Application of Distributed Fibre Optic Sensor (DFOS) in Bi-directional Static Pile Load Tests

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ABSTRACT: This paper describes a case study of a bi-directional load test on a working pile located at limestone formation area. The test pile was instrumented with Distributed Fibre Optics Strain Sensors (DFOS) to measure the change in strain and to determine the pile shaft friction and end bearing. This paper highlights the advantages and limitations of DFOS in measuring the continuous strain profile of a test pile. Interpretation on the anomalies detected through the DFOS results is discussed. The paper aims to introduce to the industry, the superior information obtained using the innovative fibre optic technology for geotechnical testing and monitoring.

KEYWORDS: Bidirectional static load test, Distributed fibre optic strain sensor, Pile load test.

1. INTRODUCTION

Pile load tests are very important to assess the actual behaviour of piles and to verify the geotechnical design parameters and construction approaches. The Bidirectional Static Load Test (BDSLT) was introduced as one of the Maintain Load Test (MLT) options and applied since 1980's. Although the BDSLT offers time, cost and technical advantages over the conventional MLT, this approach is not widely accepted by the industry due to the fact that it is not a top loaded test.

To compute the skin friction and end bearing of the pile, instrumentations such as vibrating wire strain gauges, electrical-resistance strain gauges, load cells and tell-tale extensometer are used to obtain the strain and displacement at a discrete point within a pile. Recent advancement in the fibre optics technology has opened up more opportunities for studying the actual pile-soil interaction behaviour. The distributed fibre optic strain sensors are able to acquire detailed continuous strain data along a pile, and this unique nature of the DFOS has an advantage in helping engineers locate anomalies such as deformation and non-uniform distributed soil structure interaction forces (Mohamad *et al.*; 2016 & Mohamad *et al.*, 2017). These anomalies might be caused by heterogeneous ground formation or non-uniformity in pile stiffness (Soga *et al.*, 2015), which is difficult to be detected using point wise sensors. Although the DFOS system has been utilized in many different areas for a long time, the understanding of its full capabilities in civil engineering applications is still lacking.

The authors understand the importance of knowledge and experience sharing in order to raise the awareness and confidence level of the industry to embrace and to specify newer technologies. Hence this paper describes the successful application of the DFOS systems to measure the strain within a bored pile during the BDSLT and its corresponding interpretation of the continuous strain profile in the assessment of the bored pile condition.

2. GENERAL PRINCIPLES

This section describes the general principles of bidirectional static load test (BDSLT) and distributed fibre optic sensor (DFOS).

2.1 Bidirectional Static Load Test (BDSLT)

Rapid urbanization process is now a global phenomenon in response to a fast-growing population and development. Construction of skyscrapers and mega infrastructure projects can be seen in many developing countries to serve specific functions in meeting the urban need and to improve the quality of life. The construction of these gigantic structures in urban areas with space constraints often adopts very long bored piles of large diameter, in order to limit the number of piles within a pile group. Therefore, tremendous loads will be imposed on these piles.

Maintain load tests (MLT) assigned to evaluate such a high capacity piles significantly increase the time, cost and risk of the pile test. Testing these highly loaded piles using conventional MLT methods will require the construction of massive reaction systems, i.e. dummy piles, platforms, reaction beams or concrete blocks. The conventional MLT methods with such a massive reaction system involve very high cost and risks, sometimes even human cost. There are many literatures on MLT failures due to platform collapse (Figure 1), tension bar snapped, etc (FPS, 2006).

Figure 1 Examples of MLT kentledge system failure

The Bidirectional Static Load Test (BDSLT) was first introduced by Pedro Elisio and Jorj Osterberg in the 1980's as an alternative approach to the conventional MLT such as Kentledge system and reaction pile system. The main difference between BDSLT and conventional MLT is the position of jacks and the reaction systems. The conventional MLT systems apply load from the pile top. While, in the BDSLT, the jacks are located in an equilibrium point of the pile shaft, separating the pile body into upper section and lower section. When pressure is applied by the hydraulic pump, the bidirectional jack expands and push the upper shaft upwards (to mobilize shaft friction) and lower shaft downwards (to mobilize the shaft resistance and end bearing). Therefore, in a BDSLT, the jack capacity is only half of the test load which leads to a significant saving in cost. The typical schematic diagram of BDSLT set up is shown in Figure 2.

In a conventional MLT, sometimes it is not easy to determine the ultimate shaft friction and end bearing of rock socket in a bored pile, as the magnitude of top down load significantly reduces before it reaches the pile toe to fully mobilize the bored pile. In order to optimize the design of a rock socketed bored pile, designers are keen to determine/verify the ultimate shaft friction and end bearing of the rock layer. This can be achieved relatively easier in a BDSLT as the jack can be placed in the rock layer. The load will be directly exerted on the rock layer before it is transferred to the upper portion soil layer.

