DYNAMIC PILE MONITORING FOR OFFSHORE PILE ACCEPTANCE

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Dynamic pile Monitoring Services (DMS) have been used for testing of offshore piles for several decades. However, most of this testing was to measure hammer performance and pile stresses rather than pile capacity or soil resistance. As the need for more economical solutions for foundation support increases, the use of DMS has provided a better means of evaluating in-place pile capacity for offshore structures, and therefore a method of determining pile acceptance. The authors have been involved in several offshore projects where DMS has been the primary source of data for acceptance of the driven pile foundations. The use of DMS for evaluation of offshore foundations requires considerations quite different from land applications and these considerations are often necessary for the careful and thorough analysis of the collected data in nearly real time for determination of pile acceptance. Case Method capacity estimates are often of little use due to the non-uniform cross sections of the foundation piles and their deep penetrations. Therefore, capacity assessments should be made using CAPWAP which is a signal matching program whose utility and reliability has been proven on thousands of projects every year, both on land and offshore. Depending upon the soil conditions at a given location, capacity assessment after soil setup effects may be necessary. In addition, tension capacity is often critical when pile refusal occurs significantly before the design pile penetration depth. Evaluation of uplift capacity can be accomplished by a CAPWAP analysis, which results in a total capacity prediction and its shaft resistance component. The use of hydraulic hammers for driving of these piles also adds the requirement of controlling hammer energy such that over-stressing of the pile is avoided. The calculation of non-axial stresses such as bending or eccentric stresses may also be involved in order to prevent pile damage.

Introduction

Dynamic pile testing using the Pile Driving Analyzer and analysis has been used for evaluation of driven piles for nearly 40 years. The testing methods and results have been well accepted worldwide for both public works and private industry projects. The use of dynamic pile testing for offshore projects, mainly in the oil and natural gas industry, has not been so widely accepted. Much of the early testing performed for these structures was primarily done to evaluate hammer performance and pile stresses, with results concerning static pile capacity of a secondary nature. In the past 10 to 15 years this practice has steadily changed as the industry has begun to take advantage of the potential for savings on foundation costs as well as the ability to evaluate driving conditions where difficult driving may be encountered.

Preconstruction Drivability Studies

Dynamic Monitoring Services (DMS) are typically provided for offshore structures where pile driving conditions are not well known or where difficult driving conditions may be expected. Normally, these services begin with a drivability analysis designed to:

- assess the anticipated driving conditions and to determine what, if any, special precautions may be necessary during pile installations
- determine if the proposed hammer(s) are suitable for the pile installations
- predict the combined dynamic and static bending stresses
- provide recommendations concerning the predicted dynamic compressive stresses.

The drivability analysis needs to be performed by the DMS engineer during the design phase of the project so that changes to the pile makeup or driving system(s) may be provided if necessary. This allows for a complete evaluation of the actual pile makeup that will be installed and the specific hammer(s) proposed. Recommendations are provided for hammer energy settings or limitations for pile stick up during driving, as well as the potential for pile refusal prior to the final design pile toe elevation.

Field Testing and Analysis

DMS testing should be provided for pile installations if there is a potential for cost savings or where difficult driving conditions exist. Typically, the DMS engineer can install test gages without delay to the project while the pile sections are located on the "materials" barge. This is easily accomplished when an internal gripping tool is used to hoist the pile sections into place, reducing the risk of gage damage during pile handling and placement. Testing may then be provided over the entire driven pile length, or for those sections near the end of pile installation. While the location and value of the maximum static bending stresses along the pile are not measured through DMS, they can be calculated for a given pile stickup. The expected dynamic stresses at these locations should be estimated by the pile top DMS measurements. The DMS measurements should then be used to determine proper hammer energy settings in order to avoid over-stressing of the piles by either dynamic compression stresses or the combination of dynamic compression and static bending stresses. Finally, evaluation of the static pile capacity should be provided. Usually, the Case Method equations normally used for pile capacity estimates during testing are of little use for typical offshore piles because of the non-uniform cross sections of these piles as well as the relatively deep penetrations required. Therefore, CAPWAP analyses must be performed in order to estimate static pile capacities. As with all dynamic testing situations, it is usually important that restrike testing be performed at or near the final pile penetration. For offshore structures, this involves testing after waiting periods of between 12 and 48 hours due to construction constraints. From CAPWPAP analyses carried out at end of drive and beginning of restrike of the piles, it is often possible to estimate the additional pile capacity which will be obtained with additional setup time.

