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Prediction of Long Term Capacity using Dynamic Testing for Underwater Skirt Pile Foundation

Liqun Liang,[1] *Scott Webster,*[2] *Ruhua Yuan,*[3] *Hongxing Tian,*[3] *Yanbo Han,*[3] *Chao Wang,*[3] *Liang Yu,*[3]
[1] Pile Dynamics Inc., Cleveland, Ohio, USA
[2] GRL Engineers Inc., Cleveland, Ohio, USA
[3] Installation Division, Offshore Oil Engineering Co., Ltd.
Tanggu, Tianjin, China

ABSTRACT

This paper presents the application of HSDT to dynamically monitor the underwater pile foundations of the PY34-1 CEP Jacket in South China Sea. The eight-legged CEP Jacket, standing in a water depth of approximately 190.2m, is secured to the seafloor with 16×ø108" (2743mm)×148.3m pipe piles. HSDT has been used to measure soil resistance at the end of initial driving and during restrike driving at different waiting time periods. Studies on the SRD have been performed to predict LTSR and establish the relationship between LTSR and SRD in this area, which can be used to optimize future design and driveability analysis.

KEY WORDS: Dynamic pile testing; signal matching; CAPWAP; deep foundation; soil resistance; underwater.

NOMENCLATURE

CAPWAP®	=	CAse Pile Wave Analysis Program
CEP	=	Central Processing Platform
PDA	=	Pile Driving Analyzer®
HSDT	=	High Strain Dynamic Test
SRD	=	Static soil Resistance to Driving
LTSR	=	Long Term Static soil Resistance
EOID	=	End of Initial Drive
BOR	=	Beginning of Restrike

INTRODUCTION

Recently more and more large and mega large jackets have been installed in the Asia Pacific area, such as South China Sea, Gulf of Thailand, Western Australia, and so on. The design and installation of jacket pile foundation becomes more and more challenging due to complicated soil strata, large pile diameter, very long and heavy piles, driving under deep water, etc. During the pile driving, the major concerns include pile damages, hammer performance, driveability in various soil strata, as well as pile bearing capacity. High Strain Dynamic Testing techniques (HSDT) have been used for over 40 years to monitor pile installation, to determine Static soil Resistance to pile Driving (SRD) and evaluate Long Term Static soil Resistance (LTSR) by restrike (Rausche et al., 1972; Goble et al., 1975). Due to its advantages such as quickness, portability, capabilities to evaluate the stresses, energy transferred, potential damage and soil resistance distribution, plus the availability of large and heavy duty pile driving hammers, HSDT has been the best solution to address the concerns involved in the design and installation of offshore piles.

There are several other methods available to determine the bearing capacity of a driven pile, such as static load test and rapid load test. These methods cannot be used to determine driving stresses, evaluate the hammer performance, and have rarely been used in offshore environment due to lack of feasibility. These methods cannot be used for underwater test.

Since HSDT was introduced in early 1970's at Case Western Reserve University, the Case Method (named after the university), practice, procedures, and equipment for high strain dynamic pile testing have evolved considerably (Likins et al., 2009) and the method has now become the standard of practice for evaluation of driven pile foundations, as well as cast-in-place shafts. Dynamic testing is required by various specifications and codes (Beim and Likins, 2008) worldwide.

HSDT has also been used in offshore piling for several decades (Hussein et al., 1989) to monitor/accept the jacket foundation and wind turbine piles (Webster and Givet, 2010; Webster et al., 2008; Schallert and Klingmüller, 2012). So far, HSDT has been mainly used for above water driving conditions. It is much more challenging to apply this method to underwater driving due to various environmental factors such as water proof concerns for transducers, cable and connections, difficulty in cable handling due to its weight and interference with other equipment, such as Remotely Operated Vehicles (ROV). A successful application of HSDT for underwater testing as deep as this was presented by Yu et al., (2013). However, there was no restrike test performed to determine the LTSR.

Due to lack of load testing results in the South China Sea area, the long term soil resistance (LTSR) of a pile and soil resistance to driving (SRD) during installation can only be estimated based on the soil boring and laboratory tests. During actual installation, both very easy driving and refusal conditions could be experienced and cause concern with respect to the pile capacity. To verify the LTSR, restrike tests are needed to help establish the relationship between LTSR and SRD (soil setup-factors) which can be used to optimize future design and driveability analysis in this area.

PROJECT BACKGROUND

The PY34-1 Gas Field is located in the eastern part of South China Sea and about 300 km southeast of Hong Kong. The PY34-1 Central Processing Platform (CEP) located about 33 km away from the LW3-1 CEP is an 8-leg platform with production treatment facilities and living quarters, and is mainly designed to process the oil, gas and water from Panyu Gas Field, which is then transported to LW3-1 CEP. The mixed dry gas and dewatering condensate are processed and boosted at the LW3-1 CEP, before sending into a shallow water subsea pipeline leading to the onshore terminal gas plant in Zhuhai, near Macao.