Another advantage of BDSLT is the improved safety as the sacrificial jack is embedded within the pile body located deep down in the ground. There is no need for platform preparation or stacking of concrete blocks on the bored pile and this can eliminate the risk of collapse due to soft ground. Besides, for BDSLT, there is no need to extend the pile up to ground level (especially for pile with deep cut off level), hence there is a saving in time and cost. The bored pile can be cast up to cut off level only as the jack exert forces within the pile body.

Without the requirements for massive reaction systems, BDSLT can be performed with minimal space and is able to significantly reduce time, cost and risk.

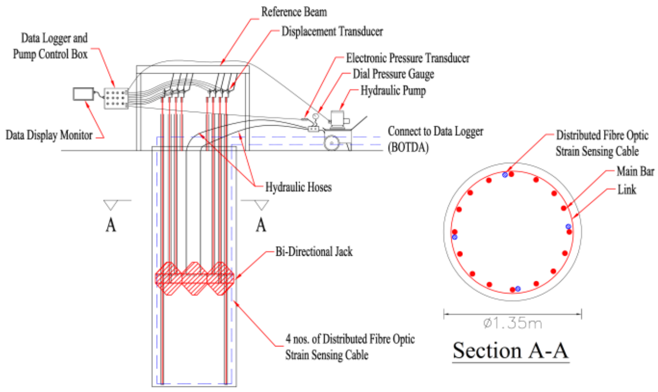

Figure 2 Typical Schematic Diagram of Bi-Directional Static Load Test

2.2 Distributed Fibre Optics Sensors (DFOS)

The application of fibre optic (FO) in the civil engineering industry is still new as compared to its application in telecommunications and the medical industry (Annamdas, 2011). For structural monitoring purposes, there are three main FO sensing approaches, i.e. the Fabry-Perot interferometers (discreet sensing points), Fibre Bragg sensors (quasi-distributed sensor system), and Distributed Fibre Optics Sensing system, DFOS (continuous sensor system).

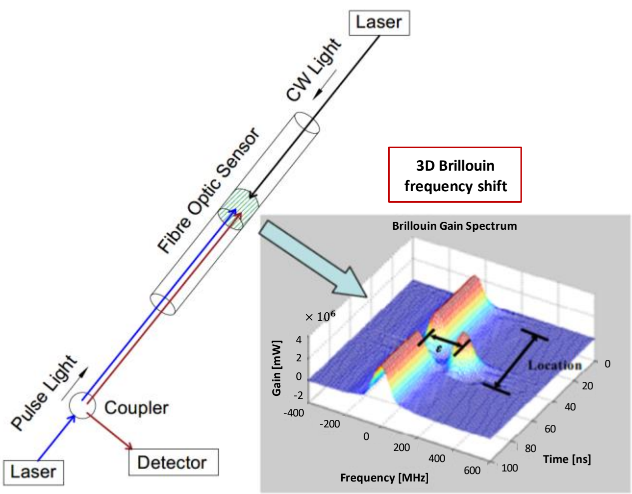

Figure 3 Principle of BOTDA sensor system (Lan *et al*, 2012)

This paper focuses specially on DFOS in measuring strain along the pile body during the bidirectional static load test. The Distributed Fibre Optic Sensor (DFOS) system is a new approach which provides flexibility and capability in structural monitoring based on two major motivations, integrity monitoring and performance-based monitoring. For instrumented test pile, DFOS is used to measure strain, which is then computed to pile shaft friction and pile end bearing through analyses. The DFOS functions based on the properties of spectrum of the backscattered light within the optical fibre, using the Brillouin Optical Time Domain Analysis (BOTDA) system. The principle of BOTDA sensor system is as shown in Figure 3 and the typical BOTDA analyser is as shown in Figure 4. The BODTA technology is a proven robust system in determining the load distribution along the pile shaft and pile toe. Light waves travelling in the fibre optic cable reacts with the glass material in the fibre. The reaction causes a change in density and also a change of frequency, i.e. the Brillouin frequency shift. By resolving the frequency shift and the propagation of time, a continuous full strain profile can be determined.

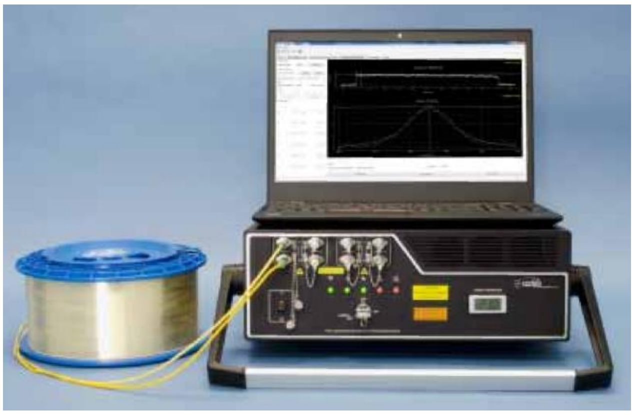

Figure 4 BOTDA Analyser (OZ Optic Ltd)

The main strengths of fibre optic sensors are the nature of glass which is immune to electromagnetic interference, corrosion resistant and inert to chemical reactions. The FO sensors are very versatile and can be safely used in many harsh environments such as high-voltage, marine or explosive environments. Besides, the DFOS cables are thin and therefore simplifies the installation on the reinforcement cage without disrupting the concreting procedures. The DFOS cables embedded in concrete piles are durable and can be measured for many years, hence the DFOS is also very good for long term monitoring. Finally, as compared to conventional sensors, the DFOS system provides a continuous strain measurement along the pile rather than discrete measurements at several designated points.