In order to illustrate the effectiveness of DMS testing for offshore installations, details from five recent platform installations where DMS testing was provided are summarized below. All five installations were located in the Arabian Gulf and all of them consisted of testing between four and six piles at each platform location. All of the piles tested were 1067 mm outside diameter, open ended pipe piles driven with a Menck MHU 500T hydraulic hammer.

Site A

At this platform location a total of four 1067 mm OD (Outside Diameter) x 38 mm wt (wall thickness), piles were proposed to be driven. The piles had a 45 mm wt x 3 m long driving shoe at the pile bottom. The design pile penetration for the four piles ranged from 37 to 42.5 meters depending on the pile location. The piles were proposed to be driven in three sections with the first section having a length of approximately 67 m and the remaining sections having lengths of approximately 19 and 17 meters. The water depth at this location was approximately 47 m. The general soil profile, summarized in Table 1, consisted of primarily very stiff to hard carbonate or calcareous clays.

Table 1 - Site A soil conditions

From (m)	To (m)	Description
0	11	Very stiff to hard calcareous clay
11	17	Very Dense calcareous silty fine sand
17	47	Hard carbonate/calcareous clay w/ sand seams & layers
47	57	Dense calcareous sandy silt
57	60	Hard calcareous clay
60	64	Very dense calcareous silty sand
64	75	Hard calcareous clay

Based upon the platform design, the required pile capacities are summarized in Table 2.

Table 2 - Site A pile capacities requirements

Platform Leg	Applied Ultimate Compression Load, MN	Applied Ultimate Tension Load, MN
A1	17.9	4.9
A2	19.3	8.2
B1	19.7	6.8
B2	16.3	4.9

Based upon the subsurface profile and experience from previous platform installations, it was expected that the piles would drive easily to the planned pile penetration. Ultimate pile capacity estimates would need to be provided by performing restrike testing on one or more of the platform piles. As such, DMS testing was provided on the final two add-on sections for each pile. The primary purpose of this testing was to provide recommendations concerning hammer energy settings to prevent over-stressing of the piles as well as to allow for setup calculations to predict pile capacities.

Based upon the preconstruction drivability analysis, it was expected that the Menck MHU 500T hammer energy would need to be reduced during restart driving for the add-on pile sections. The drivability analysis estimated that a restart hammer energy setting of 65% would result in a combined dynamic compression and static bending stress of 331 MPa, with a static bending stress of approximately 80 MPa. Figure 1 displays a plot of the expected dynamic and combined dynamic and static bending stresses. During pile installation the DMS testing indicated that the maximum pile top stress was 224 MPa at the restart of driving for the final add-on section for pile B2. A CAPWAP analysis indicated that the maximum dynamic stress along the pile shaft was 256 MPa. The CAPWAP also indicated that the 256 MPa stress occurred at the top of the stabbing guide of the 2nd welded add-on section of the pile (P3 section) resulting in a localized stress maxima. Since this localized stress increase occurred at the top of the P3 stabbing guide, just above the splice location, the combined dynamic and static bending stress was approximately 336 MPa (256 MPa dynamic plus 80 MPa static bending stress). However, this stress occurred when the hammer was being operated at a hammer energy setting of approximately 80%, not at the 65% anticipated from the drivability analysis.

