Review of driven pile design, construction & performance in extremely soft clay condition for a major bridge link project in Southeast Asia

Examen de la conception de pieux battus, de la construction et du comportement dans la condition d'argile extrêmement douce pour un grand projet de pont de liaison en Asie du Sud-Est

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ABSTRACT: The recently completed Brunei Temburong Bridge is a 30km road link crossing the shallow Brunei Bay and the swampy Temburong National Park area to link up the enclave of Temburong District to the capital city of Brunei, a country situated in the northern coast of the Borneo Region, Southeast Asia. The construction of the bridge involved the installation of over 11,000 driven precast prestressed concrete piles into the ground with extremely soft clay, at a maximum depth of over 70m. Eurocode 7 was adopted for the design and testing of the foundation, which was the first of its kind in the Borneo region. Abundance of pile testing and installation records were available upon the completion of the project. This enables an extensive and thorough review of the design, construction, and the performance of the driven pile foundation. In this paper, the authors will describe the pile design philosophy and the Eurocode adoption during the design process. The findings on the review conducted will be summarised and explained. The authors will discuss on the lesson learnt and propose key improvement to the planning and design process for further projects.

KEYWORDS: Eurocode 7; precast prestressed concrete pile; extremely soft clay; driven pile foundation.

1 INTRODUCTION

The recently completed Brunei Temburong Bridge is a 30km road link spanning over Brunei Bay to connect relatively isolated district of Temburong with the more developed Brunei-Muara district (see Figure 1). Due to the presence of low strength superficial deposits along the alignment in Brunei Bay and Temburong, precast prestressed concrete piles (spun piles) were chosen as major foundation type for Marine Viaducts and Temburong Viaducts (Yiu et al. 2015).

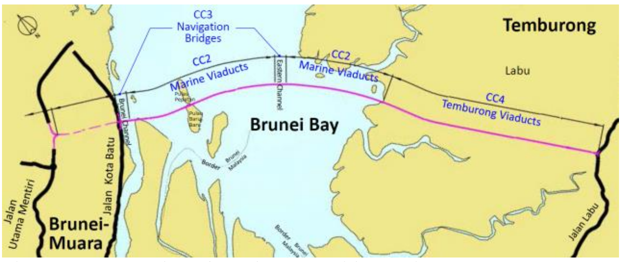

Figure 1. Layout plan of Temburong Bridge Project.

In the following sections, the pile design methodology, the findings and recommendations on the review of extensive construction data for over 11,000 spun piles of 1m diameter will be discussed.

2 METHODOLOGY OF PILE DESIGN

2.1 *Design Standards*

In this project, pile design was conducted in accordance with Eurocode 7 and followed the requirements of BS EN 1997-1:2004, in conjunction with the UK National Annex.

2.1.1 *General*

Design Approach 1 in UK National Annex to Eurocode 7 (UKNA) was adopted for design calculations. When calculating the pile axial load capacity and checking against the actions (axial loading), Combination 1 and 2 were adopted as stated in EC7 2.4.7.3.4.2(2).

$$Combination\ 1: A1 \text{ " + " } M1 \text{ " + " } R1 \tag{1}$$

$$Combination\ 2: A2 \text{ " + " } (M1\ + M2) \text{ " + " } R4 \tag{2}$$

Where A1 & A2 are actions, M1 & M2 are material strength (for example c' and phi') and R1 & R4 are the resistance (pile capacity). Each set has its specific combination of partial factors according to UKNA.

In ULS checks, Combination 1 requires a partial factor > 1.0 on permanent actions while Combination 2 requires a partial factor > 1.0 on resistance. Table 1 generally summarises the partial factor approach for pile design.

Table 1. Summary of partial factors for pile design

Approach	Actions	Soil Parameters	Resistance
Combination 1	A1 > 1.0 For Permanent and Variable Action;	M1 = 1.0	R1 = 1.0
Combination 2	A1 = 1.0 For Permanent Action; A1 > 1.0 For Variable Action	M1 = 1.0	R4 > 1.0

2.1.2 *Partial Factors*

Partial factors shall be applied to actions, material strength and the resistance. The partial factors for actions were adopted according to EN 1990 and the UKNA. For material strength, UKNA Table A.NA.4 the partial factors for soil parameters for the STR and GEO limit state and UKNA Table A.NA.2 the partial factors for soil parameters for accidental conditions were applied to the foundation design of this project (see Table 2 and Table 3). For resistance, the partial factors specified in UKNA Table A.NA.6 and A.NA.7 for driven piles were adopted and summarised in Table 4.