The limitations of the DFOS system in this form, is that it is not suitable for dynamic measurements. Also, the current technology requires several minutes to obtain a complete measurement. Currently, the cost of the analyzer is high as compared to conventional loggers. However, the price for DFOS system will surely be more competitive when the system is more commonly adopted.

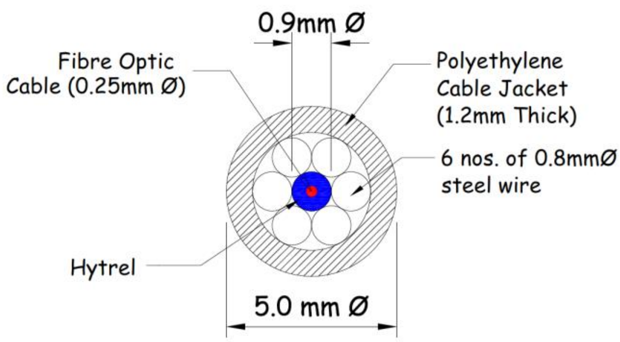

Figure 5 Configuration of strain sensing optical cable

The optical fibre cable used for the bored pile is as shown in Figure 5. The single core single mode optical fibre is reinforced with several strands of steel wires and polyethylene cable jacket. The diameter of fibre optic sensor is 0.125 mm with 0.25 mm cable coating, while the overall cable diameter is 5 mm. The glass core is firmly fixed together with the coating in order to ensure the full transfer of the strain from coating to inner glass core.

3. METHODOLOGY

This section describes the geological information, pile information and the procedures of testing. Criteria for the termination of load test is also included in this section.

3.1 Geological Information

This paper describes the case study of an instrumented pile load test using DFOS in a residential building project. The test pile is a working pile located at limestone formation. The bidirectional static load test aims to verify the geotechnical design parameters for the highly fractured limestone formation.

The soil profile consists of loose to medium dense silty clay, with SPT-N values ranging from 0 to 28 as shown in Figure 5. The limestone layer was found at approximately 61 m below ground level. The Rock Quality Designation (RQD) of the limestone samples are all 0 %, indicating highly fractured limestone, with total core recovery (TCR) ranging from 47 % to 100 % as illustrated in Figure 6.

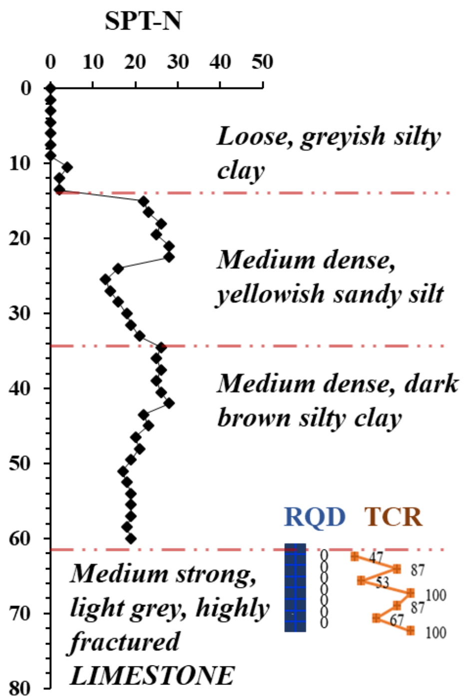

Figure 6 Geological Information of pile location

3.2 Test Pile Information

The test pile is 1.35 m diameter and was designed to a length of 57.3 m with a 6.8 m rock socket length. The pile was cast up to cut off level only, i.e. 10.2 m (empty bored) from the ground level. The bidirectional jack was embedded at 5.8 m from pile toe as shown in Figure 7. Concrete grade 40 was used and the main reinforcement consists of fifteen number of T25 steel bars extending from top to toe of the pile. The boring and coring procedures took approximately 4 days, using bentonite as drilling fluid.

Prior to the construction of bored pile, a bidirectional jack was attached to the reinforcement cage. The DFOS cables were tied to the reinforcement cage at four sides, at 90-degree interval around the circumference of the reinforcement cage.

