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Application of High Strain Dynamic Testing Technique to Underwater Skirt Pile Foundation

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ABSTRACT

This paper presents the application of HSDT to dynamically monitor the underwater foundation pile installation of the Liwan 3-1 CEP Jacket in South China Sea. The eight-legged CEP Jacket, standing in a water depth of approximately 189.5m, is secured to the seafloor with 16×ø108"(2743mm)×158m foundation piles. It is very challenging to apply this technique to underwater piling, especially as deep as this project. The testing procedure is discussed in detail and summarized in this paper. The test results combined with CAPWAP® analysis are presented. A refined wave equation analysis was performed to help determine the capacities of other piles not tested.

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®
GRLWEAP	=	GRL Wave Equation Program
HSDT	=	High Strain Dynamic Test
SRD	=	Static soil Resistance to Driving
LTSR	=	Long Term Static soil Resistance

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, West 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 damage, hammer performance, drivability in various soil strata, as well as pile bearing capacity. High Strain Dynamic Testing technique (HSDT) has 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 dynamic 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 procedure, 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. The following procedure is now commonly used for pile acceptance using HSDT:

1) The axial force and velocity data are acquired, usually using strain and acceleration transducers attached to piles subjected to hammer impact, either during pile installation or after a wait period during restrike. For a uniform pile, stress wave propagation theory and the Case Method are used to calculate stresses along the pile, assess the shaft integrity, evaluate the hammer performance, and estimate the SRD using an assumed damping constant in real time.

2) The data is then further analyzed by the "signal matching" software such as CAPWAP using an elaborate pile-soil model to compute the total resistance and resistance distribution along the pile and at the pile toe more accurately. Compression and tension stresses at all points along the shaft are also determined more accurately. This step is necessary for non-uniform pile and/or complicated soil condition.

For driven piles, the dynamic testing and analysis are often used to optimize the design, as well for quality control and assurance. This may involve:

- Perform driveability analysis using wave equation analysis program prior to installation to select a suitable hammer and check stresses in the pile (Rausche et al., 2004);
- Perform HSDT during initial installation and/or restrike;
- Optionally perform refined wave equation analysis (Rausche et al., 2009) based on measured result to more accurately simulate the field conditions for the following purposes:
    - To predict the driving with slightly different configuration such as different pile length, hammer energy level, etc.;
    - To determine capacity based on recorded blow count and energy level for the piles not tested.

HSDT has also been used in offshore piling for several decades (Hussein et al., 1989) to monitor/accept the jacket foundation piles and wind turbine (Webster and Givet, 2010; Webster et al., 2008; Schallert and Klingmüller, 2012). So far, HSDT has been mainly used for above water driving condition. It is much more challenging to apply this method to underwater driving due to the factors such as water proof concern for transducers, cable and connection, difficulty in cable handling due to its weight and interference with other equipment, such as Remotely Operated Vehicle (ROV). Thus, there have not been many successful and well documented cases for underwater testing, especially as deep as this project. The experience gained from this project is definitely useful for future under deepwater application of HSDT.

PROJECT BACKGROUND

The Liwan 3-1 Gas field is located in the east part of South China Sea and the development consists of deepwater subsea developments and associated architecture, deepwater pipelines, shallow water production facilities (including the CEP platform, shallow water subsea pipelines) and onshore gas plant. The Central Processing Platform (CEP) located about 60 km away from the Liwan 3-1 deepwater gas field in the northwest is an 8-leg fixed steel jacket platform with production treatment facilities and living quarter and mainly designed to treat production well fluids that are transported from Liwan 3-1 subsea wellheads, and to receive mixed fluids consisting of dry gas & dewatering condensate. The mixed dry gas and dewatering condensate are processed and boosted at the CEP Platform, before sending into a shallow water subsea pipeline leading to the onshore terminal gas plant in Zhuhai, next to 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 31,139-ton CEP jacket. During the installation, there were two pile driving hammers available on the derrick barge: Menck MHU 1200S and 1900S are both double acting hydraulic hammer, with a ram weight of 648.4 kN and 1,011.8 kN, an equivalent stroke of 1.85 m and 1.87 m which results in a maximum rated energy of 1,200 kJ and 1,900 kJ respectively. 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 Liwan 3-1 CEP Jacket standing in a water depth of approximately 189.5m, is secured to the seafloor with 16 foundation piles through four vertical skirt sleeves at each corner leg. The piles installed vertically underwater (Fig. 1) consist of Ø2,743 mm O.D. steel pipe piles with wall thickness ranged from 50.0 mm to 100.0 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 100.0 mm wall thickness. The piles consist of one section 158.0 m long and were driven to the design penetration of 135.0 meters. Each pile weighs 754 tons and the design bearing capacity is around 130 MN.

