Large Diameter Open-end Steel Piled Foundations for the Hong Kong Offshore LNG Terminal – Design and Installation

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ABSTRACT

Large diameter tubular piles are the most common offshore foundation type in the energy sector due to their relatively easy installation compared to other methods, yet local experiences with regards to their design and offshore installation are still limited. Successful installation of pile foundation on the Hong Kong Offshore LNG Terminal (HKOLNGT) Project provides valuable experience for future offshore developments in the territory. Unlike onshore piling works, offshore piling works are heavily limited by the available machinery, site constraints and weather conditions. This Paper shares the experiences gained on the HKOLNGT Project and draws together solutions to several challenges pertaining to the design and offshore installation of large diameter pile foundations, such as limitations arising from offshore environmental conditions.

Keywords: Offshore foundation, Tubular piles, LNG terminal

1 Introduction

To support the energy transition in Hong Kong, and as part of the HKSAR Government's Climate Action Plan 2050 that targets to increase the use of natural gas for power generation, the two local power companies, CLP Power Hong Kong Limited and The Hongkong Electric Co., Ltd., are jointly developing an offshore liquefied natural gas (LNG) import facility using Floating Storage Regasification Unit (FSRU) technology. The proposed Hong Kong Offshore LNG Terminal (HKOLNGT) will enable procurement of LNG from global markets improving Hong Kong's long-term natural gas supply stability.

When in operation, a FSRU vessel of up to 263,000m³ storage capacity and a supplying LNG carrier will be double berthed in parallel at the proposed offshore jetty, located in the southern waters of Hong Kong SAR, to the east of the Soko Islands. The offshore Jetty Terminal, totalling approximately 385m in length, is formed by six steel mooring dolphin substructures (also termed jackets) and three larger central breasting dolphin substructures, each measuring 50m by 30m on plan housing all the equipment for operating the facility. The breasting dolphins are supported on vertical 1.83m OD cold-formed steel tubular piles driven to tentative 65m depth below the seabed. While, the mooring dolphins are supported on piles of the same diameter but raked at 1v:6.25h and of slightly shallower embedment. The smaller dolphin substructures provide mooring anchorage points for the berthed vessels. Each dolphin substructure will be decked over by prefabricated steel topside superstructures, which will be interconnected by access walkways at the topside level at nearly 16m above sea level. In addition, vertical 1.26m OD steel tubular piles support two smaller mooring dolphins at the northern end of the jetty to provide mooring for larger crew boats, including fireboats, visiting the jetty. The regasified natural gas will be delivered to the Black Point Power Station and Lamma Power Station via 45km and 18km subsea pipelines, respectively. **Figure 1** to **Figure 3** show the location and general arrangement of the proposed jetty.

The following sections start with the overall geological setting of the Project, followed by design aspects of the offshore pile foundations with focus on the capacity analysis as the deformation of

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offshore structure is generally less of a concern compared for example to onshore buildings and is not the focus of this paper. The paper moves on to the discussion of the high-strain dynamic loading tests using Pile Driving Analyzer (PDA) and CAPWAP (Case Pile Wave Analysis Program) analysis and comparison with CPT-based design methods. An observed site-specific pile capacity set-up is discussed and a site-specific set-up curve developed during the project is presented, followed by a discussion of the construction methodology at the end of this paper.