3.3 Testing Procedures and Criteria

The BDSLT was conducted in accordance to ASTM D8169/D8169M-18 (Standard Test Methods for Deep Foundations Under Bi-Directional Static Axial Compressive Load) and in accordance to load schedule and requirements provided by the design engineer. The test pile is a working pile designed to a factor safety of 2. Therefore, the test load of the pile is twice the working load (WL=12,800 kN and TL=25,600 kN). The loading sequence is summarized in Table 1.

The increment or decrement of loading can only be executed if the rate of the pile settlement is not more than 0.0625 mm per 15 minutes, or 0.25 mm per hour. If this criterion is not met, the maintaining time of the loading stage shall be extended.

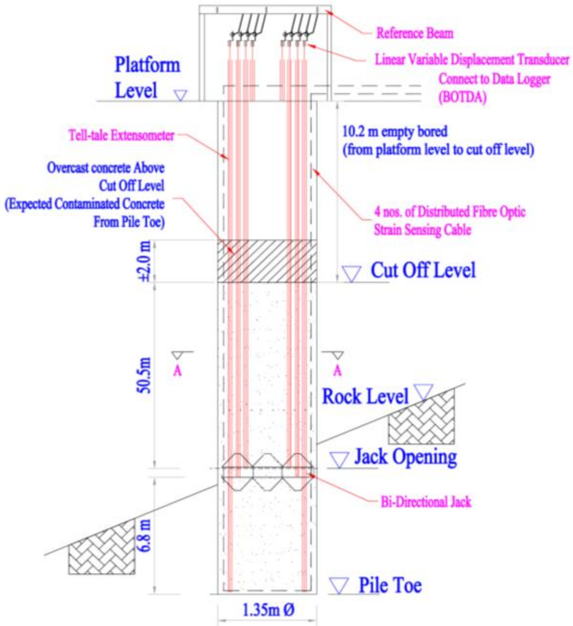

Figure 7 Schematic diagram of the BDSLT set up

Table 1 Loading cycle of BDSLT

Loading Cycle	Working Load	Duration	Loading Cycle	Working Load	Duration
	%	Hour		%	Hour
First Loading Cycle	0	0	Second Loading Cycle	0	0
	20	1		20	1
	40	1		40	1
	60	1		60	1
	80	1		80	1
	100	12		100	1
	80	1		120	1
	60	1		140	1
	40	1		160	1
	20	1		180	1
	0	2		200	24
				160	1
				120	1
				80	1
				40	1
				0	2

This bidirectional static load test shall be terminated if one the following conditions occur:

1. Maximum stroke of the jack is reached (150 mm).
2. Test load is fully applied.
3. Instruction from design engineer.

In this test, excessive upward movement (>30 mm) was recorded on the pile above the jack with applied test load still less than designated maximum test load. The design engineer instructed the termination of test in order not to damage the working pile. The jack and gap beside the jack were post-grouted with epoxy and non-shrink cement grout after the load test to ensure the integrity of the working pile.

4. DATA ANALYSIS

This section describes the strain profile, pile shaft friction and the pile movement. The potential causes of the low shaft friction are also discussed in this section.

4.1 Strain Profile

Figure 8 shows the continuous axial strain profile at several loading stages, i.e. 60%, 100%, and 150% of the working load (WL). Along the pile, the DFOS measurement indicates consistent strain increment as the load increased. It is interesting to observe that at the depth between 16 m to 43 m, spikes were recorded at the same localise depth for the four sensing cables located at the four different corners of the pile. They are visible at the same depth in all cables despite each having an independent baseline profile. This spike is likely due to the variation of pile stiffness ($E_cA_c+E_sA_s$) (caused for example by features such as local change in concrete, E_c and steel modulus, E_s; cross sectional area of concrete, A_c and steel, A_s or isolated stiffening bands in the reinforcement cage) (Soga *et al*, 2015).

If the spike is due to the inhomogeneity of concrete, the concrete modulus can be back calculated by assuming consistent pile diameter and steel area along the pile. Figure 9 shows the back calculated concrete modulus at selected depth along the pile. At depth without spikes, etc. 30.0 m, 35.0 m and 46.5 m, the concrete modulus is generally in the range of 20 to 21 kN/mm^2. At depth with spikes, etc. 27.7 m, 36.5 m and 41.65 m, the concrete modulus was much lower, at about 10 kN/mm^2. These could indicate that the concrete strength at those pile depths with spike are likely lower than the average strength. Possible cause of this is to the presence of muck from pile toe that was not completely flushed above the cut-off-level and may have caused contamination to concrete on certain depths of the pile. If the muck was completely flushed, localise spike on strain profile should had been recorded at the 1 to 2 m overcast concrete above the cut-off-level.