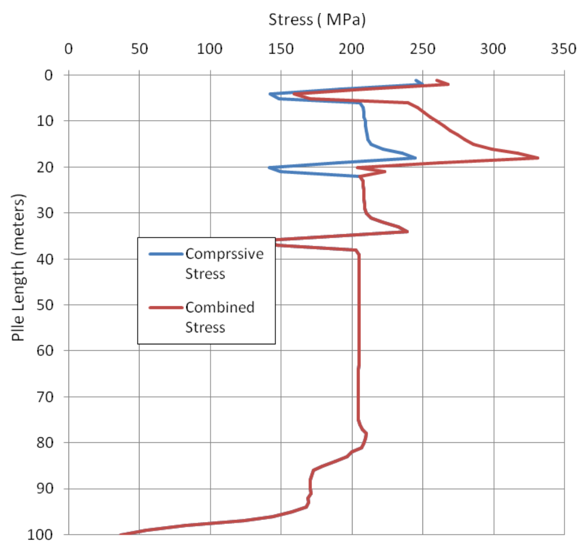

Figure 1 - Site A pile stress profile

At final driving, the CAPWAP estimated pile capacities ranged from 8.1 to 11.5 MN with final recorded blow counts of 17 to 25 blows per 0.25 m. Restrike testing of one pile was performed at a final tip penetration of 42.5 m after a waiting period of 4.5 days. This unusually long waiting time was only possible due to a significant weather delay which occurred during the platform installation. Restrike testing indicated an ultimate pile capacity of 31 MN with a recorded blow count of 50 blows for 50 mm. In addition, a skin friction setup factor of 3.1 was calculated when comparing the end of initial driving skin friction with the beginning of restrike skin friction. Finally, the ultimate pile capacity for 60 days after pile installation may be estimated using the techniques developed by Bullock et al. (2005). This technique uses the end of driving pile capacity and the restrike pile capacity to develop a plot of pile capacity with the log of time. Using this method the estimated ultimate pile capacity at 60 days after pile installation ranged from 29.2 to 36.8 MN for all four piles tested. Figure 2 provides a plot of these capacities versus the static pile capacity calculations based upon the API methods.

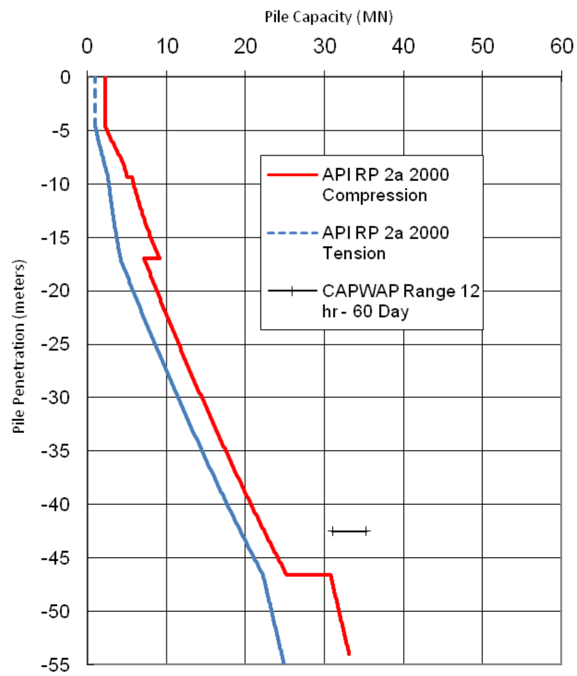

Figure 2 - Site A static versus DMS capacities

Site B

At this platform location, a total of six open ended pipe piles were proposed to be driven to a design penetration of approximately 60 m. The piles were to be driven in a total of three sections with the first section having a length of 50 m and the remaining two sections having lengths of approximately 19 m each. The water depth at this location was approximately 14.4 m and the primary subsurface conditions consisted of medium dense to very dense fine sand as summarized below in Table 3.