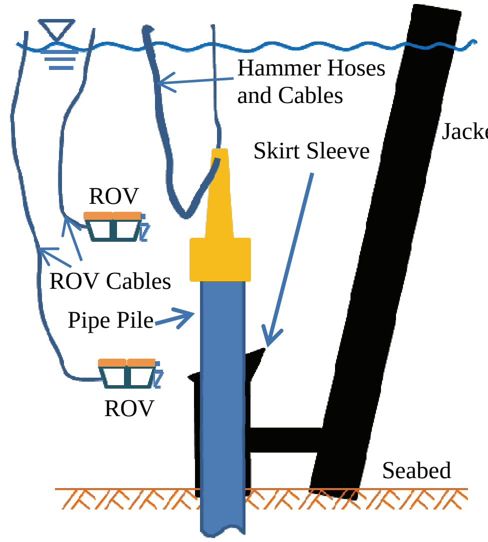

Fig. 1. Sketch to illustrate foundation details and underwater driving scheme

Soil Characterization

The 170 m soil boring performed for this site indicates that the predominant subsurface conditions consist mainly of soft to very stiff clay interlayered with medium dense to dense sand and sandy silt which is underlain by a dense to very dense sand layer. The details are listed in Table 1.

Table 1. Site Soil Condition

Depth (m)		Description
From	To	
0.0	3.0	Medium dense fine to coarse SAND
3.0	10.9	Soft to firm silty CLAY
10.9	13.2	Medium dense to dense silty fine SAND
13.2	21.0	Interlayered/laminated dense sandy SILT and stiff silty CLAY
21.0	24.6	Stiff silty CLAY
24.6	33.0	Dense silty fine SAND
33.0	53.3	Interlayered stiff silty CLAY and medium dense SILT, sandy SILT, silty fine SAND
53.3	59.4	Stiff silty CLAY
59.4	64.8	Medium dense sandy SILT/silty fine SAND
64.8	76.2	Very stiff silty CLAY
76.2	79.6	Medium dense sandy SILT
79.6	108.9	Very stiff silty CLAY
108.6	111.8	Medium dense sandy SILT
111.8	131.4	Very stiff silty CLAY
131.4	170.2	Dense to very dense fine to medium SAND

Based on this soil profile, static geotechnical analysis performed by COOEC according to API code indicated that the pile should reach a LTSR of 130 MN at a penetration of 135 m. During pile driving, the soil properties change and the pile encounter SRD, which is usually less than LTSR and the difference between LTSR and SRD is called soil setup. Prior to installation, the wave equation analyses were performed by COOEC to check the driveability and stress. To count for the uncertainty, following cases were analyzed using Menck MHU 1200S:

1) Assume soil setup factors (LTSR/SRD) of 3.3 and 1.0 for clay and sand respectively, and no plug at the pile toe and no loss at the toe during driving. This is the most optimum situation and the estimated blow count at end of drive is about 130 blows/m with a total SRD being about 62.0 MN.

2) Change soil setup factors to 2 for clay and keep rest assumptions the same as 1). The estimated blow count at the end is about 190 blows/m with a total SRD being about 77.0 MN.

3) Assume soil setup factors of 1.0 for both clay and sand and fully plugging at the toe and no loss at the toe during drive. This is the most difficult situation as an upper bound solution and the refusal is reached when the penetration reaches about 108 m. Switching to MHU 1900S would help to drive the pile to 131 m and the estimated SRD at the end of drive is about 114.0 MN.

Due to the heavy weight of pile and hammer, the estimated free penetration was approximately 18.0 m. Due to a possibility of refusal before 135 m penetration, HSDT was requested to check the capacity if refusal did happen.

DYNAMIC MONITORING

Equipment and Setup

The HSDT field equipment used in this project consists of following components (refer to Fig. 2):

1) The main unit (PDA – Pile Driving Analyzer) is used to collect, store and display data. Basic real time analyses such as CASE Method and/or iCAP can be performed to give a quick result on SRD, stresses, hammer performance, etc. The PAX model of PDA (Fig. 2a) was used in this project;

2) The main cable connects the PDA to transducer wires to transfer signals. If the test is performed above water, this cable may be avoided by using wireless transducers (Likinset et al., 2009; Schallert and Klingmüller, 2012). Unfortunately, wireless transducers are not available for UW (UnderWater) test and the heavy duty main UW cable (Fig. 2b) has to be used for this project;

3) Transducers consist of strain transducers and accelerometers. For this project, two UW strain transducers and two UW piezoelectric accelerometers (Fig. 2c) were bolted to diametrically opposite sides of the piles to monitor strain and acceleration, between 5.5 and 6.0 m below the head of piles to avoid stress concentration and hit by the sleeve. These strain and acceleration signals were conditioned and converted to forces and velocities by the PDA.

General Considerations for Offshore HSDT

The best location for PDA is where the penetration marker can be seen to help record blow count in PDA and the communication with hammer operation for controlling stress level purpose is easy. For this project, PDA was placed in the hammer control room since this room is equipped with ROV monitor screens.

To attach transducers to the pipe piles, holes for bolt should be drilled and tapping in the material yard to avoid delay and difficulty to access at site.