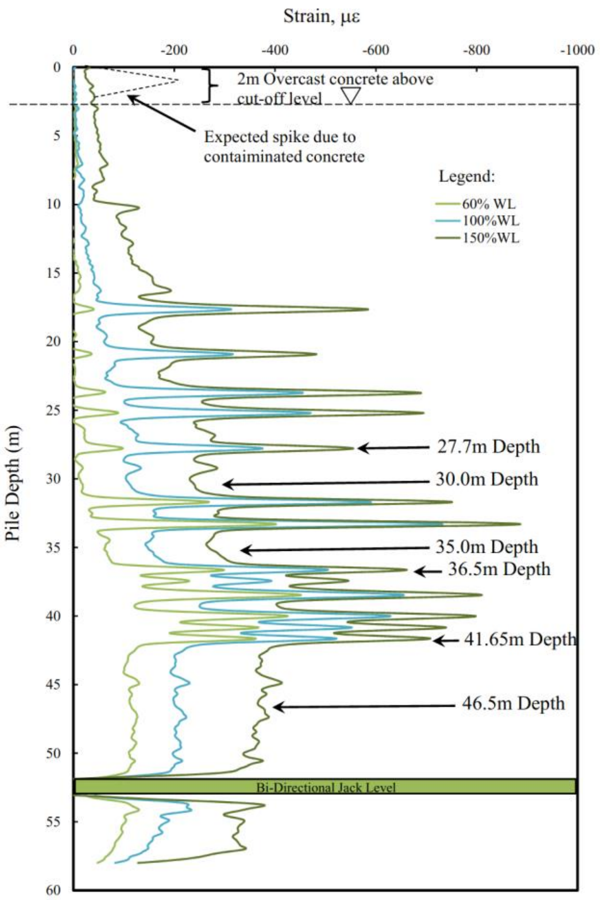

Figure 8 Change in strain along the pile during BDSLT

The back calculated concrete modulus, E_c is lower than usual 28 days concrete modulus as suggested in BS8110. This could be due to the pile being tested 14 days after concreting. The concrete may have achieved the required strength but the concrete modulus may have not fully gain it yet.

For such a long pile, the concrete might be slightly contaminated and compromised the quality of concrete (e.g. Young's Modulus, E) at these localized areas resulting in much higher strain measurements recorded. Local change in steel cross sectional area, reinforcing links or isolated stiffening bands in the reinforcement cage may also have caused the recorded spikes.

These localised depth-with-spikes are excluded from the interpretation of load transfer and mobilised shaft friction analysis as the pile stiffness are uncertain.

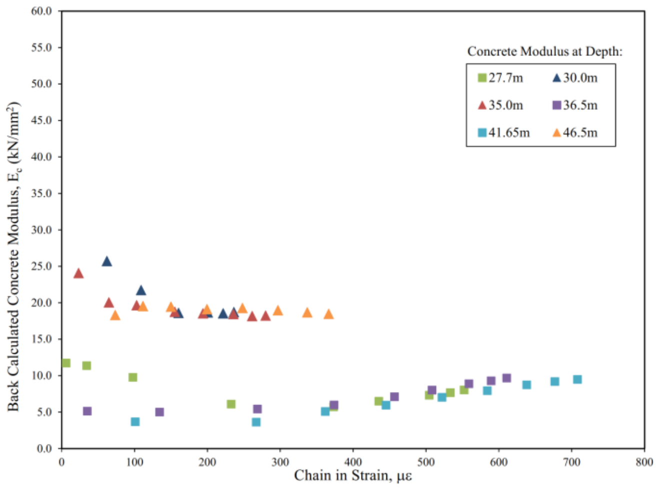

Figure 9 Back Calculated Concrete Modulus

4.2 Pile Shaft Friction

The strain data in this paper is analysed to obtain the axial forces. It is important to note that uniform axial stiffness along a bored pile is nearly impossible due to the generally non-uniform shape of bored pile and the minor inconsistency in the concrete stiffness. However, the pile stiffness, *EA* is assumed as constant along the pile to facilitate the interpretation of pile load transfer, mobilised shaft friction and mobilised end bearing mechanism. Several input parameters for the load distribution analysis are summarized in Table 2.

Table 2: Input parameters for load distribution analysis

Pile diameter	1350 mm
Working load	12,800 kN
Test load	25,600 kN
Main reinforcement bar	15T25
Area of bar	4713 mm^2
Area of concrete	1426861 mm^2
Steel yield strength	460 N/mm^2
Grade of concrete	40

Figure 10 illustrates the load distribution along the test pile. The jack was embedded within the limestone bedrock with 0.8 m rock cover above the jack. However, from Figures 11 and 12, it is observed that the 0.8 m rock layer above the jack did not contribute to an expected high shaft friction to the pile. The maximum mobilised ultimate shaft friction at the rock layer above the pile is only 39 kN/m^2, which is lower than the soil layer above it that has mobilised shaft friction up to 62 kN/m^2. The ultimate soil friction value was estimated based on SPT-N method. Initially, the ultimate soil friction was estimated based on a constant ultimate shaft resistance factor, K_{su} of 4 throughout the soil layer. However, the denser soil layer only achieved K_{su} value of 2.5; while, the looser soil layer achieved K_{su} value of approximately 3.5.