Table 3 - Site B soil conditions

From (m)	To (m)	Description
0	7	Loose to medium dense calcareous sand
7	10	Very stiff to hard calcareous clay
10	53	Very dense fine calcareous sand with silt & clay layers
53	58	Hard calcareous clay
58	62	Very dense fine calcareous sand
62	64	Hard calcareous clay
64	75	Very dense fine calcareous sand

The piles had an outside diameter of 1067 mm and a wall thickness of 38 mm except over the final three meters of pile length where the wall thickness was increased to 44 mm. Based upon the static pile capacity calculations at a final penetration of 60 m, the expected pile capacity was approximately 27 MN when calculated using the API recommended methods and 32 MN when calculated using the ICP methods (Figure 3). However, the actual platform design suggested that the required pile capacity ranged from 20.3 to 23.8 MN depending upon the pile location. Based upon the static pile capacity calculations shown in Figure 3 it appears that the required pile capacities of 20.3 to 23.8 MN could be obtained at pile penetrations of 30 to 60 meters depending upon the method used. However, both methods indicated a localized increase in pile capacity at about 40 m.

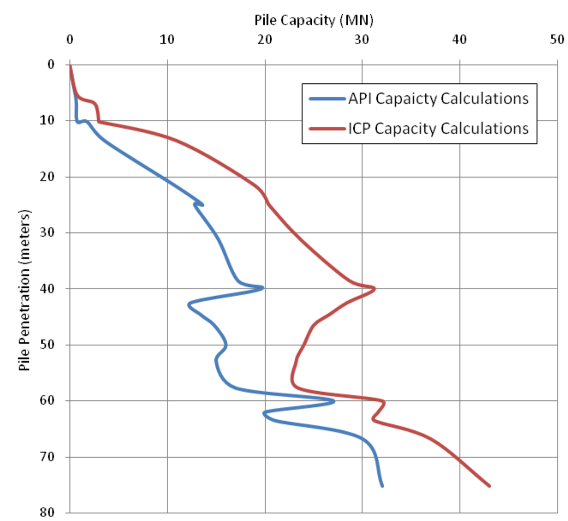

Figure 3 – API RP 2a and ICP pile capacity calculations

Based upon the above circumstances, DMS testing was determined to be desirable for this installation. In general, based upon previous testing results in this area as well as the subsurface conditions indicated at this location, it was believed that driving of the piles to the full 60 m penetration may not be necessary. As such, DMS testing for each of the six piles was performed. Testing of the first pile sections was performed and the driving of these sections to a pile penetration of approximately 27.5 m indicated pile capacity estimates ranging from 10.7 to 12.5 MN. At the restart of driving, CAPWAP analyses indicated that the pile capacity increased slightly, and that a skin friction setup factor of approximately 1.2 could be expected. Driving of the piles to a pile penetration ranging from 42.25 to 44.25 meters resulted in CAPWAP estimated pile capacities ranging from 21.1 to 25.8 MN. Restrike testing of one pile after a waiting period of 1.5 days indicated a skin friction soil setup factor of 1.25. Using the techniques described above, the pile capacity at a time period of approximately 60 days after installation has been provided. Ultimate pile capacity estimates ranged from 27.1 to 29.2 MN using these techniques. Figure 4 displays a plot of the estimated ultimate pile capacities for the piles tested and the static pile capacity calculations using the API and ICP methods.

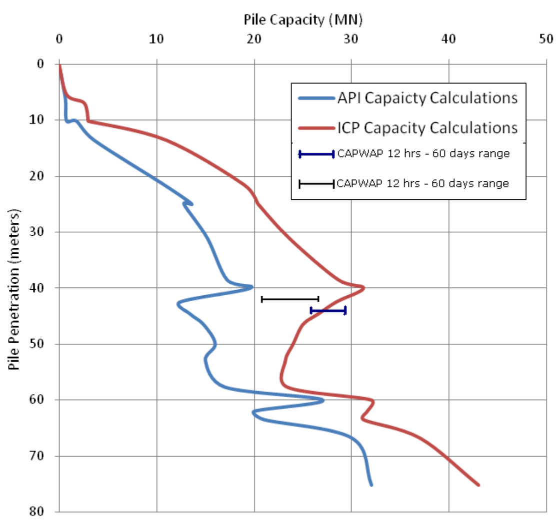

Figure 4 – API RP 2a, ICP and DMS pile capacities

Site C

At this site, four open ended piles having a 1067 mm outside diameter were proposed to be driven to final planned pile penetrations of 47 to 52.5 meters. However, the soil conditions presented in the soil report indicated that difficult or refusal driving could be expected. Specifically, the soil profile indicated the primary soil conditions consisted of hard calcareous clays with several lenses, seams or layers of gypsum rock. The gypsum rock is well known to have highly variable strengths varying from very weak to extremely strong. The general soil profile is summarized in Table 4.