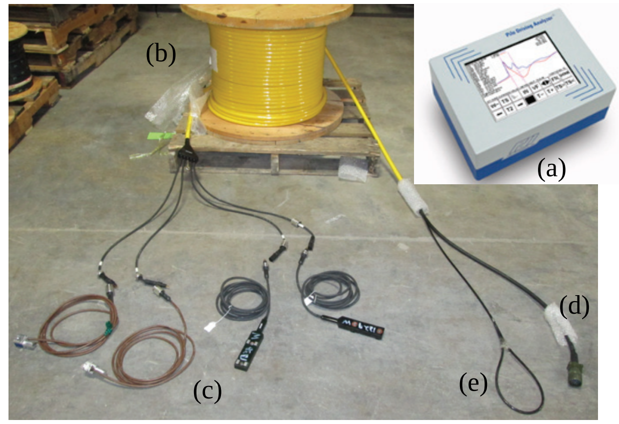

Fig. 2 HSDT field equipment: a) PDA; b) UW cable; c) UW transducers; d) Connecter to PDA; e) Steel wire for pulling cable

Considerations Special for Underwater Application

For underwater application, one of the challenges is to handle the heavy duty UW cable since it is heavy, invisible, possibility to tangle with other cables and hoses, even other equipment. For this project, the main UW cable was connected to transducer wires just before stabbing the test piles. After testing is finished, it is difficult and not economic to retrieve the transducers. However, the main UW cable can be retrieved and reused. To easy these operations, the connection between pile, transducers and cable is important.

Connection: Fig. 3 illustrates the recommended connection similar to what was used in this project. The internal steel wire rope of UW main cable is connected to the padeye on pile through the tether line (steel wire rope), shackle and quick link shown as the Path ACD in Fig. 3. This connection should be strong enough to sustain all pulling force between UW main cable and pile since the connection wires to transducers are not intended to take any force. The pulling forces are affected by the current, the weight of cable and/or unexpected hit by other equipment. The transducers are connected to the UW main cable through under water connecters at location B shown in Fig. 3 and the connection path is indicated by ABCDE. To avoid tangle, tie or tape transducer wires and tether line together. At location D, another quick link is optional to tie the transducer wire. Due care should be taken as follows:

1) The length of path ABC should be longer than AC to assure that no pulling force acting on the wires;

2) If an additional quick link is used to tie transducer wires at D, the transducer wire length should be slightly longer than the distance between D and E plus the length of the quick link;

3) If an additional quick link is not used at D and transducer wires are tied to the wire rope at C, the transducer wire length should be slightly longer than the length CDE;

4) Transducer wire should be given enough relaxation so when the cable swings to different directions, no pulling is applied to any

transducer wire. However, too much relax should be avoided to prevent from tangling to other equipment.

UW cable retrieval: Two common methods can be used to retrieve the UW main cable as follows:

1) ROV can be used to cut the connection at C (Fig. 3). This is preferred method and was used in this project.

2) Crane can be used to pull the steel wire rope inside the UW main cable to break the weak link at D (Fig. 3) first and then break the transducer wires beyond D since the wires were tied to the quick link at D. For this method to work properly, the quick link connected to the pad eye at D (Fig. 3) served as the weak link should be selected with a proper break force to keep UW main cable attached to pile for whole test and to be broken to protect main cable with a larger pulling force.

Note: It is important that the break point is beyond the UW transducer wire connecters (B in Fig. 3). If the connectors or wire above connectors are broken, the UW main cable cannot be used for further test.

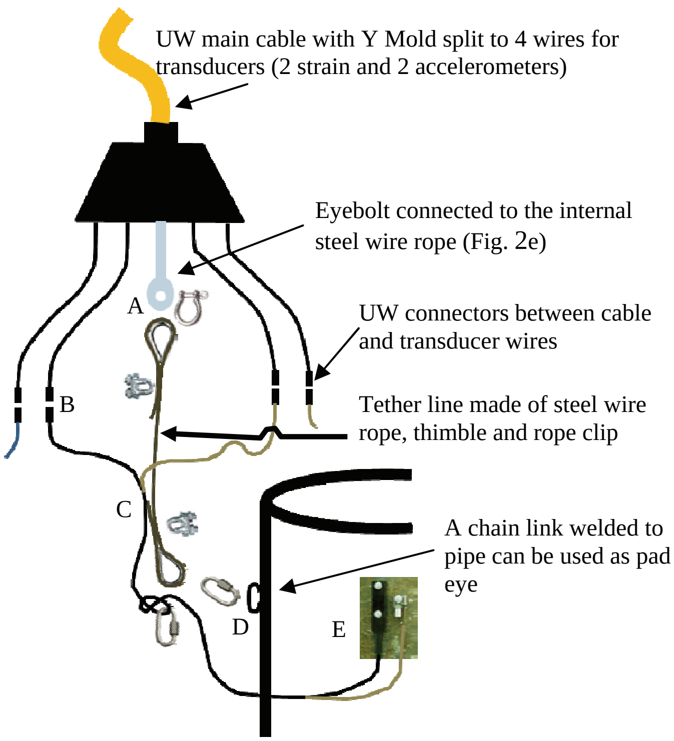

Fig. 3. Connection sketch between main cable, transducers and pile (Only two transducers: one strain and one accelerometer are shown in this sketch for simplicity. This sketch is only for illustration purpose.)