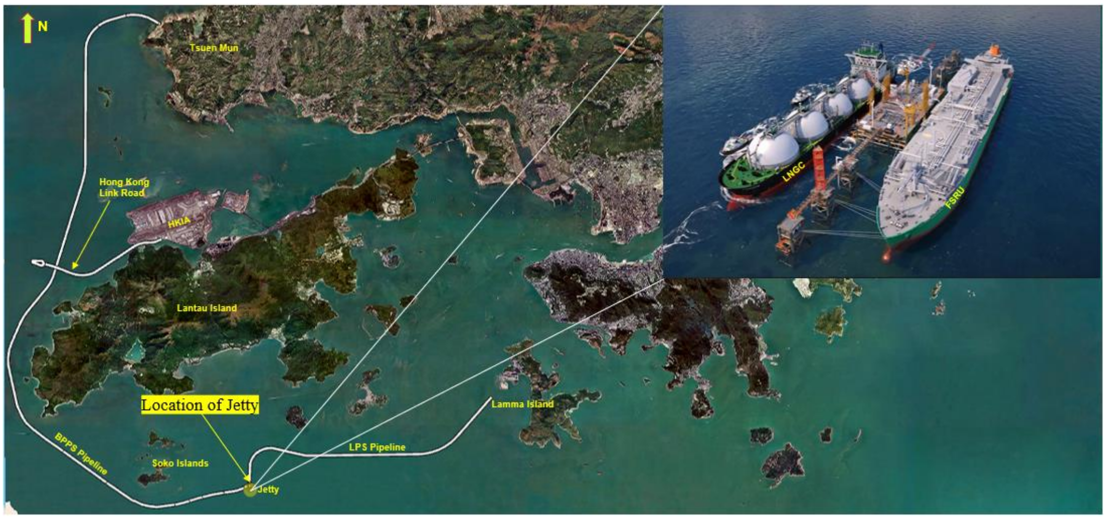

Figure 1: Location Plan of Jetty and Associated Pipelines

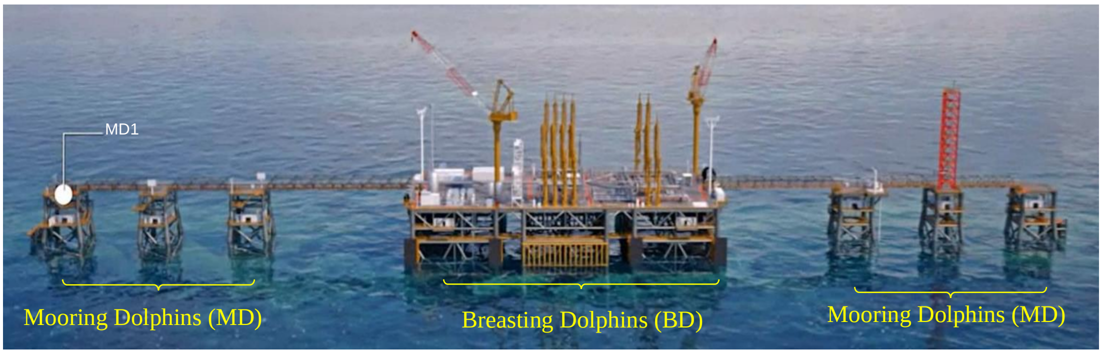

Figure 2: General Arrangement of Jetty

Figure 3: Site Photo of Jetty

2 Site Geological Condition

Site investigation results indicate that the seabed is underlain by 10m to 15m of Marine Deposits (plastic Clays), followed in sequence by Upper Alluvium (Clay/Silt), Interbedded Alluvium (plastic Clay/Sand and Silts), Alluvial Sand (thick dense Sand) and Lower Alluvium (consists of Clays, Silts

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and Sands). Completely Decomposed Granite (CDG) is encountered at approximately -100mPD, whilst rockhead (Grade III or better) is typically greater than 90m depth (at approx. -110mPD).

The piles of the Jetty are all founded on the dense to very dense Alluvial Sand. A geological longitudinal section based on the available ground investigation stations at the Jetty location is shown in **Figure 4**.

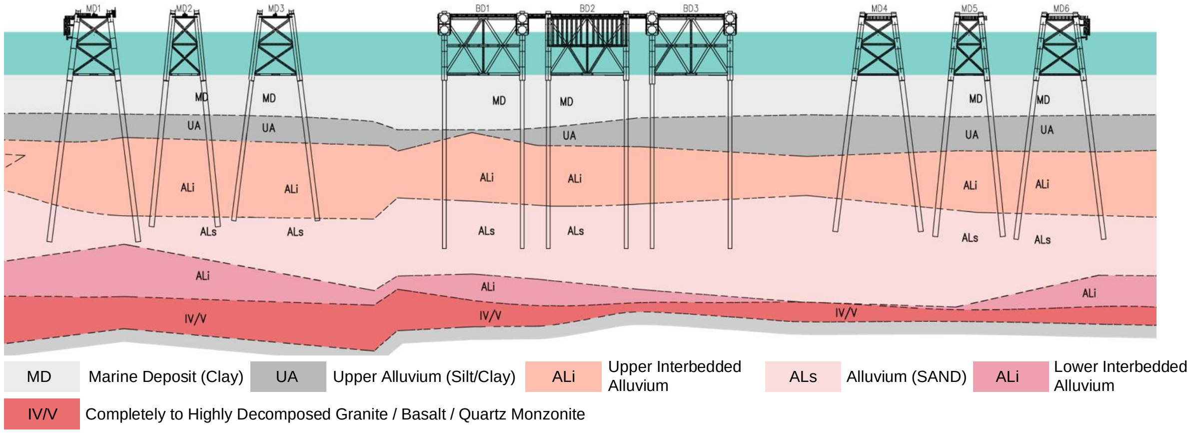

Figure 4: Geological Condition Over the Jetty Site

3 Design of the OFFSHORE Pile Foundation

Tubular piles are the most commonly used offshore foundation type in the oil and gas industry, and considerable experiences of designing such foundation type are reflected in the international codes (API, 2011; DNVGL, 2017). However, the use of large diameter tubular piles in offshore geological condition is less common in local practice and tubular piles are considered non-recognised pile type by the local authority. In this project, two separate designs have been undertaken satisfying the relevant international codes and standards to align the design to international practice, as well as a separate design following the Hong Kong SAR local code of practice to conform to the established local practice, make use of established local experience and obtain design approval by local authorities. In terms of the geotechnical design of the piled foundation, the American Petroleum Institute (API) codes (API, 2011, 2014) are primarily adopted. Reference is also made to the Hong Kong SAR local codes/standards where appropriate to perform checking to fulfil the requirements of local authorities for obtaining statutory approvals.

3.1 Design Factor of Safety

For a pile foundation design according to GEO Publication No. 1/2006 (GEO, 2006), static load test should be performed on preliminary piles, which is usually not practical for offshore developments at project level. This is because the required test loads for offshore piles would be large due to substantial design environmental loading from wave and wind, and the test itself can be both impractical and costly. In offshore foundation practice, a design factor of safety (FOS) of 2.0 is usually adopted for the normal condition. For more onerous extreme design environmental conditions, a lower FOS of 1.5 is deemed acceptable considering the likelihood of the extreme case is less compared to normal operating conditions. If a higher test load to achieve a higher FOS of 3 is desired, static loading test will become practically impossible to conduct for high-capacity offshore piles.