The derrick barge Lanjing owned by COOEC (China Offshore Oil Engineering Co. Ltd.) with a lifting capacity of 7,500 tons was used to install the CEP jacket. During the installation, there was one pile driving hammer available on the derrick barge. The Menck MHU 1200S is a double acting hydraulic hammer, with a ram weight of 648.4 kN, an equivalent stroke of 1.85 m which results in a maximum rated energy of 1,200 kJ. No hammer cushion is used with the Menck driving system. This derrick barge was equipped with two ROVs, where one was used to monitor pile penetration near sea bed and another was used to check hammer and top of pile.

Foundation Details

The PY 34-1 CEP Jacket standing in a water depth of approximately 190.2 m, is secured to the seafloor with 16 foundation piles driven through four vertical skirt sleeves at each corner leg. The piles consist of Ø2,743 mm O.D. steel pipe piles with wall thickness ranged from 55.0 mm to 90.0 mm over the pile length. The bottom 2.0 meters of pile length formed the driving shoe, which consisted of the same Ø2,743 mm O.D. section with 90.0 mm wall thickness. The piles consist of one section 143.3 m long and were driven to the design penetration of 122.8 meters. Each pile weighs 623 tons and the design bearing capacity is approximately 95.7 MN.

Soil Characterization

The 150 m soil boring performed for this site indicates that that the predominant subsurface conditions consist mainly of soft to very stiff clay interlayered with medium dense to dense sand and sandy silt. At the design pile penetration (122.8 m) the soil boring indicates a transition from hard sandy clay to dense clayey fine sand. The dense clayey fine sand continues to a depth of 132.0 meters where it transitions to dense sandy silt. The dense sandy silt continues to the boring termination depth of 150 meters. Details of the soil stratigraphy are listed in Table 1.

Based on this soil profile, a geotechnical analysis performed by COOEC based upon API code indicated that the pile should reach a LTSR of 95.7 MN at a penetration of 122.8 m. Since the pile just stops at top of the dense clayey fine sand layer, there is not much toe resistance. Therefore, the majority of static resistance is from skin friction between 40 and 122.8 m below mudline

Due to the heavy weight of pile and hammer, the estimated self penetration was approximately 24.3 m.

During pile driving, the soil properties change and the pile encounters SRD, which is usually less than LTSR and the ratio between LTSR and SRD is called soil setup. Prior to installation, the wave equation analyses were performed by COOEC to check the driveability and stresses. To account for the uncertainty in SRD, following cases were analyzed using Menck MHU 1200S with an assumed hammer efficiency of 85% to account for underwater driving condition:

1) Lower coring: 25% and 100% of the static soil skin friction were assumed as SRD for clays and sands respectively. No plug and no loss of resistance were assumed for pile tip, i.e., only the steel area of pile tip was included into the computation of toe resistance. The estimated blow count at end of drive is about 82 blows/m with a total SRD being about 46.1 MN
2) Upper coring: 40% and 100% of the static soil skin friction were assumed for clays and sand respectively. No plug and no loss of resistance were assumed for pile tip, i.e., only the steel area of pile tip was included into the computation of toe resistance. The estimated blow count at end of drive is about 108 blows/m with a total SRD being about 56.0 MN
3) 100% plugged: 100% of the static soil skin friction for both clays and sands. A fully plugged pile toe and no loss in the resistance at toe were assumed. This is the most difficult driving situation as a upper bound solution and the refusal is reached when the penetration reaches about 120 m with an estimated blow count of about 693 blows/m and a total SRD being about 98.0 MN.

Table 1. Site Soil Condition

Depth (m)		Description
From	To	
0.0	6.6	Loose to medium dense silty SAND
6.6	12.5	Firm silty CLAY
12.5	19.6	Laminated or interlayered firm silty CLAY and medium dense SILT/sandy SILT
19.6	42	Firm to stiff silty CLAY
42	47.2	Dense to very dense fine SAND and silty fine SAND
47.2	51.3	Stiff silty CLAY
51.3	59.3	Medium dense to dense silty fine SAND and fine SAND
59.3	69.7	Stiff silty CLAY
69.7	82.8	Laminated dense SILT, sandy SILT and very stiff silty CLAY
82.8	91.4	Laminated dense SILT, sandy SILT and very stiff silty CLAY
91.4	103	Very stiff silty CLAY
103	119.2	Very stiff silty CLAY
119.2	122.8	Hard sandy CLAY
122.8	132	Dense clayey fine SAND
132	150	Dense sandy SILT

DYNAMIC MONITORING

The HSDT field equipment and procedure were basically the same as what used for LW3-1 DEP and were reported by Yu et al. (2013) in detail. The pile driving hammer, Menck MHU 1200S, was used to drive all piles.

Fig. 1 shows the deck layout sketch of DB Lanjing including the relative locations of major equipment (ROVs and hammers) which were operated underwater, the plan of the jacket and pile numbers. In this sketch, the barge was assumed in the location to drive piles along Row 1 of the jacket, i.e., the piles at legs A1 and B1. The location on DB Lanjing labeled as T1 was used to lay out the UW (Under Water) main cable to water to help avoid cable interference with ROV and hammer.