Marker UW cable: It is important to maintain a proper length of UW cable underwater such that its relaxation is enough to avoid break, as well as to reduce the possibility of tangle or hit by other equipment. Since the UW portion of the cable is invisible, it is necessary to marker the cable with length markers. How much length of cable should be left in the water depends on the distance between transducer location and the location on barge where the cable is kept, so water depth, pile penetration and barge location relative to the pile need to be considered. Potential barge movement such as pulling away from jacket due to high wind/waves should be considered, so communication is important.

UW cable handling: The UW main cable is heavy. Depending on the length, it could weigh close to 100 kg with reel. A winch is the best choice to release or roll back the cable. If no winch is available, stands should be built to support the cable reel which allows to roll cable out and back as shown in Fig. 4a. Before testing, the cable should be completely rolled out as shown in Fig. 4c since one end of the cable needs to connect to PDA. To help control cable release, the bollards can be used as shown in Fig. 4b.

Visibility: The UW main cable is bright yellow (or orange) which help ROV to see it. However, between A, D and E in Fig. 3, the transducer wires are in dark color. They were painted with yellow color to help to see through ROV in this project, which make the cut at end of test and tangle avoidance easy.

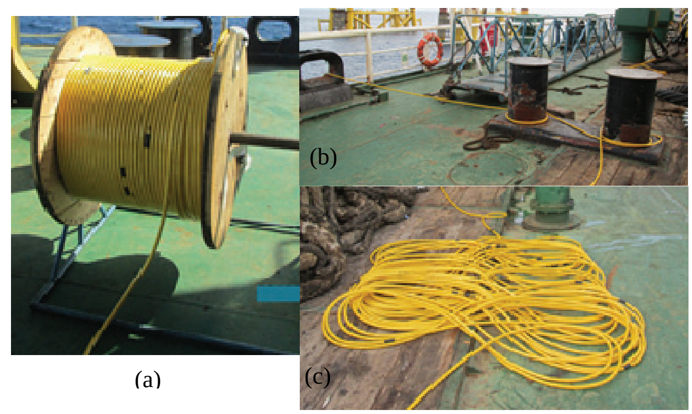

Fig. 4. UW main cable handling: a) Cable reel on a stand; b) Bollard used to prevent cable falling; c) Cable layout to prevent twist since one end need to connect o PDA

Location for cable release to water: It is important to help reduce possibility of tangling. Fig. 5 shows the deck layout sketch of DB Lanjing including the relative locations of major equipment (ROVs and hammers) which were operated underwater. In this sketch, the barge is assumed in the location to drive piles along Row 1 of the jacket, i.e., the piles for legs A1 and B1. Two spots labeled as T1 and T2 are recommended locations to roll out the UW main cable to water. When testing piles at leg B1, T1 is preferred location while keep ROVs and hammer on one side. If the piles at leg A1 are tested, T2 is preferred location. For this project, the two piles at B1 leg (B1-2 and B1-3) were dynamically monitored, so location T1 was used.

Procedure

Since the soil condition for each leg is similar, two piles at leg B1 (B1-2 and B1-3) were selected to test.

Following procedure was used in this project and comments were added to make it easy to change to a general guideline for other project:

1) Transducer attachment just before picking up test piles for stabbing (on material barge)
   i) If this is not done at the material yard, drill and tap pile for transducer attachment at desired location. For this project, 5.5 m to 6 m from pile top considering: a) 2D below top; b) below hammer pile sleeve; c) close to pile top to avoid transducers going under skirt sleeve.
   ii) Weld a 40 to 50 mm pad eye (or chain link from Come-

Along) on the pile at about same level as the transducer location at 90 degree from the transducer location if two sets of transducers are mounted at opposite side of the pile. Paint a thick circle around the chain link with bright color paint to make it visible to the ROVs.

iii) Attach transducers on the pile with connection method as shown in Fig. 3, but leave connectors at B and connection A open for later connected to the main cable.

iv) Paint tether line with bright color paint to be visible underwater and help cut by ROV for this project;