The recommended FOSs in the offshore practice are acknowledged to be less than those typically adopted for onshore foundation practice, which may be due to offshore platforms being less sensitive to displacement for serviceability limit state conditions and are usually unmanned during the design storm events (Lehane et al., 2005).

In Hong Kong, a larger design FOS of 3 and preliminary pile(s) to be load tested are usually required (GEO, 2006) for new or less commonly used pile types to increase the confidence of the design. For piles which will be subjected to some form of proof testing to verify their pile capacities, a high FOS of 3 may not be necessary from a technical point of view. For offshore piles, PDA tests with CAPWAP analyses can be and are often conducted during and after installation for assessing the capacities of installed piles. Such pile testing technique is now a widely accepted reliable proof loading test method in the offshore piling industry as an alternative to static loading test (Webster et al., 2008; Yu et al., 2013). In this project, the jetty will be unmanned, all vessels will be disconnected from the dolphins when typhoon signal No. 3 or higher is hoisted and all the piles have been tested by PDA with CAPWAP analysis during driving and restrike after a period of time as proof load tests. In order to satisfy local practice, it is required that all piles in the Project be designed and tested by PDA with CAPWAP analysis to a FOS of 3.0 for the long-term loading conditions. However, as suggested in the following discussions, site-specific pile set-up behaviour could be considered as a part of the pile capacity verification process to achieve cost-effective offshore pile foundation designs in Hong Kong.

3.2 Pile Axial Load-carrying Capacity

Tubular piles can behave plugged or unplugged depending on the pile configuration and soil resistance distribution (**Figure 5**Error! Reference source not found.). A plugged condition refers to the situation when the internal shaft friction of the soil plugged inside the tubular pile is greater than the base resistance and no relative movement between the soil plug and internal surface of the tubular pile is possible. Generally, for large diameter piles, they are rarely plugged during installation by continuous driving (Rausche et al., 2010) due to the inertia of the soil plug inside and the shaft friction is considerably disturbed by the installation process, whereas under static loading condition, plugged condition can be more dominant for pile with sufficient embedment. The pile capacity derived from the two different assumed mechanisms are given in Eq. (1) & (2).

$$Q_{c_{\mathrm{plugged}}} = Q_{\mathrm{f,c,o}} + Q_{\mathrm{b,p}} = \pi D_{\mathrm{o}} \int_{\mathrm{L}} \tau_f(\mathrm{z})\mathrm{dz} + q_b A_{\mathrm{p}} \quad \textbf{(1)}$$

$$Q_{c_{\mathrm{unplugged}}} = Q_{\mathrm{f,c,o}} + Q_{\mathrm{f,c,i}} + Q_{\mathrm{b,wall}} = \pi D_{\mathrm{i}} \int_{\mathrm{L}} \tau_f(\mathrm{z})\mathrm{dz} + \pi D_{\mathrm{o}} \int_{\mathrm{L}} \tau_f(\mathrm{z})\mathrm{dz} + q_{\mathrm{b}} A_{\mathrm{wall}} \quad \textbf{(2)}$$

where, $Q_{\mathrm{f,c,o}}$ = shaft friction at pile outer surface, $Q_{\mathrm{f,c,i}}$ = shaft friction at pile inner surface, $Q_{\mathrm{b,p}}$ = toe resistance from gross area of pile base, $Q_{\mathrm{b,wall}}$ = toe resistance from pile wall area, $\tau_f(\mathrm{z})$ = shear stress developed at failure along shaft, q_b = base resistance pressure, A_p, A_{wall} = pile gross bases area and wall area respectively, and D_i and D_0 are the inner and outer diameters of the pile.

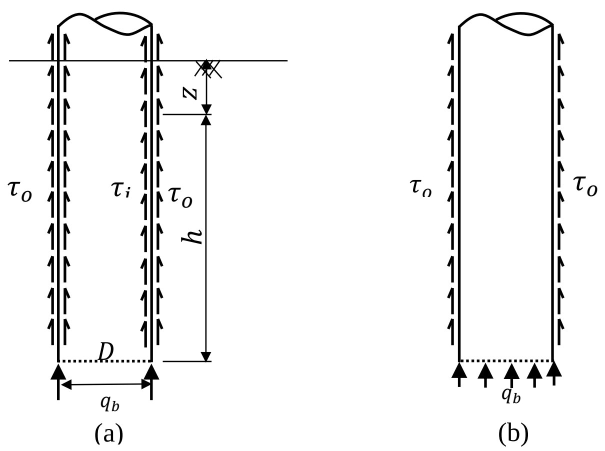

Figure 5: Load-transferring mechanism for tubular piles (a) unplugged case and (b) plugged case

In the API RP 2GEO Code, the traditional pile capacity calculation method is referred to as the Main Text Method. For the simplicity of design, the API Main Text method recommends both cases be

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checked and the lesser of the calculated capacities shall be adopted in the design. Furthermore, the same shaft friction is assumed for both internal and external friction at a given depth in the calculations, which is conservative as some degrees of arching will be developed by the driving process that tends to create higher inner shaft friction than the outer.

