Offshore pile acceptance using dynamic pile monitoring

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ABSTRACT: 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. Specifically, analysis of the collected data for both stress control and pile capacity estimates need to be provided either during driving or shortly after driving completion for pile acceptance. In offshore pile driving, piles are accepted within several hours after driving and construction proceeds so there is no opportunity for further analysis, testing, or driving. In addition, 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, whose utility and reliability has been proven on thousands of projects, 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 provides the soil resistance distribution along the pile shaft and toe. Finally, the use of hydraulic hammers for driving of these piles also adds the requirement of controlling hammer energy such that over-stressing and buckling 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.

Four examples of platform installations where DMS testing was preformed are presented. The testing results are compared with the expected or calculated pile capacities from the static pile capacity calculations. Discussion of the hammer energy setting and resultant pile stresses are also provided. The advantages provided from the DMS testing are discussed for each platform installation. Based upon these four examples the use of DMS testing to evaluate offshore piles and provide significant cost savings to the project is clearly shown. At three of the four platform locations the final pile penetrations are substantially shorter than that which was expected based upon the static pile capacity calculations. In addition, proper control of the hammer energy was maintained such that buckling of the piles would not occur during driving.

1 INTRODUCTION

Dynamic pile testing using the Pile Driving Analyzer has been used for evaluation of driven piles for nearly 50 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 primarily in the oil and natural gas industry has not been so widely accepted. In fact, much of the early testing for offshore structures was primarily to evaluate hammer performance, with results concerning pile stresses and 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.

2 PRECONSTRUCTION DRIVEABILITY 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 driveability analysis designed to:

- assess the anticipated driving conditions and to determine what, if any, special precautions may be necessary during pile installations
- to assess the ability of the hammer(s) to restart pile driving after splicing without over stressing
- to determine if the proposed hammer(s) are suitable for the pile installations
- provide recommendations concerning the predicted dynamic compressive stresses
- predict the combined dynamic and static bending stress.

The driveability 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 made 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 for hydraulic hammers or limitations for pile stick up during driving, as well as the potential for pile refusal prior to the final design penetration.

3 FIELD TESTING/ANALYSIS

DMS testing should be provided for pile installations if there is a potential for cost savings or where difficult driving conditions exist. The operator of the projects described in this paper specifies DMS testing based upon cost savings accrued and documented over several years of experience. Typically, the DMS engineer can install test gages 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 placement. Testing may then be provided over the entire driven pile length, or for those sections near the end of pile installation. While the static bending stresses are not measured through DMS testing, they can be calculated for the given pile stickup, and the expected dynamic stresses at these locations should be estimated either by wave equation analyses or from the pile top DMS measurements. The DMS measurements are used to determine proper hammer energy settings to avoid over-stressing of the piles by either dynamic compression stresses or the combined dynamic compression and static bending stresses. Control of the hammer energy output for hydraulic hammers or use of appropriately sized steam or diesel hammers should be carefully monitored and controlled by the DMS testing. Finally, evaluation of the static pile capacity should be provided at final driving. Usually, the Case Method equations normally used for pile capacity estimates are of little use for typical offshore piles. This is due to 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. Finally, 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 usually involves testing after waiting periods of between 12 and 48 hours due to construction constraints. However, 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 four recent platform installations where DMS testing was provided are summarized below. All four 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 $\times$ 38 mm pile top wall thickness, open ended pipe piles driven with a Menck MHU 500T hydraulic hammer. All of the piles had a driving shoe with variable wall thickness depending upon the site.

3.1 Site A

At this platform location a total of four piles were proposed to be driven to penetrations ranging from 37 to 42.5 meters, based on pile loads ranging from 16 to 20 MN. The piles had a 45 mm wall thickness $\times$ 3 meter long driving shoe at the pile bottom. The piles were planned to be driven in three sections, the first section (P1) was 67 meters long and the remaining sections were 19 (P2) and 17 (P3) meters long. The water depth at this location was 47 meters and the general soil profile consisted of primarily very stiff to hard carbonate or calcareous clays as described below.

Based upon the subsurface profile and experience from previous platform installations, it was expected that the piles would drive easily to the design penetration. Ultimate pile capacity estimates were specified to be provided by minimum 12 hour 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

Table 1. Site A soil conditions

From (m)	To (m)	Description
0	11	Very stiff to hard calcareous clay
11	17	V. 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

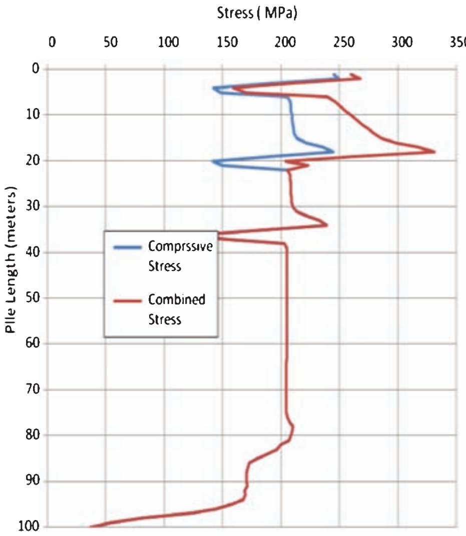

Figure 1. Site A Pile Stress Profile.