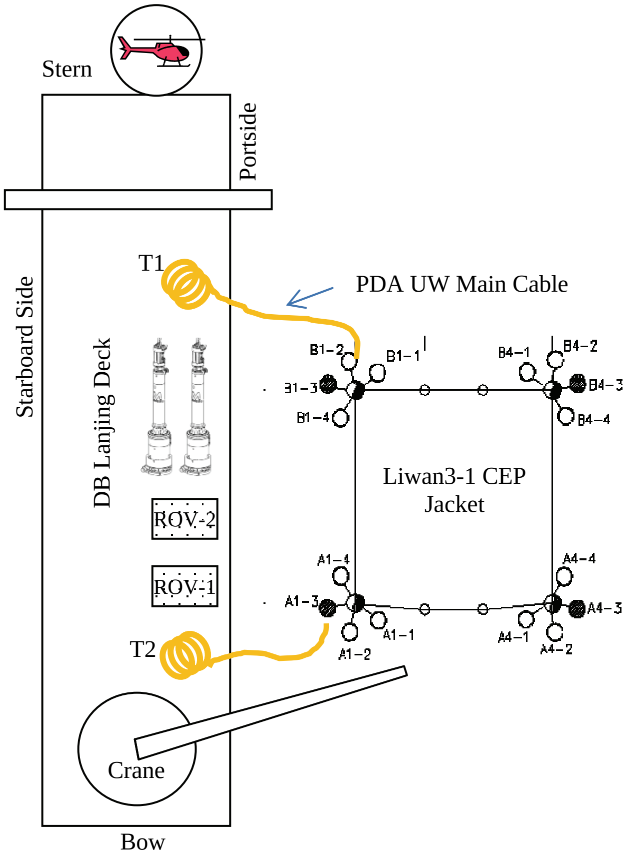

Fig. 5. Layout sketch of DB, jacket and piles: T1 and T2 are suggested locations to unload the UW cable to water.

2) Connect the UW main cable before stabbing while pile in the air
   i) After the pile is picked up from material barge and swung to the portside of DB, lower pile down such that the transducer connectors should be above water and it is safe to access.
   ii) Follow safety rules to approach the pile. For this project, work basket was used to lift personals to approach the pile to make the connection.
   iii) Connect the UW main monitoring cable to the tether line, i.e., connect at A as shown in Fig. 3 first to support the weight and pulling from the main cable on the pile;
   iv) Connect the transducer wires to the main cable at B as shown in Fig. 3.
   v) Connect another end of the UW main cable to PDA to check status of the transducers. Deck PDA cables may be needed to extend the UW cable to where PDA was setup, such as hammer control room for this project.
3) UW cable handling during stabbing the pile for monitoring
   - Lower the pile into the jacket skirt sleeve by ensuring monitoring equipment is not damaged during this operation
   - Release the UW cable to give adequate length of cable in the water;
   - ROV to check the pile and the underwater monitoring cable during the descent of the pile;
   - ROVs to check the self-penetration of the pile.

4) Pickup and stab hammer: MHU 1200S was used to drive piles for this project
   - ROVs to ensure that the underwater monitoring cable stays clear of the hammer and the jacket;
   - Test engineer checks signal on PDA to make sure transducers working properly.

5) Monitoring during pile driving
   - Two test engineers: one monitored PDA and pile penetrations and another dedicated to handling cable. A good communication between them is important;
   - Pile penetration information should be passed to the engineer who is handling the UW cable for an adequate length of cable released to water while pile advances. Especially at beginning, possible pile run needs extra cable in the water. For this project, the piles may run from about 13.0 m to about 72.0 m;
   - Monitor and ensure that stresses remain within specifications.

6) After pile driving and testing are completed
   i) Save PDA data.
   ii) Lift the hammer from the pile once driving of pile completed and ROVs to ensure that the hammer stays clear of the UW monitoring cable during hammer ascent.
   iii) If restrike is required and before restrike the barge needs to move, bring the UW main cable upper end to the jacket and secure to the jacket for later use.
   iv) If no restrike is required, separate the main cable from the transducers using one of the two methods mentioned before.
   v) Pull the cable and rewind back to the real. Due to the weight, it probably takes 4 persons to pull if no winch is available.

TEST RESULTS AND ANALYSIS

Both test piles, B1-3 and B1-2 at leg B1, were driven with the Menck MHU 1200S. B1-3 was installed first. After initially stabbed into mudline to about 12.0 m and the hammer was used to drive from 12.0 m to the design penetration of 135.0 m. After about 22 blows of low energy at the beginning of drive, the pile ran from 13.5 m to 72.0 m of penetration. After re-stabbing the hammer on the pile top, the pile B1-3 was driven to the final penetration of 135.0 m without experiencing further pile run. The recorded blow count at end of drive of the pile B1-3 was 143 blows per 0.5 m.

The pile B1-2 was initially stabbed to about 11.5 m and driven to the design penetration of 135.0 m. After about 36 blows of low energy at beginning of drive, the pile ran from 13.0 m to 71.5 m of penetration. After re-stabbing the hammer on the pile top, the pile B1-2 was driven to the final penetration of 135.0 m without experiencing further pile run. The recorded blow count at end of drive of the pile B1-2 was 125 blows per 0.5 m.

Measured Result and Case Method

During test, 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 depth. Table 2 summarizes the dynamic testing results for each pile tested at end of drive, i.e., at a penetration of 135 m.