In accordance with EC7, all values of partial factors for actions or the effects of actions in accidental situations are taken as unity. all values of partial factors for resistances should then be selected according to the particular circumstances of the accidental situation.

In addition, a model factor of 1.2 or 1.4 should be also applied to the calculated pile capacity to ensure the calculated resistance err on the safe side. The value of model factor applied to the resistance depends on the availability of maintained load test verifying the calculated, unfactored ultimate resistance.

Table 2. Partial factors for soil parameters for the STR and GEO limit state

Soil Parameter	Symbol	M1	M2
Angle of internal friction	$\gamma_{\phi'}$	1.0	1.25
Effective cohesion	$\gamma_{c'}$	1.0	1.25
Undrained shear strength	γ_{cu}	1.0	1.4
Unconfined strength	γ_{qu}	1.0	1.4

Table 3. Partial factors for soil parameters for accidental conditions

Soil Parameter	Symbol	M2
Angle of internal friction	$\gamma_{\phi'}$	1.1
Effective cohesion	$\gamma_{c'}$	1.1
Undrained shear strength	γ_{cu}	1.2
Unconfined strength	γ_{qu}	1.2

Table 4. Partial resistance factors for driven piles for the STR and GEO limit states

Resistance	Symbol	R1	R4 (Without explicit verification of SLS)	R4 (With explicit verification of SLS)
Base	γ_b	1.0	1.7	1.5
Shaft (compression)	γ_s	1.0	1.5	1.3
Total/ combined (compression)	γ_t	1.0	1.7	1.5
Shaft in tension	$\gamma_{s,t}$	1.0	2.0	1.7

2.2 *Ground Investigation*

A set of ground investigation boreholes along the alignment with approximate 300m spacing was carried out to reveal general ground conditions and determine preliminary pile length for tendering construction contract. In detailed design stage, additional boreholes, namely predrill, were undertaken at much closer spacing. They were used to verify the design assumptions in preliminary design stage and to refine the estimated pile length. For offshore piers, the piers were located in 50m spacing, at least one predrill was assigned at each pier. For swamp areas, where the piers were designed for closer spacing of 12m, the predrills were carried out alternately at the piers in 24m spacing. It was considered that the arrangement of predrill was sufficient to estimate the ground conditions of the pier without predrill. In the meantime, the predrills were carried out alternately on south bound and north bound of the pier. The variation of geological strata could then be inferred by adjacent predrills. In case geological profiles revealed from adjacent predrills were considerably various, it was good for construction planning especially preparation of pile segment. General arrangement of predrill for onshore piers is shown in Figure 2.

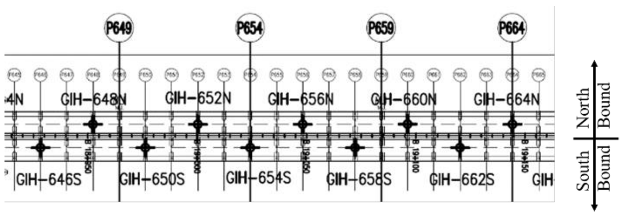

Figure 2. General arrangement of predrill for onshore piers.

2.3 *Geological Zoning for Pile Design*

The geology of Brunei Bay generally comprises 15 - 45m thick surficial very soft to soft clay, underlain by 5 - 40m thick firm to very stiff clay, and 5 - 50m thick sand layer. Hard clay and sandstone/mudstone are followed by the sand layer and revealed at shallower depth close to Brunei Bay and the shore of Temburong district. The thickness of each layer of soil or rock is varying along the alignment (Bush et al. 2015). It was expected that the pile length would be varied with geological profile. Since this is a fast track project, predrills, detailed pile design and construction would be conducted sequentially within limited period. To facilitate the pile design and provide reasonable conservatism for considerable amount of spun piles, the alignment had been sub-divided into several zones based on typical geological conditions, stratigraphic levels and variations in seabed level.