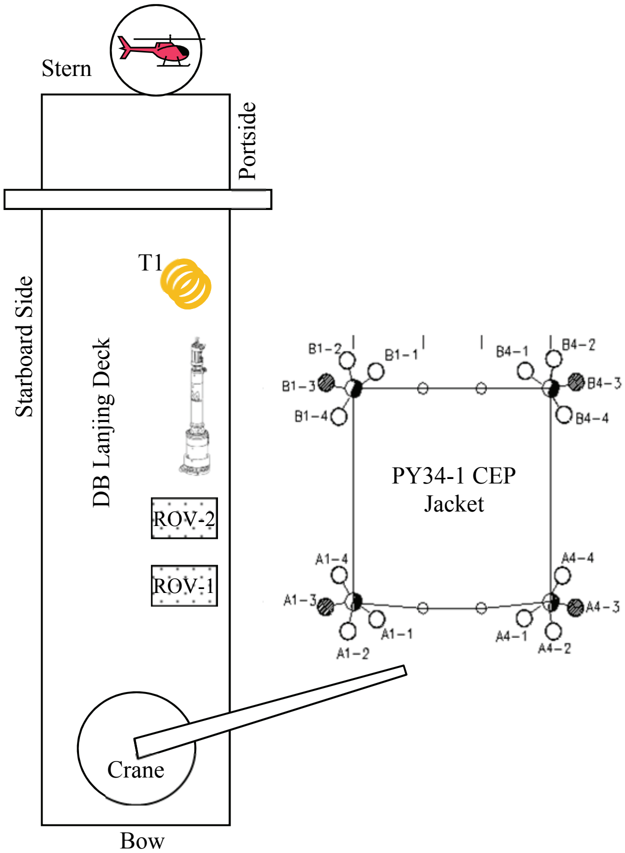


Fig. 1. Layout Sketch of DB, Jacket and Piles: T1 Is the Location Used to Unload the UW Cable to Water.

For this project, three piles (B1-4, A4-2 and A4-4) were selected for dynamic pile monitoring during initial driving and pile A4-4 was also tested during restrike driving after approximate waiting periods of 4 and 20 hours after the pile was first driven to the final penetration. The dynamic measurements of strain and acceleration were taken 5.8 m below the head of each pile. Two UW strain transducers and two UW piezoelectric accelerometers were bolted to diametrically opposite sides of the piles to monitor strain and acceleration. These strain and acceleration signals were conditioned and converted to forces and velocities by the PDA. For each hammer blow, the PDA calculates values for the maximum hammer energy transferred to the pile, the maximum compression stress at the gage location, and estimates the pile capacity by the Case Method.

After initially stabbed into mudline to about 41.0 m, pile B1-4 was driven with the hammer from 41.0 m to the design penetration of 122.8 m. No pile run was experienced during the driving of this pile. The recorded blow count at end of drive of pile B1-4 was approximately 31 blows per 0.5 m.

Piles A4-2 and A4-4 were initially stabbed into mudline and obtained a self penetration of approximately 27.0 m. Pile A4-2 was driven with the hammer from 27.0 m to the design penetration of 123.0 m. After about 28 blows of low energy at beginning of drive, the pile ran from 28.0 m to 42.0 m of penetration. Pile A4-2 was then driven to the final penetration of 123.0 m without experiencing further pile run. The recorded blow count at end of drive was 38 blows per 0.5 m. Pile A4-4 then driven with the hammer from approximately 27.0 m to 122.5 m. After about 20 blows of low energy at beginning of drive, the pile ran from 27.5 m to 42.0 m of penetration. Pile A4-4 was then driven to a penetration of 122.5 m without experiencing further pile run. The recorded blow count at the end of drive was reported to be 32 blows per 0.5 m. Pile A4-4 was also driven during restrike driving after waiting periods of 4 and 20 hours. The reported restrike driving blow counts were 123 blows per 0.5 m and 50 blows per 0.1 m of pile penetration, respectfully.

TEST RESULTS AND ANALYSIS

Measured Result and Case Method

During driving, the PDA interprets measured dynamic data to evaluate hammer and driving system performance, pile head compression stresses and structural integrity. The SRD is also computed according to the Case Method equations (Rausche et al., 1972; Goble et al., 1975). Each hammer blow recorded by the PDA is given a sequential blow number, which is used with the pile driving log to correlate PDA output with pile penetration. Table 2 summarizes the dynamic testing results for each pile tested at end of drive and beginning of restrike, i.e., at a penetration of between 122.5 and 123 m.

Table 2. Summary of Dynamic Monitoring Results

Pile	Reported blow count	Average Pile Head Compr. Stress (MPa)	Energy Transferred to Gage Location (kJ)	Energy Transfer Efficacy (%)	Case Method Capacity RMX J-0.5 (MN)
B1-4	18/0.3m	163	820	68	17.0
A4-2	38/0.5m	183	823	69	18.9
A4-4	32/0.5m	171	782	65	18.1
A4-4*	123/0.5m	187	980	82	50.8
A4-4**	50/0.1m	196	1111	93	61.6

* Beginning of restrike after a waiting period of approximate 4 hours
** Beginning of restrike after a waiting period of approximate 20 hours

Hammer and Driving System Performance

As shown in Table 2, the Menck MHU 1200S transferred energy to the PDA gage location near the pile head averaged over the final 0.50 m increment ranged from 782 to 823 kJ at end of drive for the three piles monitored. These transferred energies were obtained with a reported MHU 1200S read-out hammer energy at end of initial driving ranging from 764 to 916 kJ.