Table 2.Summary of Dynamic Monitoring Results

Pile	Reported blow count (blows/0.5 m)	Average Pile Head Compr. Stress (MPa)	Energy Transferred to Gage Location (kJ)	Energy Transfer Efficacy (%)	Case Method Capacity RMX J-0.5 (MN)
B1-2	125	162	769	64	46
B1-3	143	172	759	63	48

CAPWAP Analyses

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. The CAPWAP is 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 the 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. Table 3 summarizes the CAPWAP results for each pile tested at the final pile penetrations and Fig. 6 only shows the results for the pile B1-3 since the results from the pile B1-2 are similar.

Table 3.Summary of CAPWAP Results

Pile	Ultimate Capacity (MN)			Smith Damping (s/m)		Soil Quake (mm)	
	Shaft	Toe	Total	Shaft	Toe	Shaft	Toe
B1-2	30	19	49	0.12	0.88	1.4	4.8
B1-3	30	19	49	0.40	0.79	1.3	4.2

Hammer and Driving System Performance

The Menck MHU 1200S transferred energy to the PDA gage location near the pile head averaged over the final 0.50 m increment was 769 kJ at end of drive of the pile B1-2, and 759 kJ at end of drive of the pile B1-3 as shown in Table 2. These transferred energies were obtained with a reported MHU 1200S read-out hammer energy at end of initial driving of 989 kJ for the pile B1-2 and 961 kJ for the pile B1-3. If the maximum rated energy of 1200 kJ is used to compute the energy transferred efficiency, the efficiency values during the final driving of the piles B1-2 and B1-3 were 64% and 63% respectively.

During driving of the two piles tested, the Menck MHU 1200S was manually controlled to maintain acceptable stress levels in the piles and to prevent unnecessary pile runs. A comparison between the reported Menck MHU 1200S read-out hammer energy and the transferred energy at the final pile penetration indicates the hammer was transferring an average of approximately 80 percent of the desired energy to the pile head near the end of driving of the two piles tested.

Driving Stresses and Pile Integrity

For each hammer blow, the maximum average compression stress at the transducer location near the pile head was calculated by the PDA using the average signal from the two strain transducers. During monitoring of the piles, the maximum average stresses measured near the pile head of the piles B1-2 and B1-3 were 162 MPa and 175 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 maximum dynamic compression stress at other locations in the pile may be greater and can be evaluated by the CAPWAP analysis. Based upon the CAPWAP analyses, compression driving stresses at other pile locations were up to 1.16 times the pile head compression driving stress. The maximum computed CAPWAP compression stress was 190 MPa and occurred near end of drive of the pile B1-3. It should be noted that these reported stresses are only 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 toe.

During data acquisition, force and velocity records were evaluated for indications of pile damage below the gage location. Observable signs of significant pile damage were not indicated in this assessment of the dynamic records.

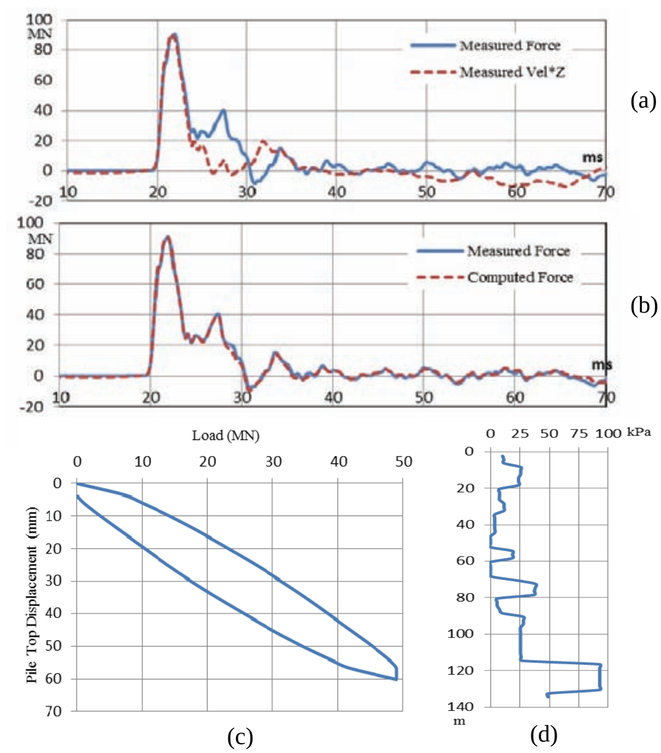

Fig. 6 CAPWAP result for B1-3: a) Measured force and velocity at pile top; b) Measured force and computed force based on the match at pile top; c) Simulated static load-set curve; d) Unit skin friction vs. depth based on CAPWAP analysis

