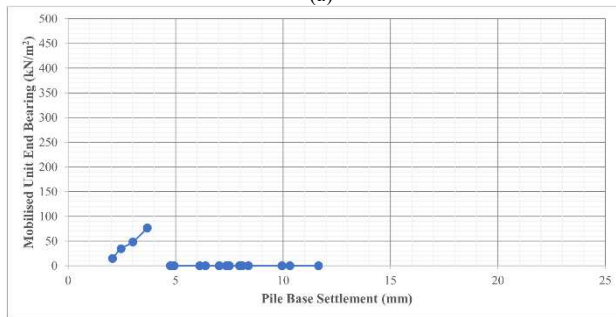
VWGSG Level	Soil / Rock condition	Mobilised shaft friction (kPa)	Back analysed shaft resistance factor
1.5m to 6.0m	Soil (average SPT'N of 15)	173	12.36
6.0m to 11.0m	Hard soil (weathered siltstone, RQD=0%, CRR=40%)	278.1	
11.0m to 13.5m	Soil (average SPT'N of 24)	93.7	3.90

13.5m to 17.0m #	Soil (average SPT ^N of 24)	363.1
17.0m to 19.5m	Rock (weathered quartzite, RQD=72%, CRR=100%)	161.4
19.5m to 20.5m	Rock (weathered quartzite, RQD=93%, CRR=100%)	145.8

rock encountered during bored pile works.

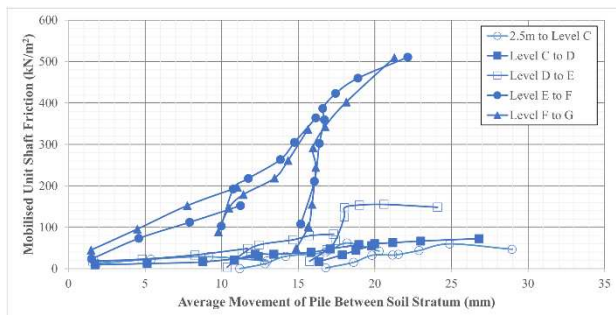


(a)

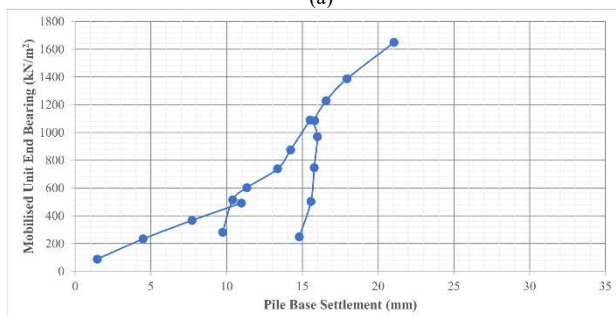


(b)

Figures 10a and 10b. Mobilised shaft friction and end bearing resistance vs pile settlement for PTP-V1A-01.

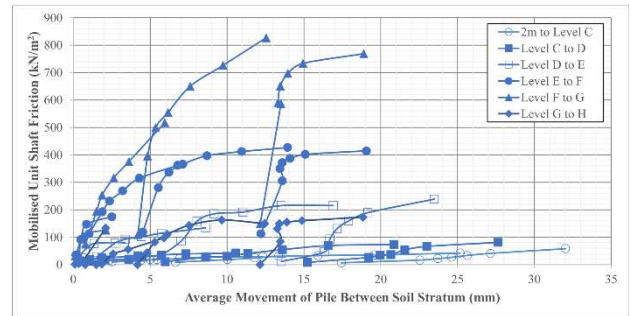


(a)

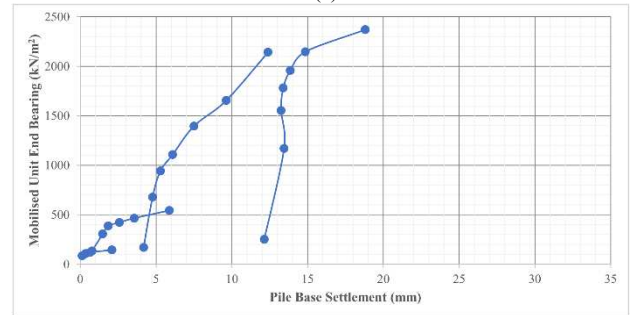


(b)

Figures 11a and 11b. Mobilised shaft friction and end bearing resistance vs pile settlement for PTP-V1-01.

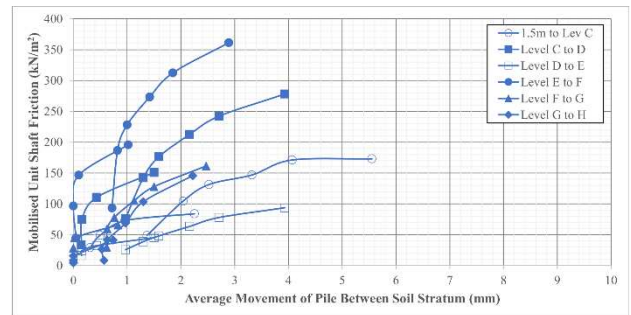


(a)

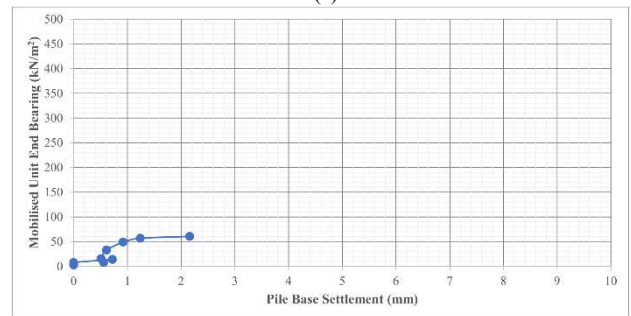


(b)

Figures 12a and 12b. Mobilised shaft friction and end bearing resistance vs pile settlement for PTP-V1-02.