Table 4 - Site C soil conditions

From (m)	To (m)	Description
0	1.5	Loose to medium dense calcareous sand
1.5	3.9	Very stiff to very hard calcareous clay
3.9	4.8	Moderately strong gypsum
4.8	9.2	Hard calcareous clay
9.2	12	Moderately weak to moderately strong gypsum
12	16.4	Very stiff calcareous clay
16.4	17.8	Moderately weak gypsum
17.8	24.1	Hard calcareous clay
24.1	26	Moderately weak gypsum
26	28.5	Hard calcareous clay
28.5	31.5	Moderately weak gypsum
31.5	32.9	Hard calcareous clay
32.9	38.9	Moderately weak gypsum
38.9	41	Hard calcareous clay
41	44.7	Moderately strong gypsum
44.7	46.7	Medium dense, weakly cemented, calcareous sand
46.7	54	Hard, calcareous clay

Because these soil conditions were identified, it was expected that refusal driving may occur at one of the gypsum layers. Since refusal driving could be expected, careful consideration of the refusal criteria and hammer energies to be used needed to be made during pile driving. Depending upon the depth where refusal driving actually occurred, significantly higher dynamic compression stresses at the pile toe could be expected. If the pile penetration at refusal was very low then minimal skin friction would be present. With minimal skin friction and extremely high end bearing, the stresses at the pile toe could be twice that measured from the DMS testing at the pile head. This condition would be further aggravated by the fact that the piles are driven on a batter angle. As such, the pile toe may only be in partial contact with the rock surface which would also create a condition where non-uniform stress conditions could be experienced. Of course such non-uniform stress conditions could not be evaluated using the DMS testing results. As such, caution would need to be used when driving through the gypsum layers at shallow pile penetrations.

Due to these subsurface conditions it was considered extremely important that DMS testing be performed during the entire driving of the platform piles. In fact, refusal driving conditions were encountered at a pile penetration of only 10 meters below the sea bed. At final driving the Menck MHU 500T hammer energy was reduced to approximately 50% of the maximum rated hammer energy. At this lower hammer energy setting the pile top stress was approximately 190 MPa. However, based upon CAPWAP analyses the maximum compression stress near the pile toe was 280 MPa. These stresses are average stresses over the entire pile area. If the pile toe is only in partial contact with the rock surface then it could be expected that higher eccentric pile stresses may have occurred.

At final driving the estimated pile capacity was approximately 26 to 29 MN for compression loading. Based upon the platform design these compression pile capacities were well above the required capacities. However, tension capacity requirements were not met based upon the results obtained for the first pile tested. As such, one of the remaining three piles was driven to a pile penetration of only 9.5 m and was then allowed to set for slightly more than 12 hours. Restrike testing on this pile indicated an estimated tension pile capacity of 6.7 MN, or a factor of safety of 1.5. It should be noted that the estimated tension pile capacity was calculated as two thirds of the skin friction resistance from the CAWPAP analysis. The two thirds factor is typical for such calculations.

Based upon the DMS results it was shown that that platform piles had obtained the required compression and tension loading. Figure 5 displays the results for tension pile capacity along with the predicted compression and tension pile capacity from the API static pile capacity calculations. In addition, results from nearby pile pull out tests are also provided which appear to show relatively good agreement with the DMS results. The final analysis needed to accept the platform piles was that sufficient lateral pile capacity would be available.