DISCUSSION AND RECOMMENDATION

Pile Capacity

During initial driving, the maximum Case Method equation RMX with a Case damping factor of 0.50 (RX5) was used to obtain estimates of the SRD. The mobilized SRD as calculated by the Case Method at the end of driving for the piles B1-2 and B1-3 were 46.1MN and 47.9 MN (Table 2). The CAPWAP analyses (Table 3 and Fig. 6) on records obtained near the final penetration depth for each of the two foundation piles to obtain a more refined estimate of the mobilized SRD of the piles at end of initial driving. These analyses also provide an approximation of the shaft friction and the end bearing components of the mobilized SRD at end of drive. The computed mobilized capacities in compression at end of initial driving of the piles B1-2 and B1-3 were 48.9 MN and 49.0 MN, respectively, with about 30.0 MN in shaft friction for both piles. LTSR can be derived from restrike data after sufficient time has elapsed between the end of drive and the restrike test. Since no restrike was carried out on this project, no LTSR could be determined for the two piles tested. However, from the CAPWAP analysis, a shaft setup factor of 3.0 would be necessary to achieve the required maximum ultimate capacity of 109 MN for the B1 corner of the Liwan3-1CEP Jacket. Based upon authors' experience shaft setup values between 2 to 4 (Webster et al., 2008; Webster and Givet, 2010) would be expected for the soil types indicated at the Liwan 3-1 CEP Platform.

The static pile capacity from dynamic method calculations is an estimate of the mobilized, axial compression pile capacity during driving. Increases and decreases in the pile capacity with time typically occur (soil setup/relaxation). Hence, dynamic method estimates of pile capacity from initial driving tests only may under-estimate or over-estimate LTSR. Therefore, dynamic restrike tests after an appropriate waiting period are usually a better indication of long-term pile capacity.

Recommended Testing Sequence

For future planning purpose, following is the recommended testing sequence to include restrike without moving the derrick barge to other side of the jacket. In this sequence, it is assumed: 1) Pile A1-2 is tested both at end of initial drive and restrike; 2) Piles B1-2 and B1-3 are only tested at end of initial drive; 3) Driving is started after stabbing two piles to reduce hammer lift time:

1) Stab Piles A1-2 and B1-2 with transducers and cable attached
2) Drive Pile A1-2 with monitoring
3) Drive Pile B1-2 with monitoring
4) When driving both piles are completed, cut and retrieve the UW cable attached to Pile B1-2; the cable will be used for Pile B1-3
5) Stab Pile B1-4
6) Stab Pile B1-3 with transducers and cable
7) Drive Pile B1-4 without monitoring
8) Drive Pile B1-3 with monitoring
9) Cut/retrieve the cable attached to Pile B1-3
10) Restrike Pile A1-2 with monitoring
11) Cut/retrieve the cable attached to Pile A1-2

With this testing sequence, before restrike Pile A1-2, operations including driving Pile B1-2, picking up, stabbing and driving Piles B1-4 and B1-3 are performed, which would probably take more than 72 hours. There is no extra hammer lift. The only extra work is to take care of the cable attached to Pile A1-2. The sequence can be easily modified to test other piles.

REFINED WAVE EQUATION ANALYISIS

Based on the PDA measurement and CAPWAP analysis, a refined wave equation analysis can be performed (Rausche el al., 2009) to help assess the pile capacity for other piles not tested. In this paper, GRLWEAP is used for this analysis. The soil resistance distribution and other soil dynamic parameters from CAPWAP were used to setup initial soil mode for GRLWEAP.

To setup the hammer model for GRLWEAP, special care is required for underwater application since submerged weight should be considered. GRLWEAP 2010 hammer database includes Menck MHU 1200S and the assembly weight. However, the assembly weight is only for the standard configuration which may not include the pile sleeve weight since the size of pile sleeve depends on the size of pile. Based on the data sheet of MHU 1200S underwater configuration for this project, the anvil weight is 305 kN and the submerged weight can be estimated as 266 kN if it is assumed as solid steel. The submerged weight of total hammer including pile sleeve is listed as 1,635 kN. Subtracting ram weight (648.4 kN) and anvil submerged weight (266kN), the assembly submerged weight is about 720 kN. GRLWEAP 2010 allows change the gravities for hammer and pile to consider the submerged weight in static balance computation. For solid steel, the effective gravity to consider submerge is 8.56 m/s^2. Since GRLWEAP calculates masses of helmet and assembly based on in-air weight for dynamic analysis, so the equivalent in-air weight of assembly, 824 kN (=720*9.8 /8.56), and the helmet weight of 305 kN were used. Since both pile and hammer were completely submerged, to consider buoyancy effect on the static equilibrium, an effective gravity of 8.56 m/s^2 was used for both pile and hammer. The other input for hammer such as hammer efficiency and driving system stiffness are shown in Table 4. Fig. 7 shows a schematic representation of the wave equation model analyzed. To match stress and energy transferred, adjustment of the driving system parameters (stiffness to 32,000 kN/mm and coefficient of restitution to 0.63) is required, which may be explained by helmet bending, ram/helmet interface behavior and water in the pile for underwater driving. To match the blow count, the damping factors need adjustment (Table 4).