2.4 *Initial Pile Length Prediction*

In general, pile length of driven piles is determined on the following basis:

- Static axial pile capacity
- Soil condition (i.e. SPT N value)
- Driveability analysis
- Required pile length to satisfy lateral resistance

2.4.1 *Static Axial pile Capacity*

To estimate the ultimate and factored shaft friction and base resistance for specified pile lengths, in-house geotechnical program *Oasys PILE 19.5* was employed, which was fully complied with the UK National Annex to Eurocode 7. Total stress approach (α-method) and effective stress approach (β-method) associated with characteristic soil strength parameters were used to determine shaft friction and base resistance provided by cohesive and non-cohesive soils respectively.

For α-method, the ultimate base resistance was estimated with the undrained shear strength, c_u as follows:

$$q_b = 9c_u \tag{3}$$

The ultimate shaft resistance was estimated as follows:

$$\tau_s = \alpha c_u \tag{4}$$

where α is the adhesion factor. With reference to Bowles, 4th Edition, the following adhesion factor are adopted for pile design:

$\alpha = 1$ for $c_u \leq 25kPa$
$\alpha = 0.75$ for $25kPa < c_u \leq 50kPa$

$\alpha = 0.5$ for $c_u > 50kPa$

Design undrained shear strength was derived using some common correlation with effective overburden and standard penetration test (SPT) blow count and verified with vane shear strength, CPT results and documented values (Thomas et al. 2015). It was limited by the correlated undrained shear strength with uniaxial compressive strength of mudstone.

For β-method, the ultimate base resistance was calculated with effective overburden, σ_v', and the bearing capacity factor, N_q as follows:

$$q_b = N_q \sigma_v' \tag{5}$$

The bearing capacity factor, limit of shaft friction and base resistance were assumed initially based on our experience on similar soil materials. With site specific pile test data, these values were back calculated as summarised in Table 5.

Table 5. Summary of design parameters for β-method

			Limiting value, kPa	
	β	*N_q*	*Shaft*	*Base*
Medium dense sand	0.37	40	140	2250
Dense sand	0.46	80	140	2250
Very dense sand	0.6	102	140	4500

As the top soil surrounding the piles would be eroded by the current when piles are installed, contribution of soil within the scour depth to the pile capacity was ignored.

2.4.2 *SPT N-value Profile*

For the areas that harder stratum such as mudstone was found shallower than the estimated pile lengths predicted by the static calculation. Despite the calculation would indicate the necessity of penetrating into this harder stratum to obtain the required design resistance, in reality, it was anticipated in the design stage that the driving of the piles would terminate at the top of the harder stratum, and the required design resistance would be achieved through the demonstration of final set.

Based on the past experience in Hong Kong, Macau and other southeast Asia countries, it was considered that spun piles would usually be driven to a depth where SPT-N value is between 50-80 with the required design resistance attained. Therefore, the tentative pile toe levels were determined by locating the SPT-N 50-80 elevation. These levels were then compared to those obtained by static calculation. If the predicted level using SPT profile is shallower, then it would be adopted as pile toe.

2.4.3 *Driveability and Final Set Criteria*

Despite having predicted the pile lengths using the methods mentioned above, it was considered a final set criteria should be imposed on design piles at the end of pile drive to ensure the required design resistance would be achieved. The Hiley formula was used for the purpose of deriving a final set and a set table, as well as a suitable hammer and drop height. Pile driving simulation software, GRLWEAP was employed to predict the penetration rate at specified penetration depth and assess whether the pile could be driven to design toe level. The results of driveability analysis would be cross checked with and refined the set table. The final set criteria were generally set at a typical range between 25mm and 50mm per 10 blows for stiffer ground, where the pile would be terminated shallower than the required depth determined by static axial pile capacity and the pile length satisfied lateral stability. For the pile founding on relatively soft soil, the final set criteria might not be achieved. The pile would be driven to the minimum depth calculated based on static axial pile capacity with the predicted penetration rate. The predicted penetration rate by GRLWEAP was adopted as one of termination criteria of pile installation.

2.4.4 *Pile Length based on lateral Stability Analysis*

Due to the presence of thick soft clay, the driven spun piles shall be found on a deep level to achieve the required pile capacity. As such, lateral stability was not governing the design pile length.

In area where shallow mudstone was encountered and where final set could be achieved at shallow depth, driven steel piles were proposed and an assessment of the minimum required embedment to satisfy lateral stability was undertaken. During construction, pre-boring or drive-drill-drive could be required to achieve the minimum required embedment if shallow refusal during driving is encountered.