These analyses were provided by others and the platform piles were accepted at this minimal pile penetration of 10 m.

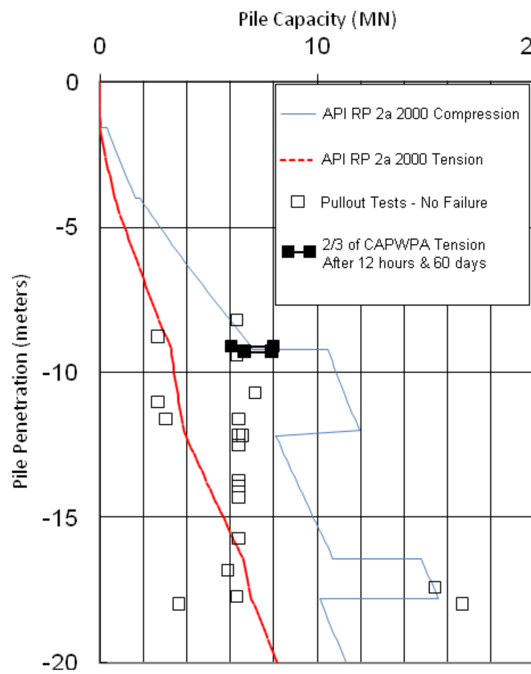

Figure 5 - Pile capacity from API, DMS and Pile pullout tests

Site D

Once again, this platform consisted of four main piles having an outside diameter of 1067 mm and expected final penetration depths of 54 and 60 meters. However, the soil boring performed at this location indicated that the predominant subsurface conditions consisted of alternating layers of calcareous clays and gypsum rock. As such, difficult driving and even refusal driving on the gypsum rock layers was expected similar to Site C. The general soil profile is summarized in Table 5.

Based upon previous installations in this area as well as the soil boring information, it was expected that refusal driving might be encountered in the gypsum layer at approximately 31.5 meters. However, refusal driving conditions might occur prior to this penetration and therefore, caution needed to be exercised to prevent the piles being damaged during hard driving. As such, the Menck

MHU 500T hammer energies were reduced to approximately 50% or lower to prevent pile toe damage as discussed above.

Table 5 - Site D soil conditions

From (m)	To (m)	Description
0	3.1	Loose calcareous, clayey sand
3.1	6.7	Very stiff to hard calcareous clay
6.7	9.0	Very weak gypsum inter-layered with clay
9.0	10.5	Hard calcareous clay
10.5	11.5	Very weak gypsum with clay layers
11.5	13.5	Hard calcareous clay
13.5	14.7	Very weak gypsum with clay layers
14.7	21	Hard calcareous clay
21	22.3	Very weak gypsum with clay layers
22.3	31.5	Hard calcareous clay
31.5	34.8	Very weak gypsum with clay layers
34.8	38.6	Hard calcareous clay
38.6	41.6	Very weak gypsum inter-layered with clay
41.6	47.7	Hard carbonate clay
47.7	51	Weakly to moderately cemented calcareous silty sand
51	55	Hard calcareous clay
55	58.6	Very weak gypsum with clay layers
58.6	68	Hard calcareous clay

Unexpectedly hard driving was encountered at a pile penetration of 18.5 m even though a gypsum layer was not indicated at this depth. The blow count increased to 670 blows per 0.25 m (800 blows per ft) with a hammer energy setting of about 60% of the maximum rated energy. A CAPWAP analysis indicated that the maximum uniform dynamic compression stress was approximately 225 MPa and occurred not at the pile toe but at the top of the stabbing guide for the add-on pile section. Compression stresses at the pile toe were estimated to be approximately 170 MPa. Considering that the pile toe may only be in partial contact with the rock surface, it was estimated that the pile toe stresses could be as much as twice that indicated by the CAPWAP analysis. However, due to the inconsistent result of refusal driving where a gypsum layer was not indicated, it was decided that an increase in the hammer energy would be allowed to determine if the piles could be driven to a deeper depth. The Menck MHU 500T hammer energy was increased to approximately 80% of the maximum energy and the piles were driven to a final penetration of 29 m where again refusal driving was encountered. Pile top stresses were indicated to be approximately 225 MPa based upon the DMS testing. CAPWAP analyses indicated that the maximum stress along the pile shaft was approximately 260 MPa. Pile damage was not indicated during the DMS testing.