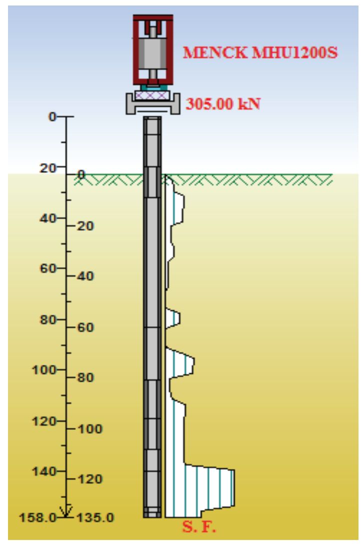

Fig. 7. Wave Equation Model

Table 4. Measurement and CAPWAP result together with corresponding GRLWEAP input/output values

Quantity	Default/Measured/ Computed	GRLWEAP
Capacity Total/Toe (MN)	49/19	49/19
Damping Shaft/Toe(s/m)	0.4/0.79	**0.61/0.8**
Quake Shaft/Toe (mm)	1.3/4.2	1.3/4.2
Hammer Energy (kJ)	961 (rated 1200)	961
Equivalent Stroke	1.48	1.48
Hammer Efficiency	0.95	0.95
Dr. System Stiffness (kN/mm)	N/A*	**32,000**
Dr. System Coeff. of Resti.	N/A	**0.65**
Pile Top Stress (MPa)	172	172
Transferred Energy (kJ)	759	759
Blow Count (Blows/0.5 m)	143	**142**
*N/A: No input-rigid assumption		

After a satisfied match is reached, the resulting pile/soil/hammer model for GRLWEAP can be used to assess the capacities for other piles which are probably driven with different energy levels. Since the same hammer is used to drive other piles in this project, an inspector's chart analysis is performed to establish the relationship between blow count and hammer energy settings or equivalent strokes for a given total capacity (49 MN is used in this analysis) as shown in Fig. 8.

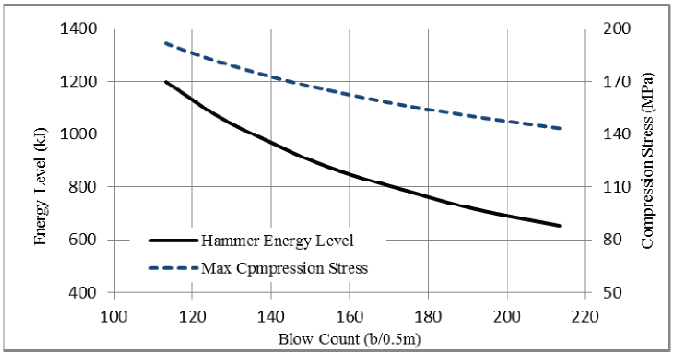

Fig. 8. Inspector's Chart: Capacity = 49 MN

Take Pile B1-4 for example, for the final 0.5 m penetration, the recorded blow count is 147 with hammer energy setting being 971 kJ. The corresponding location in Fig. 8 is above the solid curve, so the soil resistance should be large than 49 MN. For Pile A1-3, it took 190 blows with 774 kJ to drive last 0.5 m, which also made above the solid curve.

CONCLUSION

HSDT has been used to optimize design and for quality control/ assurance of driven piles both onshore and offshore. As offshore industry grows rapidly, larger structure and deepwater pile driving are frequently encountered. The demand of HSDT for underwater 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 the large skirt piles of a mega large jacket under about 189.5 m deep water and document the procedure and experience in details for future reference. It was proved that HSDT can be successfully applied to underwater pile driving. If planned and executed correctly, HSDT will not interfere with normal operation much and help quality control and assurance on the pile installation and acceptance. The procedure presented in this paper can be easily modified to help to specify HSDT for future projects. The main factors to perform a successful underwater HSDT are:

- Good communications are important to understand the operation sequence, potential barge movement and what to expect from all parts involved in the test;
- The connection and attachment between transducers, pile and UW main cable should be well designed to reduce connection time and help cable retrieval;
- Feasible locations to release cable to water should be carefully selected to avoid potential tangling and help easy handling;
- A dedicated and skilled person should be assigned to take care of the cable.

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(probably at end of initial drive) 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, the designer felt confidence on the LTSR if the piles reach the required minimum tip elevation, i.e., a penetration of 135 m, so no restrike test was requested. However, a recommended testing sequence is proposed in this paper to perform the restrike test with minimum interruption on the installation operations for future planning purpose.

Usually, not all of the piles need to be tested. For tested piles, the SRD can be determined using the CASE Method and/or CAPWAP. However, if a different hammer and/or a different hammer energy level are used to drive other piles, the blow count from the tested piles cannot be used to indicate the capacities for other piles. A refined wave equation analysis is performed using GRLWEAP to demonstrate how to create in inspector's chart for the piles not tested by HSDT.

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