3.3 Conventional Driven Pile Design Approach

In terms of the effective stresses, the shaft friction τ_f along a pile can be expressed as Eq.(3).

$$\tau_f = \eta \cdot \sigma'_{r0} \cdot \tan\delta_f = \eta \cdot K_c \cdot \sigma'_{v0} \cdot \tan\delta_f \quad \textbf{(3)}$$

where, σ'_{r0} = radial effective stress, $K_c = \sigma'_{rc}/\sigma'_{v0}$, η = resistance adjustment factor, and δ_f = interface angle.

For cohesive soils, a total stress analysis method is conventionally adopted for its simplicity without considering the complex stress development. The shaft friction can be related directly to the undrained shear strength of soils s_u, known as the alpha method. Such simplification has been stated to have limitations in principle (Jardine et al., 2005). However, such formulation is the current industry standard in design and have been adopted both in the main text of API as well as referenced in the GEO Publication 1/2006. For the granular soils, the term $\eta \cdot K_c \cdot \tan\delta_f$ is lumped to β and it is known as the beta method, which directly relates the shaft friction to vertical effective stresses. The pile shaft friction can be expressed as Eq.(4).

$$\tau_f = \begin{cases} \alpha \cdot s_u \leq \tau_{s,\mathrm{lim,clay}} & for\ cohesive\ soil \\ \beta \cdot \sigma'_{v0} \leq \tau_{s,\mathrm{lim,sand}} & for\ granularsoil \end{cases} \quad \textbf{(4)}$$

In addition, a limiting shaft resistance $q_{b,\mathrm{lim}}$ is further introduced to cap the shaft friction that can be utilised in design. Using the limiting shaft friction could be misleading (Kulhawy, 1984) but nonetheless it has been used in the present design in accordance with the common practice in Hong Kong. API RP 2GEO recommends a limiting shaft friction of 120kPa for very dense sand, whereas in GEO Publication No. 1/2006 higher shaft friction of 150kPa has been suggested for bored piles in granite saprolites. The seemingly larger observed shaft limits suggested for onshore piles could be attributed to a) API Main Text Method developed the limiting values based on un-instrumented piles, the limiting values are more of a fitting tool rather than the actual shaft friction and b) discussion within GEO Publication No 1/2006 are based primarily on land piles tested at a much later time after installation, whereas the loading test for marine piles referenced in API are usually conducted only a few days after driven due to the harsh offshore environment and machinery availability. Such time-dependence could have a significant effect on the apparent pile capacity, which will be further elaborated on in the later **Section 4.1** of this paper. To assess the long-term pile axial compression capacity, a limiting value of 140kPa has been adopted.

For the base pressure, it is calculated according to Eq.(5).

$$q_b = N_q \cdot \sigma'_{v0} \leq q_{b,\mathrm{lim}} \quad \textbf{(5)}$$

where, N_q = bearing capacity coefficient and σ'_{v0} = vertical effective stress acting on soil at pile base.

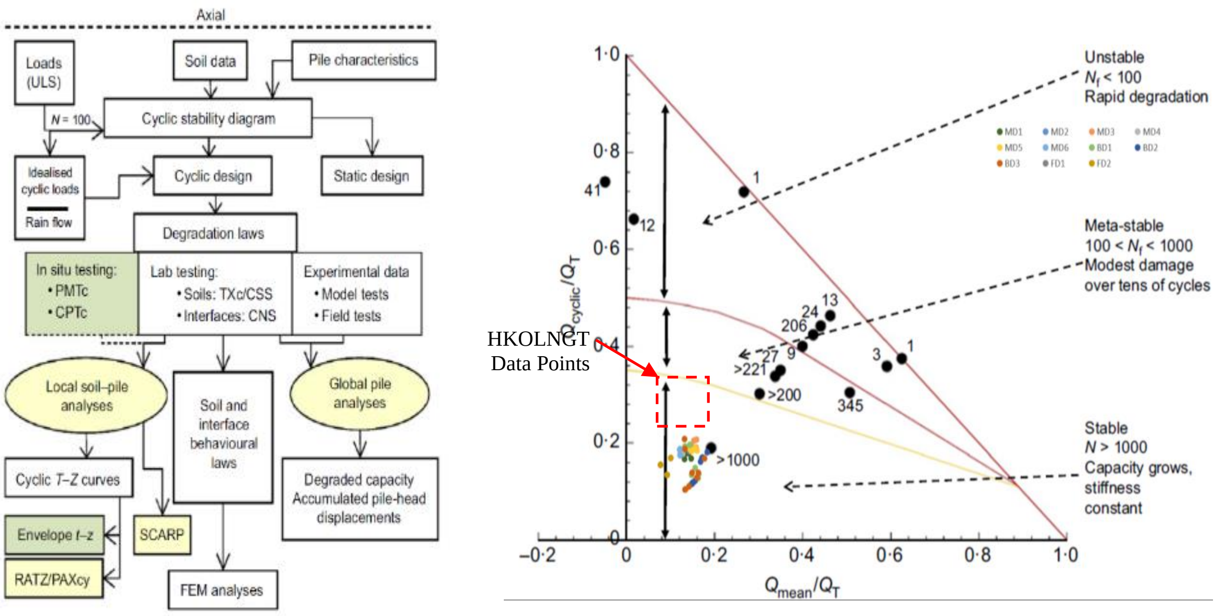
For this project, the piles have an embedment of approximately 60m below the seabed level, it is found the that $q_{b,\mathrm{lim}}$ will be triggered for most of the piles. API Main Text recommends a limit of 10~12MPa for dense to very dense sand. A conservative value of 10MPa has been adopted in the design.