2.5 *Preliminary Piles*

Prior to design and construction of working piles, static load tests and PDA tests would be carried out on full size instrumented preliminary trial piles. Preliminary pile was located at the representative location of each zone to verify the design assumptions, constructability and pile axial capacity. Meanwhile, construction method and workmanship used by the contractor could be identified and validated with the specified performance criteria.

As the preliminary piles might be tested to failure, they could not be reused and would be cut and abandoned after testing. In order to have a better prediction on working piles and facilitate afterward construction of working piles, preliminary piles and associated in-situ tests were installed as closer as possible to the proposed pile caps (see Figure 3), for instance within 25m radius of the centre of the pier. For each preliminary pile test (PPT), there were at least one borehole, one vane shear test and one CPT for determining the length of the tested pile. The borehole, vane shear test and CPT shall be carried out within 5m radius of the preliminary test pile.

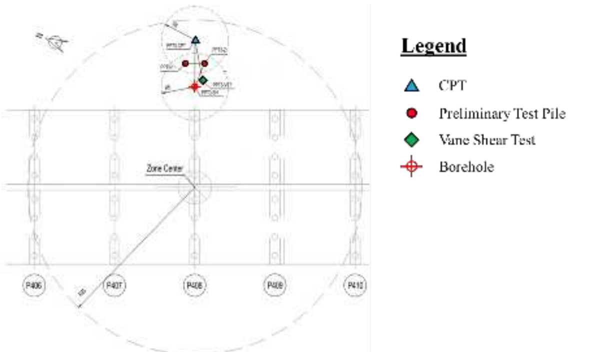

Figure 3. Location of preliminary pile and associated in-situ tests.

2.5.1 *Maintained Load Test (MLT)*

The preliminary piles would be carried static load tests and loaded to its design ultimate pile capacity, i.e. approximate 210% of the design verification load (DVL). A static load test on the preliminary pile is considered successful if the maximum specified load is reached with total movement of the pile top not more than 10% of the base diameter.

The MLT was carried out in two cycles as shown in Figure 4. In 1st load cycle, the loading to the preliminary pile was increased in a loading step of 25% DVL until 100% DVL reached. The loading was then released with same loading step. In 2nd load cycle, the loading was applied 100% DVL for 1st loading step and continue to increase with a loading step of

25% DVL until 210% DVL was reached. The pile was then conversely unloaded in a loading step of 25% DVL. For each increment and reduction of loading, the load shall be held for certain period of time to ensure the loading applied can be transferred to the surrounding soil and down to pile toe.

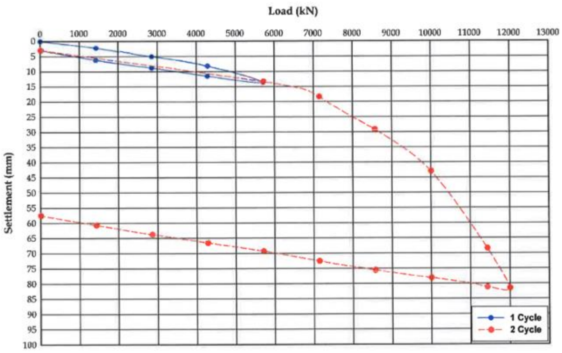

Figure 4. Loading and unloading of maintained load test

To measure the pile movement during MLT, a number of instrumentations were installed in preliminary piles including:

- 4 dial gauges to measure the displacement of the test pile top
- 4 strain gauge transducers at 5m intervals from pile top to bottom to measure the compressive strain of the pile along the pile length for the determination of load dissipation through shaft friction and end bearing
- Rod extensometers installed to pile base and at the elevation where there is change of geological stratigraphy for the determination of the pile movement at depth.

With measured strain gauge readings, ε, pile stiffness, E_p and cross-sectional area, A, pile load at each level could be calculated as follows:

$$P_p = \varepsilon \times E_p \times A \tag{4}$$

Pile load at different level could be plotted together to obtain load distribution along the pile (see Figure 5). The load distribution could be used to determine shaft friction and base resistance. The shaft friction could be considered as the change of pile force with depth. As shaft friction would exert on the pile surface over depth, it was anticipated that the pile load would decrease gradually due to load transferred to surrounding soil. Damaged strain gauge was occasionally occurred due to pile hammering, which might cause fluctuating readings and shall be eliminated from determination of shaft friction. The residual pile load at pile tip was taken by underneath soil and considered as base resistance.