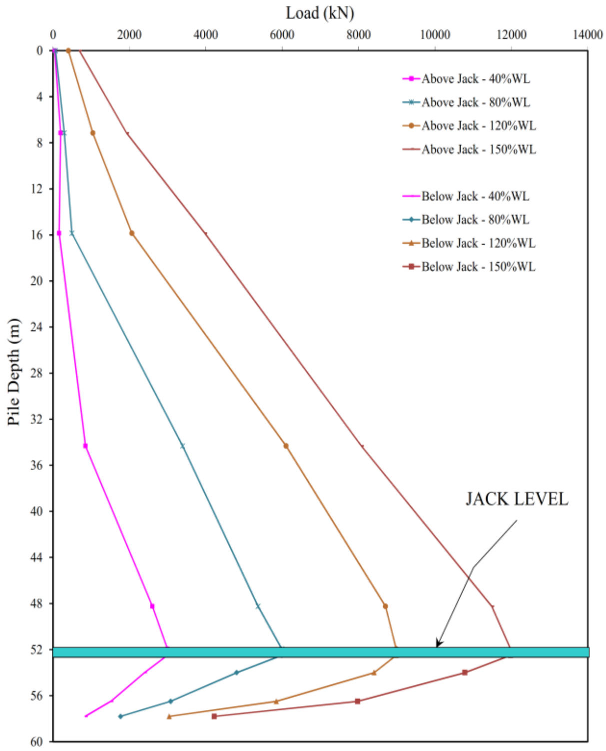

Figure 10 Load distribution along test pile

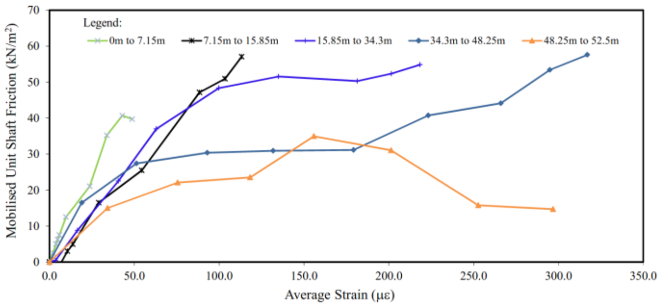

Figure 11 Mobilized Unit Shaft Friction (Upper Pile Section) against Average Strain

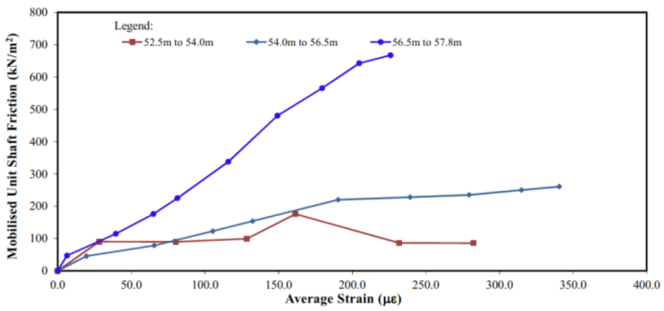

Figure 12 Mobilized Unit Shaft Friction (Lower Pile Section) against Average Strain

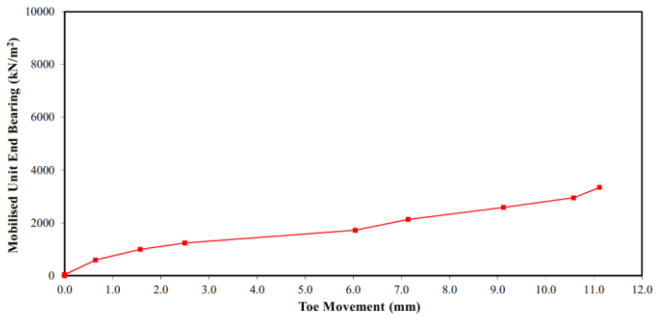

Figure 13 Mobilised Unit End Bearing against Toe Movement

The ultimate pile shaft friction of the rock socket below the jack (Figure 12) was also lower than initial proposed friction value but is near to proposed limestone friction value in the literature (Neoh, 1998) based on the RQD value of the rock. From 52.5 m to 54.0 m, the mobilised shaft friction is 176.5 kN/m^2 and 260.5 kN/m^2 at 54.0 m to 56.5 m. These two layers have mobilised ultimate shaft friction near to the suggested value in literature which suggested 300 kN/m^2 for rock with RQD <25%. However, at the last rock socket layer, 56.5 m to 57.8 m, a mobilised shaft friction of 667.3 kN/m^2 (>300 kN/m^2) was achieved.

Figure 13 indicates that the end bearing of the test pile was not fully mobilised. The mobilised end bearing was 3339.5 kN/m^2 up to maximum test load.

4.3 Pile Movement

As shown in Figure 14, the upper pile section started to experience drastic movement at 140% working load. As this was a working pile, the test was terminated at 150% to avoid the working pile being loaded to failure. At 150% working load, the upper pile movement was 33.24 mm, while the lower pile section moved 11.12 mm.