The applicability of limiting values heavily relies on the calibration with the accumulated database. It has been reported in the literature that since the α and β design approaches fail to capture the fundamental failure mechanism of a pile, it is considered less reliable than the modern CPT-based methods (Randolph & Gourvenec, 2011), which have also been compared in this study in **Section 4.2**.

3.4 Consideration of Cyclic Degradation of Pile Capacity

Cyclic effects are commonly researched for monopiles supporting wind turbine generators, which in addition to cyclic environmental loading by predominantly waves, will impose millions of rotating blade cycles onto the foundations (Buckley et al, 2018). As discussed by Jardine (2020), there is currently no internationally recognised design method for the cyclic design for piles. A design workflow chart has been proposed following the Project SOLCYP and involves a screening stage using the concept of cyclic interactive chart or stability chart, which classes the cyclic loading response as stable, meta-stable or unstable (Jardine, 2020; Poulos, 1988; Puech, 2013). When the loading data points fall within the stable zone, it can be decided that a static design approach is adequate without further considering the potential of cyclic load degradation effect on the pile axial capacity.

The foundation system of HKOLNGT is subjected to relatively mild cyclic load amplitude compared to their design ultimate pile capacities. Unlike common offshore piled foundations that are typically designed to FOS of 2, a conservative design FOS of 3 has been used in this Project. This means that the mean and cyclic axial forces under working conditions will be considerably lower than the pile ultimate capacity and soil strains will be limited to a small range that will not be sufficient to trigger cyclic shaft degradation (**Figure 6**).



(a) (b)

Figure 6: SOLCYP Workflow Process for Use in Design of Pile to Carry Axial Pile Loading (Extracted from Jardine (2020)); and (b) Interaction Diagram Developed at Dunkirk test Site (Jardine and Standing 2012, Jardine 2020) with Data Points at HKOLNGT Added

4 Proof Load Test and Design Verification

For large load-carrying capacity offshore piles, it is generally impractical to perform static pile load tests. A series of high-strain dynamic load tests using PDA were carried out as proof load tests and CAPWAP analyses used to derive their static pile capacities. Dynamic load tests have been performed at various times after the end of driving (EOD) to capture the site-specific pile capacity set-up behaviour.

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4.1 Review of Site-specific Set-up Curve

Pile capacity set-up is primarily due to an increase in the shaft capacity over time as indicated by both field and model tests (Bullock et al., 2005; Chow et al., 1998). To represent the set-up behaviour, i.e., an increase in capacity with a decreasing rate, a logarithmic relationship has been proposed for its simplicity in the literature (Axelsson, 1998; Hosseinzadeh Attar & Fakharian, 2013; Komurka et al., 2003; Rausche et al., 2010). Alternatively, a hyperbolic function in the form of Eq.(6) has also found wide application in geotechnical engineering for predicting geotechnical behaviours from available field data, e.g., settlement predictions over time (Chung et al., 2009; Tan, 1995) as well as ultimate pile capacity prediction in Chin's method (Chin, 1972). The hyperbolic function has two unknowns and further caps to $1/b$ as time (t) approaches to infinity as opposed to conventional logarithmic relationship used to assess set-up behaviour on international offshore projects, and thus the former can be viewed as being more reasonable. It should be noted that the actual pile capacity set-up behaviour is complex with an initial relatively faster recovery of pile capacity within the first one to two days after EOD. This is believed to be the result of excess porewater pressure dissipation as well as collapsing of soil arching formed during to pile driving with time. The continual slower growth in the pile capacity can be more attributed to the ageing effect of the soil.

Attempt has been made to capture the set-up behaviour after an initial stage post-EOD, when the set-up response tends to be more consistent and less prone to uncertainties introduced by the driving process. Hyperbolic function in the form of Eq. (6) has been used in this project. It is recommended that for other sites a site-specific predictive curve should be developed based on the observed set-up trend in the available test data over a particular period of development. As shown in **Figure 7(a)**, data presented in the (t/τ_f ,t) space show a good linearity indicating a hyperbolic function is a reasonable representation of the trend.

The project consists of 54 nos. of piles with varying penetration at different dolphins and diameters. To make use of all tested pile capacities, the shaft resistances are normalised by their respective pile outer surface area using Eq.(7). The resultant $\tau_{f,i}$ can be regarded as a representative shaft friction stress. Test data have been plotted in **Figure 7(b)**, which exhibits a consistent trend that can be well described by a hyperbolic function.

$$Q(t) = \frac{t}{a+b\cdot t} \; for \; t > t_0 \text{ (a, b are constants to be determined by data regression)} \quad \textbf{(6)}$$

$$\tau_{f,i}(t) = \frac{Q_{s,i}(t)}{\pi D_{o,i} H_i} \quad \left(Q_{s,i}(t), \text{measured shaft resitance for } i th \text{ pile at time t}\right) \quad \textbf{(7)}$$

where, $Q_{s,i}$(t) = the measured shaft resistance at time t after EOD for i-th pile, $D_{o,i}$ and H_i refer to the pile outer diameter and the embedment, respectively. Re-arranging the data in the (t/τ_f ,t) space and performing a linear regression, the set-up of average shaft resistance can be expressed as Eq.(8) with a coefficient of determination of R^2 approximately 0.90.