It would be more useful to obtain the ultimate geotechnical pile capacity by testing the pile to fail. In common practice, pile was considered as failed when pile head movement reaches to 10% of pile diameter. With the ultimate pile capacity determined, working pile length could be optimised so as to reduce overall construction cost. This was of paramount importance for the project with extensive amount of piles.

However, in this project, most of the preliminary piles could not be tested to ultimate pile axial capacity as it was difficult to predict the pile capacity accurately and to decide the number of reaction piles or concrete blocks for testing, and this was a fast track project. On other hand, if the preliminary pile was designed to a smaller capacity in order to fail the pile, it could not secure that working pile could achieve design capacity based on same design assumptions and construction method.

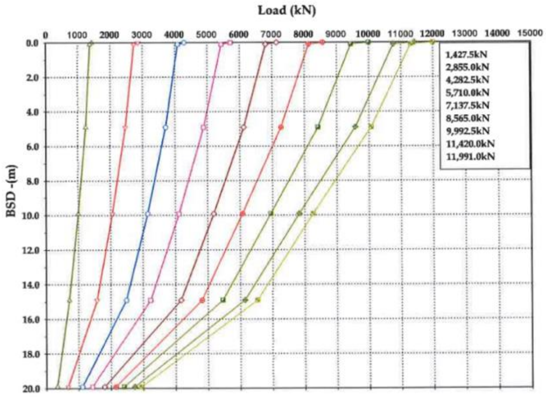

Figure 5. Back calculated pile load distribution

2.5.2 *Pile Driving Analysis (PDA)*

For each preliminary pile test, PDA was carried out on at least 1 pile to together with CASE and Case Pile Wave Analysis Program (CAPWAP) analysis to ascertain:

- the energy delivered by the pile driving hammer to the pile,
- maximum driving compressive stresses,
- pile integrity,
- location and extent of structural damage, and
- load carrying capacity of the pile.

All PDA was calibrated against static load tests carried out on comparable piles (i.e. the same type and similar size constructed in the same ground conditions at the same site using the same installation criteria) prior to conducting on preliminary piles and working piles. Approximate 20% of the total number of working piles were carried out PDA test to further verify pile capacity and integrity.

PDA test of preliminary piles was carried out at:

- immediately after the final set of the tested pile is achieved;
- 14 days after the final set of the tested pile is achieved; and
- 28 days after the final set of tested pile is achieved.

This set of PDA data could provide the ratio of soil strength gained from dynamic to static response. The ratio could provide a forecast to long-term pile capacity based on the pile capacity tested right after pile driving. For working pile, PDA test would only be conducted at end of driving. This forecast could be used to set the minimum requirement of PDA pile capacity at end of pile driving as one of the termination criteria for working piles. As pile capacity tested from PDA was an estimation based on stress wave propagation through the pile, the measured pile capacity shall be used as reference and checked against the tested pile axial capacity from MLT.

Pile integrity could also be checked from PDA test. In PDA results, pile integrity would show a beta value ranged from 0 to 1. Piles with beta value of not less than 0.8 shall be deemed acceptable, whilst piles with beta value of less than or equal to 0.6 shall be deemed as totally damaged. For beta value larger than 0.6 but less than 0.8, the piles were classified as damaged and shall be repaired or replaced.

2.5.3 *Review of test results*

From MLT, the maximum tested pile load could be considered as the ultimate pile axial capacity. The ultimate pile capacity was used to verify the design assumptions and parameters. The back-analysed and verified design parameters would then be used to predict the pile capacity and required pile length for working piles. Meanwhile, model factor could be relaxed to 1.2 for working piles as ultimate soil resistance had been verified by the MLT. Since most of the preliminary piles were not tested to failure, the maximum tested pile load might not be the ultimate pile axial capacity. As such, the back-analysed soil parameters would be conservative, and the calculated pile length of working piles would be longer than the required.