Finally, pile capacities at final driving were indicated to be approximately 17 to 26 MN based upon CAPWAP analyses. Two of the piles were restruck after waiting periods of approximately 17 and 31 hours. Based upon these restrikes, skin friction setup factors of 1.4 to 1.5 were estimated. If the restrike testing is used to estimate the ultimate pile capacities after 60 days using the techniques discussed above, these capacities would then range from 28.5 to 39.6 MN. However, considering the soil conditions encountered, this setup analysis may not be appropriate. Specifically, based upon the DMS testing, it appears that the end bearing resistance increased during the restrike driving from that predicted at the end of initial driving. Considering that the piles were expected to be end bearing on gypsum rock, it is considered unlikely that such a change in the end bearing resistance would be encountered. It is more likely that during restrike driving a slightly higher hammer energy setting was used, resulting in the higher predicted end bearing results.

Site E

At this platform location, a total of four 1067 mm OD x 38 mm wall thickness open ended pipe piles were expected to be driven in two sections to design penetrations ranging from 30 to 48 meters depending on the location. The piles had a 45 mm x 3 m long driving shoe. The soil boring performed at this location indicated that the main soil type is silica sand, with few clay layers. Weak to moderately strong sandstone layers with laminations were identified between 10.0 and 26.0 m below sea bed. The borehole also indicated a thin calcarenite layer 0.6 m thick at the sea bed level. The general soil profile is summarized in Table 6.

Table 6 - Site E soil conditions

From (m)	To (m)	Description
0.0	0.6	Moderately weak calcarenite
0.6	2.8	Shell debris
2.8	4.5	Medium dense to dense calcareous silica sand
4.5	7.6	Firm to hard calcareous sandy clay
7.6	10.0	Very dense silica sand
10.0	15.0	Weak to moderately strong sandstone
15.0	19.2	Very stiff to hard sandy clay
19.2	26.0	Dense to very dense silica sand, locally cemented, with laminations of weak to moderately strong sandstone
26.0	30.0	Medium dense to dense calcareous silica sand
30.0	36.1	Dense to very dense silica sand
36.1	39.0	Slightly cemented calcareous silica sand
39.0	43.6	Dense to very dense silica sand
43.6	46.3	Hard to very hard calcareous sandy clay
46.3	56.2	Dense to very dense calcareous silica sand
56.2	59.0	Hard calcareous clay
59.0	67.0	Dense to very dense silica sand

Based upon previous installations in this area as well as the soil data information, refusal driving was not initially anticipated before a pile penetration of 46.5 m. However, because of numerous CPT refusals and poor recovery of material in the sandstone layers and laminations, hard driving conditions could be expected between 10.0 and 26.0 meters of penetration and caution needed to be exercised to prevent pile toe damaged during driving. The first section of pile A1 was driven to a penetration of 24.5 m and hard driving was encountered at 11.5 m (248 blows per 0.25 m) and 19.25 m (210 blows per 0.25 m) with the Menck MHU 500T hammer energy reduced to about 60%. After splicing, driving continued with the Menck MHU 500T hammer running between 65 and 80% and blow counts ranging from 70 to 130 blows per 0.25 m. Even though the measured stresses remained below 225 MPa, buckling of the pile top occurred at about 38 m of penetration after approximately 5500 hammer blows on the add-on section. It was assumed that the most likely cause of the pile top damage was steel fatigue. After cutting-off 1 m at the pile top, pile A1 was driven another 2500 blows to a penetration of 42.0 m, in the dense to very dense silica sand. The pile was restruck after 12 hours and two CAPWAP analyses were carried out; one at end of drive and one at the beginning of restrike. Based upon the restrike, a skin friction setup factor of 1.3 was estimated and pile A1 achieved a capacity of 26.2 MN at beginning of restrike, versus 23.5 MN required. Pile A2 refused at a penetration of 20.5 m. The blow count increased to 668 blows per 0.20 m with a hammer energy setting of about 80% of the maximum rated energy. A CAPWAP analysis indicated a mobilized capacity of 32.8 MN, versus 23.7 MN required and a maximum uniform dynamic compression stress of 242 MPa occurring at the pile toe. Pile B1 was driven to a final penetration of 20.5 m, with a hammer energy setting of 80% at end of drive and a reported final blow count of 135 blows per 0.25 m. A CAPWAP analysis indicated a mobilized capacity at end of drive of 19.9 MN, versus 14.7 MN required and a maximum uniform dynamic compression stress of 249 MPa occurring at the splice level. Pile B2 refused at a penetration of 12.6 m. The blow count increased to 272 blows per 0.10 m with a hammer energy setting of about 65% of the maximum rated energy. A CAPWAP analysis indicated a mobilized capacity of 21.7 MN, versus 14.3 MN required and a maximum uniform dynamic compression stress of 226 MPa occurring near the pile toe.