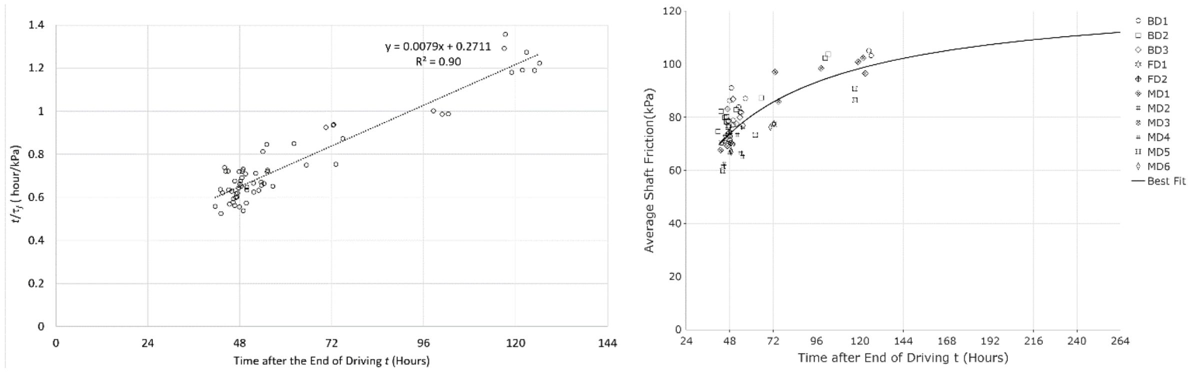
$$\tau_f(t) = \frac{t}{0.0079\cdot t + 0.2711} \; for \; t > 48\text{hours and } \tau_f \text{ in kPa} \quad \textbf{(8)}$$

The set-up effect ratio $\xi(t)$ at a particular time t referencing to a testing time t_0 can therefore be written as Eq.(9). Using a reference t_0 = 48 hour, the set-up curve in terms of a set-up ratio $\xi(t)$ can be derived in Eq.(10).

$$\xi(t) = \frac{\tau_f(t)}{\tau_f(t_0)} = \frac{t(0.0079\cdot t_0 + 0.2711)}{t_0(0.0079\cdot t + 0.2711)} \; for \; t > 48\text{hours and } \tau_f \text{ in kPa} \quad \textbf{(9)}$$

The predicted total pile capacity at any given time t can also be calculated by applying the set-up factor to the measured shaft friction at the time of the testing. Toe resistance set-up was observed at this site, but it is much less significant when compared to the shaft resistance set-up effect, and therefore has been ignored when further interpolating the pile capacities to 5 days, which is conservative.

$Q(t) = Q_{b,0} + \xi(t) \cdot Q_{s,0}$ ($Q_{b,0}$ and $Q_{s,0}$ are base and shaft resistances measured at time t_0) **(10)**



(a) (b)

Figure 7: Shaft Set-up Behaviour (a) t/q vs. t and (b) average shaft friction vs. time

Though pile capacity set-up is a well recognised phenomenon, there has been no consensus in the local industry on how such time-dependent features of pile capacity could be considered in design. It can be seen from this work that pile capacity set-up could be significant in the pile design as well as in the proposal of the proof load test regime for future offshore development in Hong Kong.

4.2 Cone Penetration Test (CPT) Based Design Method

To remove the inherent limitation of the current design approach, as highlighted in the preceding section, many international design codes are moving towards CPT-based pile design. In these methods, the concept of the shaft and toe resistance limit has been removed and a length factor h/D has been introduced to account for the effect of pile installation as well as the loading phases (Randolph, 2003). Several direct CPT methods have been proposed in the literature and four (4) CPT methods have been included in API RP 2GEO. Among the four methods, Method 1, ICP -05 (Jardine et al., 2005) and Method 2 UWA-05 (Lehane et al., 2005) have received more research attention in subsequent developments.

It should be noted that the four API CPT methods are developed for silt/sand. Typical Hong Kong offshore geology consists of a layer of marine deposit overlying alluvial deposits which are more variably composed. At the site of Jetty Terminal, the alluvial layer consists of interbedded silts and sands, occasionally clay followed by a relatively uniform dense to very dense alluvial sand where the tubular piles are toed in.

Given the layered geological profile at HKOLNGT, both CPT methods fitted for the clay site and sand sites are considered for the respective layers. Discussion on design method within clay has been provided in Jardine et al. (2005) and a simplified version adopting CPT methods is then described in the works of Lehane (Lehane et al., 2000; Lehane et al., 2013). The UWA-13 method (Lehane et al., 2013) developed for clay is appropriate where clay is encountered, and the same approach has been employed in the work of Lehane et al.(2017) for sites with the presence of both sand and clay. As the UWA-05 is largely developed for silt/sand from the ICP-05 with several modifications and have been demonstrated to provide a better predictive performance with the available field test results (Labenski & Moormann, 2016; Schneider et al., 2008), this method has been adopted in this study to provide an indication of the pile axial capacities. The simplified version of UWA-05 as presented in API RP 2GEO is shown in Eq.(12), which is considered a reasonable simplification in offshore applications (API, 2011).