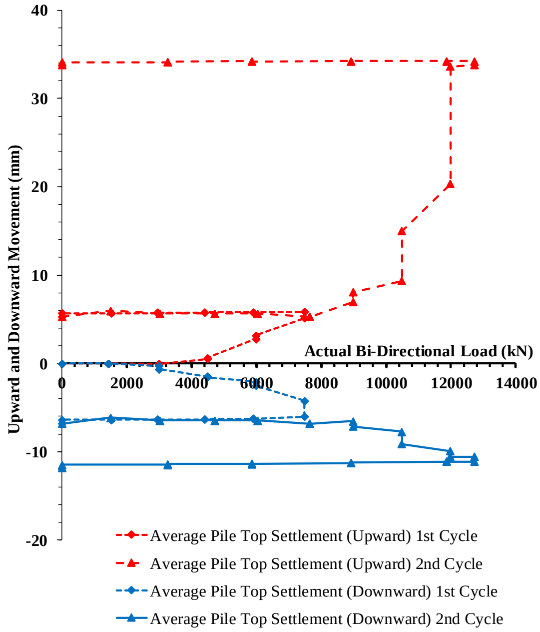

Figure 14 Upward and downward movement against applied load

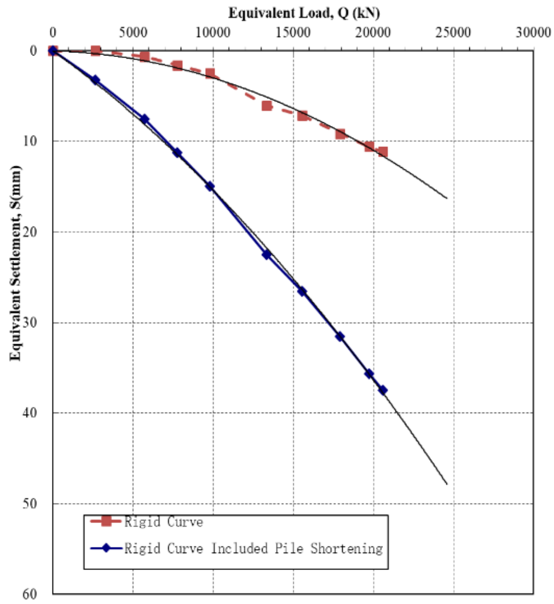

Figure 15 Equivalent top settlement curve

Figure 15 shows the equivalent pile top settlement curve, converted using an equivalent conversion method. The construction of equivalent top load versus movement curve is according to Specification for Static Loading Test of Foundation Pile – Self-balanced Method (JT/T 738-2009) and subjected to the following assumptions:

1. Pile is elastic;
2. Based on the designed pile capacity, test pile is evenly divided into upper and lower parts, with a jack placed at the equilibrium point of the two sections;
3. Loading mechanism of the bottom section of the pile (below the jack) is similar to the loading mechanism of a top-loaded pile. Therefore, it has the same load-movement behaviour as a top loading pile.
4. Shortening of the upper section of pile, Δ is equal to the sum of elastic shortening caused by the (i) downward load and the (ii) upward load – self weight of upper pile.

$$\Delta = \Delta_1 + \Delta_2 \tag{1}$$

Where,
Δ = Shortening of the upper section of pile.
Δ_1 = Elastic shortening caused by the downward load.
Δ_2 = Elastic shortening caused by the (upward load – self weight of upper pile).

$$\Delta_1 = \frac{F_{\downarrow} L}{E_c A_p} \tag{2}$$

$$\Delta_2 = \frac{(F_{\uparrow equivalent} - W)\, L}{2\, E_c A_p \gamma} \tag{3}$$

Where,
$F_{\downarrow}$ = Downward load of jack.
$F_{\uparrow equivalent}$ = Equivalent upward load obtained from the upward movement versus load curve. (i.e., upward load, where the upper pile section displaced at the same magnitude as downward movement of jack when downward load is $F_{\downarrow}$).
L = Length of upper pile section.
E_c = Concrete modulus.
A_p = Pile cross sectional area.
W = Self weight of upper pile body.
γ = Upper shaft friction adjustment factor.