The performance of a hammer and driving system may be evaluated from a driving system's energy transfer efficiency. The energy transfer efficiency is defined as the energy transferred to the gage location divided by the manufacturer's rated hammer energy. The average transferred energy values during final driving of piles B1-4, A4-2 and A4-4 correspond to energy transfer efficiencies of 68, 69 and 65%, respectively, of the maximum rated energy of 1200 kJ.

During driving of the three piles tested, the Menck MHU 1200S was manually controlled to maintain acceptable stress levels in the piles and to prevent unnecessary pile runs or very low blow counts. A comparison between the reported Menck MHU 1200S read-out hammer energy and the transferred energy at final pile penetration indicates the hammer was transferring an average of approximately 90 percent of the desired energy to the pile head near the end of driving.

Driving Stresses and Pile Integrity

For each hammer blow, the maximum average compression stress at the gage location near the pile head was calculated by the PDA using the average signal from the two strain transducers. At the end of initial driving, the maximum average stresses measured near the pile head of piles B1-4, A4-2 and A4-4 were 163, 183 and 171 MPa, respectively. These values are well below the API recommended compression stress limit of 320 MPa, i.e. 90% of the steel's 355 MPa yield stress.

The PDA calculates the maximum compression stress at the gage location near the pile head. The maximum dynamic compression stress at other locations in the pile may be greater and can be evaluated by CAPWAP analysis. Based upon the CAPWAP analyses, compression driving stresses at other pile locations were up to 1.04 times the pile head compression driving stress. The maximum computed CAPWAP compression stress was 207 MPa and occurred during the restrike driving of pile A4-4. It should be noted that these reported stresses are the dynamic stresses averaged over the entire cross-sectional area of the pile and do not account for combined stresses such as static bending stresses and stress concentrations from uneven contact surfaces at the pile head or toe.

CAPWAP Analyses

As indicated in Table 2, during driving, the maximum Case Method equation RMX with a Case damping factor of 0.50 (RX5) was used to obtain estimates of the ultimate pile capacity. The mobilized capacities as calculated by the Case Method at the end of driving for piles B1-4, A4-2 and A4-4 were 17.0, 18.9 and 18.1 MN, respectively. The mobilized capacities as calculated by the Case Method during restrike driving of pile A4-4 were 50.8 and 61.6 MN for the 4 and 20 hour restrike, respectively.

CASE Method capacity estimation is only valid if the soil damping is known (J=0.5 was used for Table 2, which matched CAPWAP result) and the pile is uniform. If soil damping is not known and/or the pile is not uniform or more information such as soil resistance distribution is needed, CAPWAP analysis is required. CAPWAP is a widely used signal matching program to compute the soil resistance forces and their approximate distribution using the force and velocity data recorded in the field during dynamic testing. Final CAPWAP results include an evaluation of the relative soil resistance distribution, the soil quake and damping characteristics, and a simulated static load-set graph.

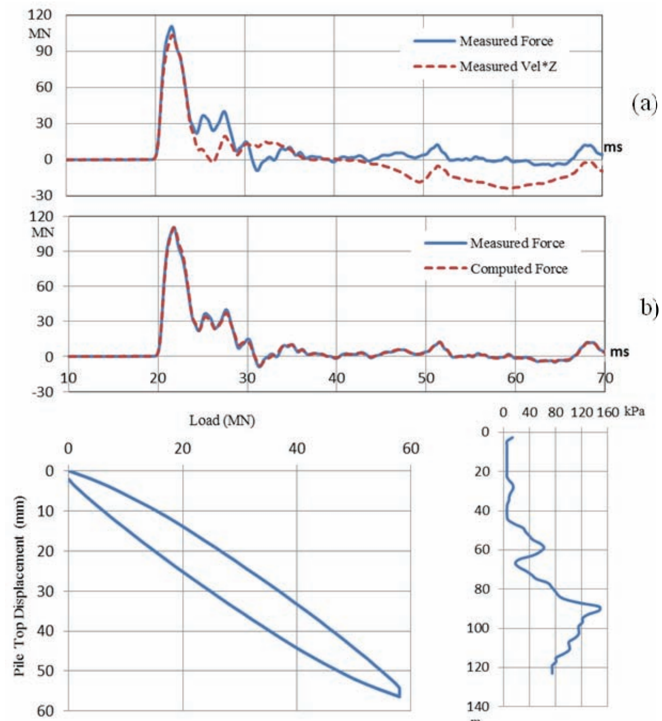
CAPWAP analyses were performed on records obtained near the final penetration depth for each of the three foundation piles, to obtain a more refined estimate of the mobilized capacity at end of initial driving and the beginning of restrike. Table 3 summarizes the CAPWAP results for each pile tested at end of initial drive and beginning of restrikes at the final pile penetrations. Fig. 2 shows the results for pile A4-4 only, at the beginning of restrike after about 20 hours waiting time, since the plots for other analyses are similar.