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$$\text{UWA}-13\ \text{(Clay)}:\ \tau_f = 0.055 q_t \left[\max\left(\frac{h}{R^*}, 1\right)\right]^{-0.2} \quad \textbf{(11)}$$

where $R^* = (R^2 - R_i^2)^{0.5}$ *for shaft resistance*

$$\text{UWA}-05\ \text{(Sand)}: \begin{cases} \tau_f = 0.03 \cdot q_c \cdot A_r^{0.3} \cdot \max\left(\frac{h}{D}, 2\right)^{-0.5} \tan \delta_{cv} & \text{for shaft resistance} \\ q_b = q_{c,av1.5D}(0.15 + 0.45 A_r) & \text{for toe resistance} \end{cases} \quad \textbf{(12)}$$

where, $\boldsymbol{h}$ = the distance between the points at z and the pile tip. δ_{cv} is the constant volume friction angle, $\boldsymbol{A_r}$ = $1 - (D_i/D_o)^2$, the pile displacement ratio, $q_{c,av1.5D}$ = average cone resistance q_c over 1.5D above and 1.5D below the pile tip. q_t = corrected cone resistance.

As shown in Eq.(12) , UWA-05 does not distinguish the plugged or unplugged case for the toe resistance and an average of q_c over 1.5D above and below the toe tip has been used. In addition, as discussed earlier, as the unplugged case under the working condition is atypical for large diameter piles. It should also be noted the shaft friction derived from the CPT-based design method does not differentiate the internal and external resistances for the design equations which are derived from fitting the database of pile load tests.

12 nos. of CPTs are available over the jetty area, five (5) of which have a penetration deeper than 60m below the seabed level and can be used for the estimate of the static pile capacity. The calculated pile capacities based on CPT method across the jetty are generally in a range of 35~40MN depending on the local variation in geological conditions as well as the pile length. It is noted that the calculated FOSs are between 2.03 and 2.63, greater than 2.0 required in international practice for normal conditions. The subsequent PDA tests on site indicate that the pile capacities typically attained a FOS greater than 2 at around two days after EOD, which suggests that CPT-based method can provide a reasonable estimate of the pile capacities after a relatively short time after installation.

The further dynamic load tests had demonstrated that all piles achieved at least FOS of 3 at the time of around five (5) days after pile installation due to the additional pile capacity set-up effect. It is noted that the site-specific set-up behaviour and the time elapsed after EOD have important influence on the pile capacity obtained at the time of PDA testing. The extensive high-strain dynamic loading tests at HKOLNGT indicate that the CPT-based design methods provide a reasonably conservative estimate of long-term pile capacity. It is suggested that the CPT-based method can be explored in future offshore pile foundation designs in Hong Kong SAR along with a FOS of 2.0 to align the designs with the international offshore practice.

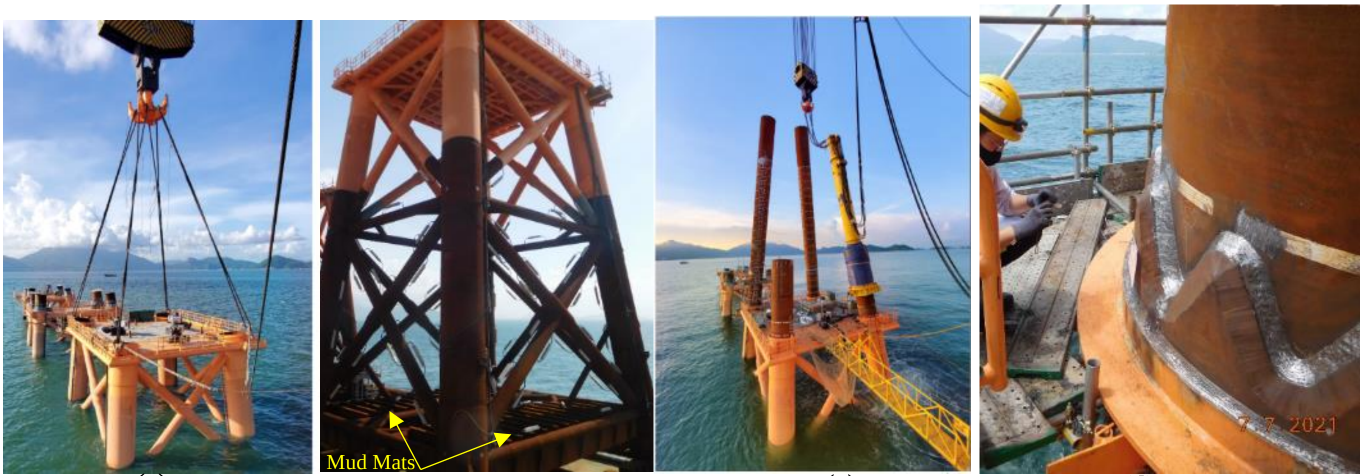
Table 1: Pile Axial Compression Capacity using CPT-based Design Method

Pile Location	**Maximum Working Compression Load [kN]**	**CPT-based Capacity* [kN]**	**FOS**
MD1	14,919	30,253	2.03
MD2	12,177	27,025	2.22
MD3	11,911	26,070	2.19
MD4	11,397	29,952	2.63
MD5	12,351	30,811	2.49
MD6	12,645	31,584	2.50
BD1~BD3	18,446	38,351	2.08

*Note: * Compression capacities based on UWA-05 and UWA-13*

5 Installation

The dolphin jackets and piles were fabricated in Qingdao, Mainland China, which were towed to site on large delivery barges for installation. During the installation, the jacket was first lowered onto



(a) (b) (c) (d)

the seabed, and was temporarily supported on the seabed by two large mudmats prefabricated onto the base of the jackets, **Figure 8(a) & (b)**. Once the jacket was lowered to be seabed, it would be difficult to adjust its location or orientation hence for this reason and to ensure safety of the lifting works, the offshore installation required a calm sea state.