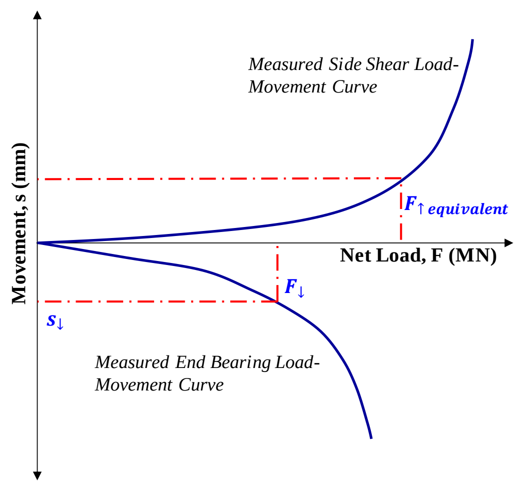

Figure 16 Upward and downward movement vs load curves

Combining the Eqs. (1), (2) and (3):

$$\Delta = \Delta_1 + \Delta_2 = \frac{[(F_{\uparrow equivalent} - W)/\gamma + 2F_{\downarrow}]\, L}{2\, E_c A_p} \tag{4}$$

The equivalent top load, F_{top} can be calculated:

$$F_{top} = \frac{(F_{\uparrow equivalent} - W)}{\gamma} + F_{\downarrow} \tag{5}$$

Therefore, the equivalent pile top settlement, S_{top} is:

$$s_{top} = s_{\downarrow} + \Delta \tag{6}$$

Where,
S_{top} = Equivalent pile top settlement
$s_{\downarrow}$ = Downward movement of jack

Figure 16 shows the upward and downward curves obtained from BDSLT and Figure 17 shows the equivalent top loaded settlement curve (after conversion).

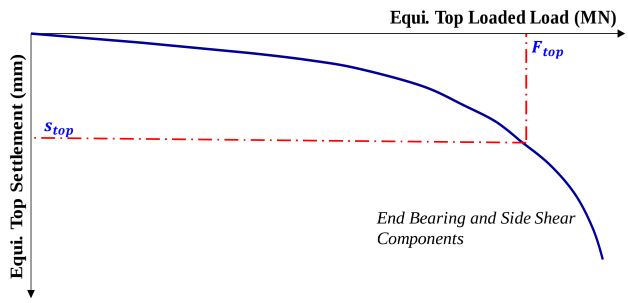

Figure 17 Conversion curve of top load versus movement

4.4 Potential Causes of Low Shaft Friction

Despite the rock socket length of the test pile being 5.8 m, the result indicates that the test pile did not achieve the designed pile unit shaft friction of 600 kN/m^2 at the limestone layers except at the final segment of the pile, i.e. approximate 1.5 m rock socket from the pile toe. This section discusses the potential causes of the low shaft friction.

4.4.1 Karstic Limestone Feature

The test pile is located in Kuala Lumpur, Malaysia, on a limestone formation. Extra considerations need to be taken to design a pile in Kuala Lumpur limestone due to the extremely karstic feature.

For the first 4 m length rock socket, the Rock Quality Designation (RQD) of the limestone samples are less than 25% and total core recovery (TCR) are generally low, indicating the pile was socketed into highly fractured limestone. This may result in lower ultimate rock friction value.

Soil investigation result also shows that the test pile was located on an inclined limestone surface as shown in Figure 6. The top of rock levels around the pile differed by a range of about 1.5 m. Therefore, the effective/actual pile rock socket length is likely less than 5.8 m, resulting in lower total mobilised rock friction obtained during the BDSLT.

The good practice of checking the solid and flat bed rock layer through dipping procedures can be carried out to ensure the pile was fully rock socketed as per design, and thus achieve expected shaft friction.

4.4.2 Construction Method

The other common potential causes of low shaft friction is the construction method which include the time taken for boring and coring, quality of stabilizing fluid, concrete quality etc. Stabilizing fluid shall not be stagnant in the bored hole for a long period of time (Tucker, 1984) to prevent the reduction in shaft friction.

In this case, the construction records and concrete test results showed that the pile was constructed in accordance to the proper procedure. Therefore, the lower RQD, lower TCR and inclined bedrock is likely the biggest contributing factor of the resulting low shaft friction of this test pile.

5. CONCLUSIONS

A bidirectional static load test on a working pile was carried out with instrumented Distributed Fiber Optic Strain Sensor cables. The test pile was constructed in Kuala Lumpur limestone formation. According to Neoh (1998), for limestone with RQD<25%, the suggested ultimate rock friction value is 300 kN/m^2. The mobilised ultimate rock friction for this pile are generally ranged from 175 to 260 kN/m^2 for first 4 m rock socket, which very close to the value suggested. The highest mobilised ultimate shaft friction achieved was at segment near the pile toe (last 1.8 m), approximately 700 kN/m^2, which also with RQD< 25% but with generally higher (>50%) TCR value.

The novel way of instrumented load test using Distributed Fibre Optics Strain Sensing indicated significant advantage by obtaining the continuous strain profile of the entire pile length easily as compared to discrete data from conventional strain gauges attached at pre-determined points along the pile, which often require data interpolation between limited sensing points, laborious installation time and data problems arising from local erroneous/anomaly measurements. The continuous strain profile obtained using DFOS not only allow engineers to understand the overall performance of the tested pile but also the quality of concrete/integrity of the pile.

The continuous strain profile of the entire pile length ensures that the engineer would not miss out on any important data along the pile shaft that may affect any computation of the skin friction and end bearing.

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