The computed mobilized capacities in compression at end of initial driving of piles B1-4, A4-2 and A4-4 were 17.5, 20.5 and 19.0 MN, respectively with about 6.0 to 6.5 MN in end bearing indicated for three piles. Long term capacity can be derived from restrike data after sufficient time has elapsed between the end of drive and the restrike test. Restrike testing of pile A4-4 indicated mobilized capacities of 40.0 and 58.0 MN after 4 and 20 hours of waiting time. However, the CAPWAP analysis for the 20 hour restrike indicated a reduction in end bearing from the end of initial driving and the 4 hour restrike testing. This reduction in end bearing resistance is likely due to the end bearing not being fully mobilized during the final restrike.

Table 3. Summary of CAPWAP Results

Pile	Ultimate Capacity (MN)			Smith Damping (s/m)		Soil Quake (mm)	
	Shaft	Toe	Total	Shaft	Toe	Shaft	Toe
B1-4	11.5	6.0	17.5	0.83	0.10	1.40	18.0
A4-2	14.0	6.5	20.5	1.29	0.34	2.50	15.5
A4-4	13.0	6.0	19.0	1.02	0.17	2.12	15.2
A4-4*	34.2	6.0	40.2	1.31	1.31	1.70	1.41
A4-4**	55.0	3.0	58.0	1.33	0.36	1.89	1.00

* Beginning of restrike after a waiting period of approximate 4 hours
** Beginning of restrike after a waiting period of approximate 20 hours



(c) (d)

Fig. 2. CAPWAP Result for A4-4 Restrike after 20 Hours Waiting: a) Measured Force and Velocity at Pile Top; b) Measured Force and Computed Force Based on the Match at Pile Top; c) Simulated Static Load-Set Curve; d) Unit Skin Friction vs. Depth Based on CAPWAP Analysis

Pile Long Term Capacity

As already mentioned, long term capacity can be derived from restrike tests. However, to accurately determine the long term capacity, the restrike tests should be performed such that the full soil resistance is mobilized, which requires sufficient impact energy be applied during the first few hammer blows if possible. Unfortunately, the Menck MHU 1200S used in this project was not powerful enough to fully mobilize a resistance of 95.7 MN and for hammer protection purposes full energy was not used by the hammer operator. For the 20 hour restrike, total 58 hammer impacts were applied and the corresponding capacity based on Case Method (RX5 = RMX for damping factor = 0.5) and transferred energy are shown in Fig. 3. After about 13 blows, the hammer was stopped to check the alignment between hammer and pile and restarted with lower energy. The impact energy was lower at beginning of the restrike test and reached a maximum between blow #7 and #13, thus blow #13 which appeared to mobilize more resistance than the other early blows was selected for CAPWAP analysis and the result was included in the Table 3 and shown in Fig. 2.

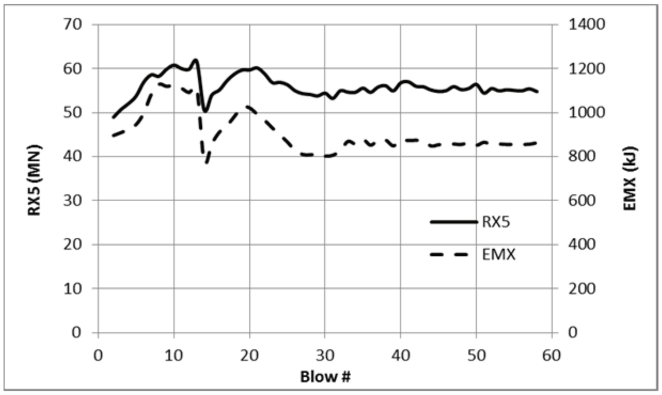


Fig. 3. CASE Capacity RMX for J = 0.5 and Transferred Energy from the Restrike Test after 20 Hours Waiting Period

Since a portion of setup was lost from blow #1 to blow #12, plus the resistance was not fully mobilized in blow #13, the result from CAPWAP analysis on blow #13 was considered a lower boundary solution. To more accurately estimate the resistance for the 20 hour restrike, additional CAPWAP analyses on blow #3 and #55 were performed to exam the change of mobilized resistance distribution. Fig. 4 shows the unit friction distribution of blows #3, #13 and #55 and Table 4 summarized the results. Comparing the unit friction distributions, the following can be observed:

1) For earlier blows such as blow #3, mobilized friction is higher above 90 m depth, see the friction distribution between depths 40 and 90 m;
2) Reduced friction resistance between 40 and 90 m for later blows such as blow #55 indicated a loss of setup;
3) Resistance below 100 m is lower for earlier blows, indicating a lack of energy to mobilize the resistance near pile toe. For later blows, due to loss of resistance (setup) for upper portion of the pile, more energy is transferred to the lower portion of the pile, which mobilized more resistance along the lower portion of the pile and the pile toe.

It is important to notice that these observations are indications that the hammer impact energy is insufficient to mobilize the full soil resistance, which could be caused by using a small hammer and/or lower energy settings.