(a)



(b)

Figures 13a and 13b. Mobilised shaft friction and end bearing resistance vs pile settlement for V2-P56.

5 MOBILISED SHAFT FRICTION AND END-BEARING RESISTANCE

PTP-V1A-01 is a 1.0m diameter pile tested to 15,000kN which resulted in a pile head settlement of 24.21mm. The mobilised shaft friction curve shows that the soil layers have approached ultimate shaft friction however the hard soil layers still have some reserve capacity. Contribution from the end bearing resistance is negligible and shows evidence of soft toe due to the gradual increase in pile base settlement with no development of mobilised end bearing resistance.

PTP-V1-01 is a 1.8m diameter pile tested to 30,000kN with a pile head settlement of 30.58mm. Similarly, this pile was installed into soil and hard soil layers with no rock socket as the rock definition criteria set out in Section 3 was not met. The soil layers have also attained ultimate shaft friction based on the mobilised shaft friction curve with the deeper hard soil layers showing some reserve in shaft friction capacity. The instrumented pile segment in hard soil (11.25m to 16.75m) only recorded a maximum mobilised shaft friction of 155.2kPa and this is likely attributed to the level of the strain gauges which considered some portion of soil shaft friction. The end bearing resistance mobilised for this pile achieved 1647.7kPa (14.0% of maximum test load) with a pile toe settlement of 21mm. It should be noted that the end bearing resistance require significant movement to mobilise which could not comply with the acceptable pile settlement.

PTP-V1-02 is a 1.0m diameter pile tested to 15,000kN with a recorded pile head settlement of 36.52mm. This pile was installed 16.8m in soil and hard soil layers and 2.5m rock socket ($I_{s(50)}$ ranging from 2.63 to 4.21). Mobilised shaft friction in soil, hard soil and rock layers have achieved ultimate condition according to the mobilised shaft friction curve. However, the lowest instrumentation level in rock socket reveals that a less than satisfactory ultimate rock shaft friction was obtained at 172.6kPa with a pile segment movement of 18.84mm. This indicates that the pile body may have slipped at this pile segment which resulted in the remaining loads to be transferred to the pile base. The maximum mobilised end bearing resistance recorded 2370.3kPa (12.4% of maximum test load) with a pile toe settlement of 18.8mm however the mobilisation of the end bearing resistance began developing gradually at smaller pile toe movements.

V2-P56 is a 1.8m diameter pile tested to 24,000kN and the resulting pile head settlement was 7.04mm. As this pile was an actual working pile, the test load was limited to two times the pile working load in order to avoid damaging the pile. This pile was installed 13.5m into soil and hard soil layers and 7.6m into rock ($I_{s(50)}$ ranging from 2.18 to 3.17). Based on the mobilised shaft friction curve, the pile segments in soil and hard soil are seen to approach ultimate shaft friction. Rock layer was encountered earlier than presumed from the borehole results and the ultimate mobilised rock shaft friction was not achieved. This was expected due to the magnitude of the assigned test load. Similarly, the end bearing resistance could not be significantly mobilised as the load transfer from the pile top was unable to reach the pile toe.

Based on the instrumented test pile results, it was observed that back analysed shaft resistance factor ranges from 3.63 to 12.36 for soils with SPT-N blow counts less than 24. On the other hand, soils with SPT-N blow counts exceeding 80 resulted in a shaft resistance factor between 2.07 to 2.66. These shaft resistance factors are slightly on the higher side compared to the findings by Tan *et al.*, (1998), Toh *et al.*, (1989) and Chang and Broms (1991).

Highly weathered / fractured rock considered as hard soil or “soft rock” resulted in a mobilised shaft friction up to 510.4kPa with an average of 342.0kPa; averaged across the lengths of pile constructed in hard soil layers. Nevertheless, the ultimate shaft friction in hard soil layers were not reached in several cases. It was observed that the mobilised shaft friction in competent rock or “hard rock” achieved up to 826.6kPa.

The achieved maximum end bearing resistance ranges between 1647.7kPa to 2370.3kPa, which translate to 12.4% to 14.0% of the total applied load however evidence of soft toe was observed from the results.

6 CONCLUSIONS

Large diameter cast in place bored piles of up to 1,800mm diameter with working load of up to 12,000kN were successfully

designed and constructed for a railway viaduct in Segamat, Johor, Malaysia. The piles were constructed in sedimentary and metamorphic rocks of Semantan and Gemas formations. This paper presents results for four numbers of instrumented pile load tests and the following conclusions can be made:

- Back analysed shaft resistance factor ranges from 3.63 to 12.36 for soils with SPT-N blow counts less than 24, which is considerably higher than the commonly adopted shaft resistance factor of 2.0. Nonetheless, the shaft resistance factor reduces to between 2.07 to 2.66 for soils with SPT-N blow counts exceeding 80.
- The mobilised shaft friction in hard soil or “soft rock” achieved up to 510.4kPa with an average of 342kPa across the pile segments constructed in hard soil layers. The mobilised shaft friction exceeds 250kPa which was adopted as the limiting ultimate shaft friction resistance in soil. This suggests that borehole information showing the rock CRR less than 60% and RQD of 0% can be used to estimate “soft rock” or hard soil layers.
- Mobilised shaft friction in rock layers achieved up to 826.6kPa.
- No end bearing resistance was considered in the design of the bored piles which tallies with the instrumented pile test results whereby end bearing resistance can only be mobilised at considerable pile toe movement.

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