Figure 8: Jacket Structure Installation (a) Lowering Jacket; (b) Temporary Mud Mats; (c) Pile Driving; and (d) Welding of the Crown Shim Plate

After the jacket was placed at the design location, the first segments of the corner piles were pitched into the jacket legs and driven to their specified levels using offshore hydraulic hammers. The second pile segments are stabbed into the first segments, welded together, tested by non-destructive tests and driven to their design founding levels, **Figure 8(c)**. The driving process was monitored using the PDA system to check that the maximum driving force did not exceed the allowable limit and on this project 0.9 times the steel yield strength has been adopted. When the PDA test results are proven satisfactory, the annulus between the jacket leg and pile is grouted and the top of the jacket connected to the piles by 100mm thick steel crown shim plates, **Figure 8(d)**.

5.1 Limitation due to Environmental Condition

Based on the Contractor's international offshore experiences, site characteristics and heavy-duty installation vessel, with up to 3,800te lifting capacity, deployed on this Project, the limiting environmental conditions for various construction activities are shown in **Table 2**. The significant wave height, H_s, heavily restricts jacket installation works. During the construction of dolphin MD1, only the first 4 days out of the 14-day construction period from 10 to 24 December 2020 recorded H_s < 1.0m, suitable for jacket lowering, which caused a considerable impact on the construction programme.

Long waves (typically peak wave periods, T_p > 6.7s) have a significant impact on the rolling motion of the installation vessel. Several precautions had to be implemented during the installation because of adverse weather conditions. For example, the primary hook was removed as it collided with the ancillary hook under severe rolling, slight adjustment of vessel orientation, and deployment of additional anchorage to achieve better stability of the vessel. Despite these enhancement measures, some of the hammer lifting and pile splicing works still had to be temporarily halted because of excessive vessel heave/roll.

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Table 2: Limiting Environmental Condition of Installation Vessel

Construction Activity	Wind Speed	Significant Wave Height, H_s	Current Speed
	[m/s]	[m]	[knot]
Jacket Lifting	≤10.0	≤1.5	≤2
Jacket Lowering	≤13.8	≤1.0	≤0.88
Piling	≤12.0	≤1.5	-

5.2 Limitation due to the Anchorage Extent

Unlike onshore piling works, the offshore construction sequence and vessel manoeuvrability are limited by the anchorage spread arrangement making it difficult/infeasible for the installation vessel to readily return to the previous location(s) due to interference between anchorage lines and completed dolphins. In addition, the long extent of the anchorage spread of the installation vessel, shown in **Figure 9**, means that it is not physically feasible to mobilise a second installation vessel of equal size to work concurrently to accelerate the works. Therefore, the dolphin jackets and piling works must be carried out in a predefined sequence with the required proof testing completed at each location before moving to the next one.

Figure 9: Marine Installation Constraints

The installation vessel will have to stay idle for a period until the pile capacity recovers after driving, which may require multiple restrike PDA tests. This could generate considerable loss of valuable fair-weather windows. Consequently, the installation sequence must be well-planned in advance with adequate allowance in the programme for weather related downtime, so that the programme can be adjusted with some flexibility against the closely monitored offshore weather forecast.

6 Conclusions and Recommendation

This paper discusses the design and installation aspects of a jetty foundation supported by large-diameter steel tubular piles. Experiences drawn by the authors in this project include:

1. The international offshore codes have been shifting to CPT-based design methods for the design of offshore pile foundations. The modern CPT-based methods provide a reasonable estimate of pile axial capacities soon after completion of pile installation, which at the HKOLNGT site approximate to the CAPWAP capacities obtained at around two days after end of driving.
2. The established local experience on onshore piles should be considered together with the time-dependent effect of pile set-up, which will affect the proof loading test proposal. An appreciable

pile capacity set-up has been observed in this project. A hyperbolic set-up curve has been developed that shows a consistent set-up behaviour across the site.

3. It is suggested that CPT-based methods could be used along with a FOS of 2.0 in future offshore developments in Hong Kong SAR to assess the design pile capacity soon after pile installation coupled with the consideration of site-specific pile capacity set-up behaviour to determine the longer-term pile capacity.
4. The use of PDA testing with CAPWAP analysis could be maximised to obtain field data on the pile capacity development at various times after end of driving to verify the set-up response, which can be very site dependent. In this way, safe and cost-effective offshore piled foundation designs can be achieved in Hong Kong SAR.
5. The installation of offshore pile foundations is heavily affected by the harsh marine environmental condition, which is further restricted by non-piling window requirements implemented to protect marine mammals, therefore a flexible and adaptive construction programme is critical to the successful completion of the project.

7 Declarations

7.1 Acknowledgements

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