Under this circumstance, the superposition method may help to obtain a more accurate estimation of the long term capacity, i.e., using the resistance computed based on earlier blows for upper portion of the pile and the resistance computed based on later blows for lower part of the pile. To do this, an envelope curve was created as shown in Fig. 5 based on the friction distributions for blows #3, #13 and #55. The more CAPWAP analyses performed, the more accurate this envelope becomes. However, even the last blow of the 20 hour restrike did not mobilize all resistance remaining in the pile/soil. Therefore, such a created envelope is still considered a lower boundary solution. Based on the unit friction distribution envelope curve, the total shaft resistance can be computed as 64 MN. Since the resistance at the toe is not fully mobilized during restrike test, the result from the end of initial driving, i.e., 6 MN, can be assumed if no relaxation is expected. Therefore the total estimated pile capacity at the 20 hour setup time would be 70 MN. The superposition result was also included in Table 4.

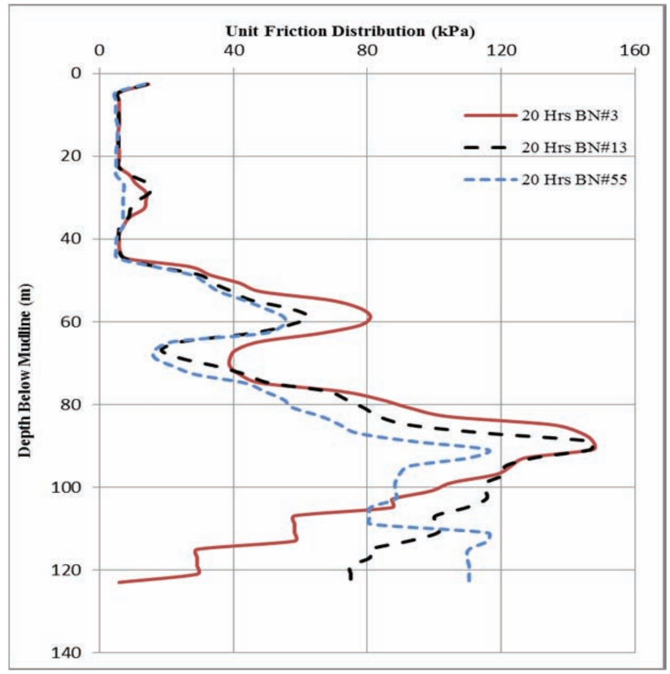


Fig. 4. Unit Friction Distributions from the CAPWAP Analyses on 3 Records Selected from the Restrike Test after 20 Hour Waiting

Table 4. Summary of CAPWAP Results for 20 Hour Restrike

Blow #	Ultimate Capacity (MN)			Smith Damping (s/m)		Soil Quake (mm)	
	Shaft	Toe	Total	Shaft	Toe	Shaft	Toe
3	51.8	3.0	54.8	1.37	1.31	1.84	1.00
13	55.0	3.0	58.0	1.33	0.36	1.89	1.00
55	48.9	3.6	52.6	1.09	0.12	1.80	2.07
	64.0*	6.0**	70.0				

* Based on the envelope curve
** Toe resistance was taken from end of initial drive

Shaft Soil Setup Factor

Based upon the restrike testing performed it is clear that significant soil setup will occur at this location. The CAPWAP analyses performed (Table 3 and Table 4) indicate that soil setup factors of 2.6 and 4.9 were achieved for total friction after waiting periods of 4 and 20 hours respectively. For easy comparison, the unit friction distributions from end of initial drive, beginning of restrike after 4 hrs waiting and the supposition solution of restrike of 20 hrs waiting were plotted together in Fig. 6.

A linear correlation between the soil resistance increase along the shaft and the log of the elapsed time ratio after the end of initial driving had been observed (Bullock et al., 2005). The following equation applies:

$$\frac{f}{f_0} = A \times \log\left(\frac{t}{t_0}\right) + 1 \qquad (1)$$

Where: A = a setup coefficient defining how fast the setup increase vs. elapsed time and ultimate setup factor relative to long term shaft resistance; t_0 = reference time, 0.1 hours is used in this study and the corresponding shaft resistance is the shaft resistance computed at end of initial driving, i.e., no setup occurred and the setup factor is 1.0; t = the elapsed time in hours for this study; f and f_0 = shaft resistance at time t and t_0.