The final analysis needed to accept pile B2 was that sufficient lateral pile capacity would be available. This analysis was provided by others and the pile was accepted at this minimal pile penetration of 12.6 m.

Soil resistance to driving was much harder than anticipated for this location, and more than 10 000 blows were required to drive pile A1 to a final penetration of 42 m. Although the measured compression stresses remained below 225 MPa during driving of pile A1, 1 m of pile top had to be cut-off after buckling at a pile penetration of 38 m. The pile top buckling was most likely due to steel fatigue. Based on DMS testing and CAPWAP analyses, the capacities of piles A2, B1 and B2 could be obtained at penetrations shorter than the design penetrations, preventing damage to the pile due to steel fatigue.

Conclusions

As described above, DMS testing and analysis can be performed effectively and efficiently for offshore projects. For normal above-water driving projects, DMS testing can be performed with little or no interruptions to the construction sequence. This can be accomplished by mounting of the DMS test gages to the piles prior to lifting and setting in place, and removal of the test gages after driving of each section. Assistance to the contractor may also be provided with recommendations for hammer energy settings or sizes to be used for each pile section driven. In this way the maximum energy may be used which will not result in damage to the piles.

The five examples provided here show that DMS testing can and should be used for evaluation and acceptance of offshore piles. At four of the five platforms, pile lengths were shortened or pile acceptance was provided when refusal driving was encountered. For the remaining platform, the desired ultimate pile capacity was obtained based upon restrike tests performed after waiting 4.5 days. Although this waiting period was substantially longer than expected, the skin friction setup factor was greater than three. Finally, if necessary, estimates of the pile capacity for 60 days after pile installation may be provided using the techniques suggested by Bullock et al. (2005).

Based upon our experience, DMS testing and analysis is capable of measuring the desired ultimate pile capacity if restrike testing can be provided after a waiting period of only 1 to 2 days. However, where pile diameters are relatively large and piles are driven into sensitive clays, this may become more and more difficult due to the limited size of the contractors' hammers or the overall drivability of the pile section.

Finally, DMS testing also provides a thorough evaluation of the stress conditions to be experienced from the pile driving. This is provided through real time DMS measurements at the pile head and by performing preconstruction drivability analyses to estimate the expected stresses either at the pile toe or along the pile shaft. Pile toe stresses while driving to rock has been shown to result in pile damage. As such, driving under such conditions should not be performed without a thorough evaluation of the expected stresses. In addition, pile damage can also result due to over stressing the piles in a combined static bending and dynamic compression stresses. Therefore, driving under these conditions should also not be performed without a thorough evaluation of the expected stresses.

Note from the authors

This paper was partially published in "Proceeding of the Eighth International Conference on the Application of Stress Wave Theory to Piles, 2008" and has been updated with additional data and examples that further support the paper conclusions.

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