Regarding pile driveability analysis, some key input parameters such as efficiency of hammer and gain/loss factor of soil resistance could be obtained and calculated through PDA test. The efficiency of hammer could be considered as the ratio of energy transferred to the pile and the energy generated by the hydraulic hammer. The impact energy could be calculated by the change of instant velocity detected by PDA sensors. The generated energy could be simply considered as the potential energy of the hammer at the highest level before free falling. Moreover, soil resistance to pile driving (SRD) and long-term soil resistance (LTSR) could be estimated from PDA tests at immediately after the end of the pile driving and 28 days after the end of pile driving. Gain/loss factor of soil resistance could then be estimated as the ratio between SRD and LTSR. Both calculated hammer efficiency and gain/loss factor would be used for pile driveability analysis of working piles.

During the construction of preliminary piles, driving records including hammer weight, drop height and penetration rate at each penetration depth could be obtained. In view of over conservative soil parameters were back-analysed, and some input factors for driveability analysis were estimated through PDA test, the driving records could be used to refine the design parameters and factors and get a prediction of pile length of working piles.

2.6 *Working Piles*

Due to the fast track construction programme, foundation works were undertaken in several work fronts based on the availability of marine and onshore transportations. The preliminary pile tests and working piles on these work fronts were required to conduct early. The rest of the preliminary pile tests were conducted simultaneously during construction of working piles.

2.6.1 *Review of Pile Length Prediction*

From preliminary pile test, the design assumptions and parameters had been verified and would be adopted for the design of the working piles. To facilitate the preparation and piling works, pile design was conducted batch by batch. Each batch would cover couple of piers which involve over 100 piles in general. It depended on the construction rate of foundation.

To design batch of piles, a more effective way was to review all relevant ground investigation information and determine a few representative design ground profiles and SPT N-value design profiles (see Figure 6). The design profiles should consider and cover all possible scenarios. The generalised design profiles were then used to estimate the required pile length based on design axial pile capacity and termination criteria of pile driving as discussed in previous section. The design pile length would be applied to the pile of each pier which had similar geological profile and SPT N-value profile. Since SPT N value was proportional to the shaft and bearing resistance, the design pile length would be adjusted by comparing the ground conditions of the constructed piles.

In foundation design, one of the main challenges was to deal with geological and geotechnical uncertainties. General approach was to make the design more conservative resulting longer pile. The piles might eventually terminate at a shallower depth due to the underestimation of the soil resistance. It would cause considerable amount of abandoned pile especially for the project with substantial amount of pile. In order to minimise the waste of pile material and ease preparation of pile segments, it was targeted to control the difference between as-built and design pile length within 3m. As such, continuous assessment was carried out to review the performance of constructed piles and refine the design approach for next batch of piles.

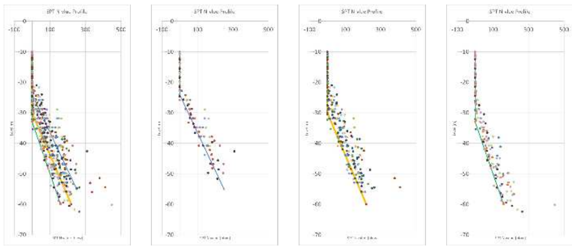

Figure 6. Sample of SPT N-value design profiles for a batch of piles

Brunei Bay and Temburong consist of a thick layer of soil interbedded clay and sand. Sand material generally has higher SPT N value. In some boreholes, a thin succession of sand layer is revealed as presented in Figure 7. When the piles were designed to consider the contribution of this layer of sand, the piles would terminate on this sand layer. Conversely, the pile would be found below this sand layer when the contribution of this sand layer was not fully relied on. The difference of pile length could be up to 10m. For this case, longer pile was designed for the first batch of piles in this ground condition to stand on safe side and minimise the impact to the construction. The reliability of this thin layer of sand would be verified through constructing piles in this location. The design approach to the sequential piles in this area would be refined subject to the performance of the constructed piles.

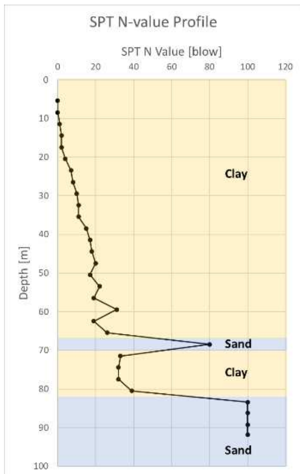

Figure 7. SPT N-value profile of a borehole

2.6.2 *Verification Tests*

Although the workmanship, design assumptions and parameters had been validated via preliminary pile test, additional verification tests were required to ascertain the integrity and capacity of the installed working piles. Therefore, pile integrity test (PIT), PDA and static load tests were assigned and conducted on certain working piles.