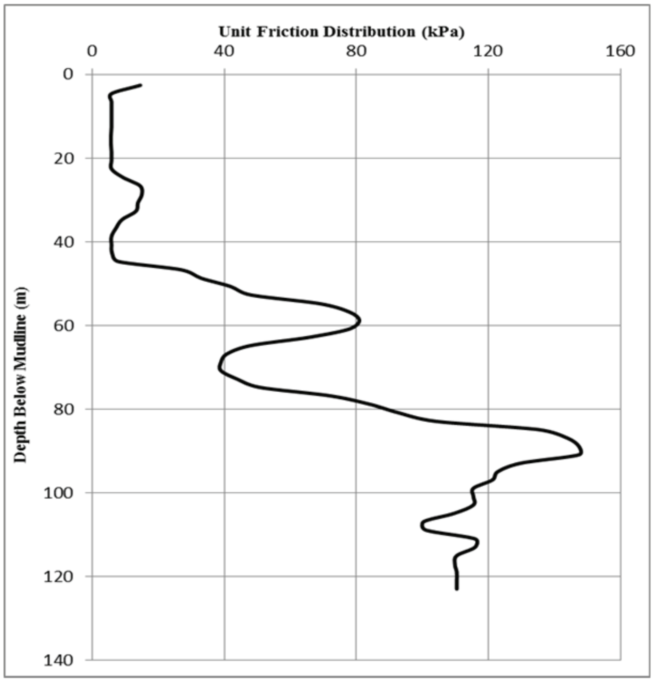


Fig. 5. Envelope for Restrike Test after 20 Hours

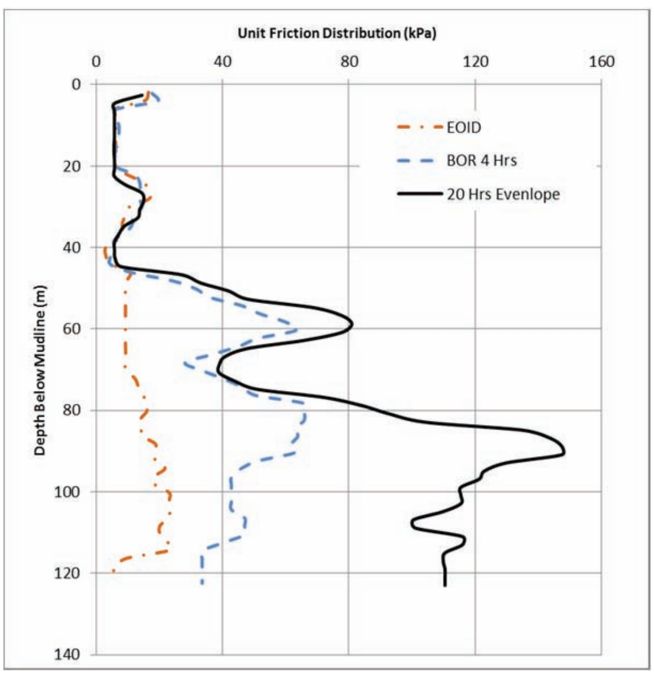


Fig. 6. Unit Friction Distributions from end of initial drive (EOID), beginning of restrike after 4 hrs waiting (BOR 4 Hrs) and the supposition solution of restrike of 20 hrs waiting (20 Hrs Envelope)

Since clays dominate this site, it makes sense to derive a setup factor for clay based on the friction distribution shown in Fig. 6. Considering 1) the bearing layer is below 40 m and 2) due to lack of energy, only the resistance above 90 m is probably fully activated, the stiff silty clay between 60 and 69 m was taken into study. The averaged unit skin friction for this layer based on Fig. 6 is 9.3, 39 and 54.3 kPa for EOID, BOR 4 Hrs and 20 Hrs envelope respectively. The corresponding setup factors are 4.2 and 5.8 for 4 Hrs and 20 Hrs restrikes, respectively.

The coefficient, A, can be determined based on measured data as shown in Fig. 7. It was found that A is 1.5 and 2 for total shaft resistance and stiff silty clay layer between 60 and 69 m, respectively. Compared to total shaft, the A value for stiff silty clay is higher, which can be explained as that the total shaft resistance includes setup effect of both clays and sands which usually have a lower setup factor, and the upper portion of the soil which did not show any setup.

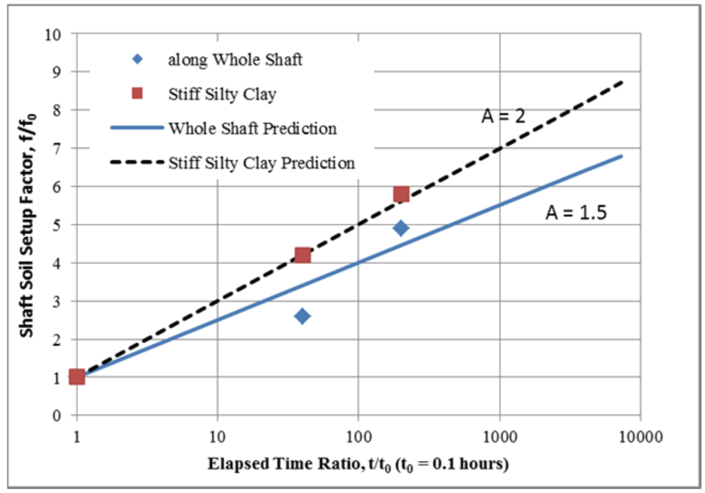


Fig. 7. Shaft Soil Setup Factors

Prediction of Long Term Soil Resistance

If no relaxation at toe is expected, it is reasonable to use the toe capacity (6 MN) determined based on the end of initial drive since it was fully mobilized. Thus to reach the designed ultimate capacity, 95.7 MN, the total ultimate shaft resistance needed is 89.7 MN, i.e., a shaft friction setup factor of 89.7/13 = 6.9. Based on equation (1), if A = 1.5 is used based on total shaft resistance setup feature, it will take more than 30 day (720 hours) to reach the required ultimate capacity. If the shaft setup feature follows the stiff silty clay (A = 2), it will take only 4 days (100 hours). This does not mean that clay regain setup faster. Since the sands and upper soil layer have less setup up, it needs clay to gain more to make up the total shaft resistance required, so it will take longer time.