PIT and PDA tests were set up relatively earlier, which were conducted at least 10% and 20% respectively of the total number of working piles. PDA test on working piles were carried out immediately after the final set of the selected piles has been achieved. PDA with CASE and CAPWAP analyses shall be carried out on preliminary piles following the same requirements and methods as for the working piles. In case the

PDA test results shows that pile might not achieve the required capacity based on the PDA findings in preliminary piles, further PDA testing or static load test might be required. In addition, static load tests were carried out on at least 1% of the total number of working piles. 150% of DVL was applied in the proof load test on working piles. It was considered successful if the total movement of the pile top is less than 120% of that of the nearest successful preliminary pile under the same load. In case the proof load test on working piles failed to satisfy the test acceptance criteria, remedial measures shall be carried out for the failed piles and two additional proof load tests would be performed on working piles.

3 RESULTS

In this project, over 11,000 spun piles were installed for a 30km sea crossing bridge. The spun piles were driven into extremely soft ground with pile length up to 90m. It took about 3 years to complete all spun piles. For construction of spun pile, it took 1 to 2 days per pile depended on the length of pile.

To minimise the construction cost, it was targeted to control the difference of pile length between design and as-built pile to within 3m tolerance. During the construction, foundation design was continuously reviewed and refined based on the data of constructed piles. As a result, 74% of total working piles had the difference less than 3m (see Figure 8). The rest of piles having larger difference that mainly came from the pilot batches of piles, when there was only limited experience gained from preliminary piles and installed working piles.

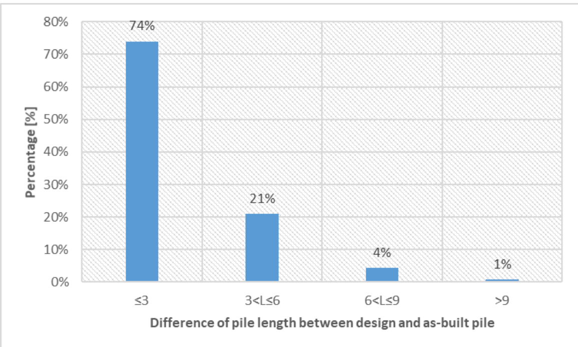

Figure 8. Distribution of the difference of pile length between design and as-built pile

4 FINDINGS AND RECOMMENDATIONS

Throughout the design and construction of substantial spun piles, some findings and recommendations on planning and design for future project are summarised as follows:

1. For such long bridge project, sufficient ground investigation should be carried out to reveal the ground conditions and capture spatial variation for foundation design. Conducting ground investigation in north and south bounds of the bridge in alternate way would be a reasonable option.
2. For preliminary pile test, instrumented piles would be driven into ground by hammering. During the driving process, the strain gauges installed on the pile might be damaged and malfunctioned. It will severely influence the back-analysed results which will be applied to the working pile. It is suggested to reduce the vertical spacing and increase number of measuring device. In the meantime, strain gauge reading should be checked with the reading from adjacent strain gauges to ensure back calculated pile force and shaft resistance are reasonable.
3. In this project, static load tests were carried out on the preliminary piles to check the actual pile axial load capacity. It was then used to verify the design methodology. However, most of the preliminary piles could not be tested to have total movement of the pile top exceeding 10% of the based diameter while the maximum specified load had been reached. It results in the over-conservative design parameters for the working piles. To ensure the preliminary piles failed in proof load tests, it is suggested to apply much higher specified load for the test.
4. Since there were not much experience or reference on installation of spun pile to this ground conditions, some spun piles were broken during the pile installation which caused additional time for reinstatement. It is suggested to closely liaise with the contractors for more evaluation and understanding of method of construction, site conditions and the surroundings. So that a more practical and appropriate construction method would be adopted to reduce the construction risk.
5. Given that SPT was carried out in equal spacing, summation of SPT N value along the depth till the toe of preliminary pile can give a reasonable estimation of pile length for working piles with the same method.
6. During the installation of preliminary pile, it is suggested to carry out real-time PDA test. From the PDA test, estimated pile axial capacity including shaft resistance and end bearing can be obtained at various depth. They are useful for back-analysing design parameters and having a better prediction for working piles.

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