DISCUSSION AND RECOMMENDATION

For this project the driving is very easy compared to prediction. With hammer energy being controlled between 60% and 80% of rated maximum energy, 1200 kJ for Menk MHU 1200S, the blow count at end of driving was between 20 to 40 blows / 0.5 m and total blows to drive the whole pile was mostly below 3000. This caused concerns that the required ultimate capacity may not be achieved. HSDT tests on three selected piles indicated that the soil resistance at end of driving was only between 17.5 and 20.5 MN with about 6.0 and 6.5 MN in end bearing.

To determine long term capacity, two restrike tests were performed on piles A4-4 after 4 hours and 20 hours time elapsed. Due to insufficient impact energy resulting from both controlling the impact energy and the hammer size, the pile capacity was not fully mobilized during restrike tests, especially for the restrike after 20 hours waiting time. To solve this problem, an envelope curve of friction distribution was created by superposing the skin friction resistances from the CAPWAP analyses on 3 records selected from different blows of the restrike test after a 20 hour waiting time. The analyses on pile A4-4 indicated mobilized capacities of 40.0 and 70.0 MN after 4 and 20 hours of waiting time with 34 and 64 MN along shaft and 6.0 MN at the pile toe, respectively.

When the impact energy is not enough to mobilize the full resistance and if multiple blows are applied, the earlier blows may mobilize more resistance in upper soil layers and later blows may mobilize more soil resistance in lower soil layers, due to loss of setup in upper soil layers. This is why the superposition of soil resistance from multiple CAPWAP analyses on different records was used in this study.

Based upon the restrike testing performed it is clear that significant soil setup will occur at this location. The CAPWAP analyses performed indicate that soil setup factors of 2.6 and 4.9 were achieved for total shaft resistance after waiting periods of 4 and 20 hours respectively.

Since CAPWAP analysis also provides the soil resistance distribution in additional to total capacity, it does not only allow the superposition method work, it also allows an estimate of the soil setup factor for a particular soil layer. The setup factors for stiff silty clay is 4.2 and 5.8 after 4 and 20 hours waiting time, respectively.

Considering the relatively short duration of the setup time provided for the restrike testing, it appears quite likely that the ultimate pile capacity will be achieved once sufficient setup time has occurred. To determine if and when the required ultimate capacity is reached, the relationship between setup and time is needed. A linear correlation between the soil resistance increase along the shaft and the log of the elapsed time ratio after the EOID had been observed (Bullock et al., 2005) as shown by equation (1) and was used in this study. The measured setup factors can be used to determine the relationship. Two results were obtained for this project: one is for total shaft resistance and another is for stiff silty clay. Based on the setup feature for whole shaft resistance, the designed ultimate capacity will probably achieved after about 30 days waiting time.

For a successful restrike test, it is important to mobilize whole soil resistance immediately which requires an adequate size hammer to provide enough impact energy during the restrike testing.

CONCLUSION

HSDT has been used to optimize design and for quality control/ assurance of driven piles both onshore and offshore. As the offshore industry grows rapidly, larger structures and deeper water pile driving are frequently encountered. The demand of HSDT for the underwater pile driving environment has been increased recently. It is much more challenging to perform underwater HSDT. This paper presents a successful case of the application of HSDT to monitor large diameter skirt piles of a large jacket installed in190.2 m deep water with restrike tests. It was shown that HSDT can be successfully applied to underwater pile driving to assess the ultimate capacity of these piles.

HSDT helped to monitor the stress to assure the integrity of the pile and estimate SRD using CASE Method or iCAP® in real time for each hammer blow. The CAPWAP analyses were performed on the selected records to correlate the correct CASE Method capacity value and obtain more information such as resistance distribution. Restrike tests performed after a period of waiting time help to estimate LTSR. For this project, two restrike tests were performed after 4 and 20 hour waiting periods and provided data for more extensive analyses to predict LTSR.

Although it is recommended that an adequately sized hammer be used to provide enough impact energy to fully mobilize the pile capacity at the beginning of restrike for a more accurate solution, such a hammer may not be available all times. It is shown that if insufficient impact energy is provided during restrike, the different portion resistance is activated by different blows, so superposition of resistance from different blows and toe resistance from end of initial driving or end of restrike if there is no toe relaxation is expected may be used. The analyses on the 20 hours strike indicated a mobilized capacity of 70 MN with 64 MN along the shaft and 6.0 MN at toe. The corresponding setup factor for whole shaft is about 4.9 which is a mixed setup factor of the whole shaft which consists of both silt, sand and clay and the clay is the dominated soil feature in this site. Based on the friction distributions obtained from CAPWAP analyses on the records collected at different times, it is possible to determine the setup feature of a particular soil layer. It was found that 20 hours setup factor for stiff silty clay is 5.8.

Based on the tests and analyses, two setup relationships for whole shaft resistance and stiff silty clay between 60 and 69 m were established respectively. If the relationship for whole shaft is used to predict, the required ultimate capacity of 95.7 MN can be achieved after about 30 days waiting time. Thus it is good idea to perform a restrike after as long as possible to better assess the long term pile capacity. To mobilize 95.7 MN, currently used Menk MHU 1200S is too small. Menk MHU 1900S which is also own by COOEC can be used and probably able to mobilize the required resistance.

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