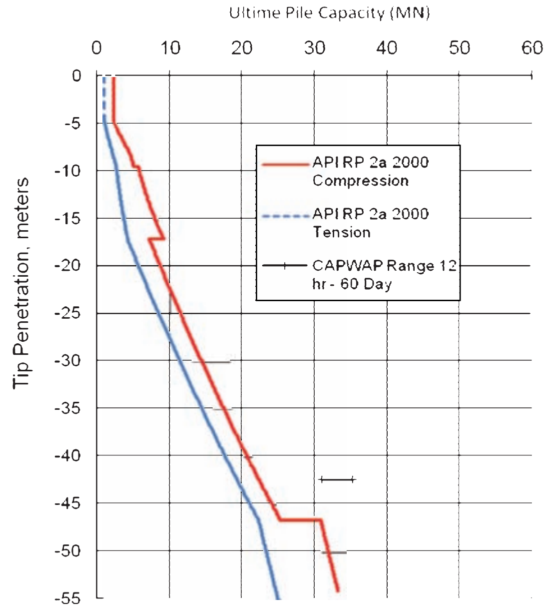

Figure 2. Site A static vs DMS capacities.

as well as to allow for setup calculations to predict pile capacities.

Based upon the preconstruction driveability 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 driveability 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. Fig. 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 for the P3 section where the pile area was increased resulting in a localized stress maximum. 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). This combined stress (336 MPa) was less than the pile yield strength of 345 MPa and therefore acceptable. However, this stress occurred when the hammer was being operated at a hammer energy setting of approximately 80%.

At final driving, the CAPWAP estimated ultimate pile capacities ranged from 8.1 to 11.5 MN with final recorded blow counts of 17 to 25 blows per 0.25 meter. Restrike testing of one pile was performed at a final tip penetration of 42.5 meters after a waiting period of 4.5 days. This unusually long waiting time was only possible due to a significant weather delay. 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. 2004. 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. Fig. 2 provides a plot of the DMS based ultimate pile capacities versus the static pile capacity calculations based upon the API methods.

3.2 *Site B*

At this platform location, a total of six 1067 mm diameter open ended pipe piles were proposed to be driven to a design penetration of 60 meters. The piles were to be driven in three sections with the first section having a length of 50 meters and the remaining two sections having lengths of 19 meters each. The pile bottom section had a 3 meter long × 44 mm wall thickness driving shoe. The water depth at this location was 14.4 meters and the primary subsurface conditions consisted of medium dense to very dense fine sand as summarized in Table 2.

Based upon the preliminary geotechnical report static pile capacity calculations, the expected pile capacity was 27 MN for a final pile penetration of

Table 2. 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 w/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

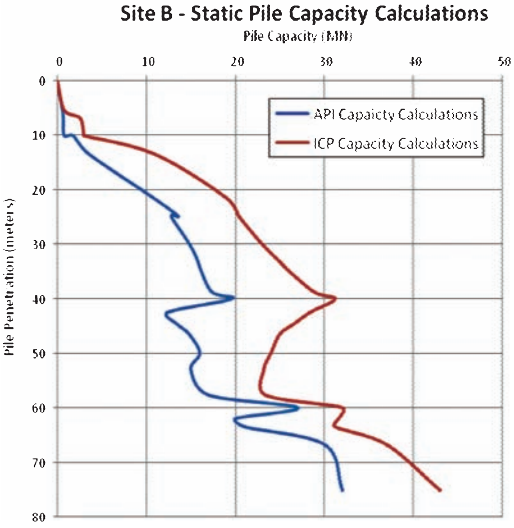

Figure 3. API & ICP static pile capacity calculations.

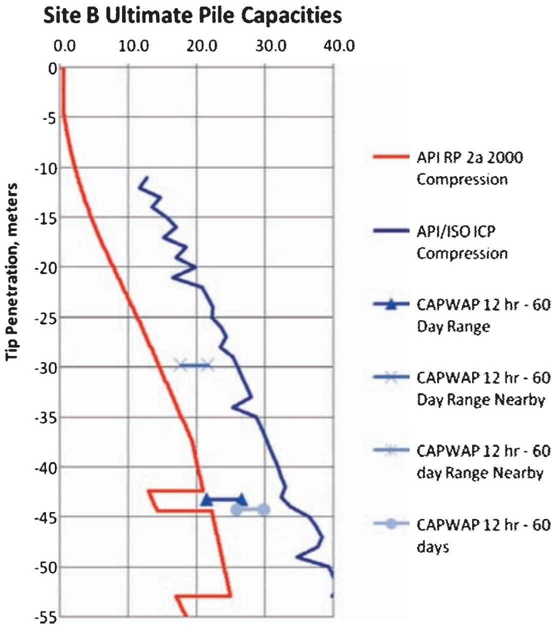

Figure 4. API, ICP and DMS pile capacities.

60 meters, when calculated using the API recommended methods and 32 MN when calculated using the ICP methods (Fig. 3). However, the 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 capacities plotted in Fig. 3 it appears that the desired ultimate pile capacities could be obtained at pile penetrations of 30 or 60 meters depending upon the method used. However, for both methods it appears that a localized increase in pile capacity was expected at about 40 meters. Based upon this data the final pile penetration was set at 60 meters to provide a conservative approach to the pile design.

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 meter penetration may not be necessary.

As such, DMS testing for each of the six piles was authorized. Testing of the first pile sections was performed and the driving of these sections to a pile penetration of approximately 27.5 meters 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 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 60 days after installation was estimated. Ultimate pile capacity estimates ranged from 27.1 to 29.2 MN using these techniques. Fig. 4 displays a plot of the estimated ultimate pile capacities for the six piles tested and the refined static pile capacity calculations using the API and ICP methods.

3.3 Site C

At this site, four 1067 mm diameter piles were proposed to be driven to final planned pile penetrations of 47 to 52.5 meters. The pile was equipped with a 3 meter long × 51 mm wall thickness driving shoe. Previous experience and 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 with many layers indicated to be extremely strong. The general soil profile is described in Table 3.

Because these soil conditions were identified, it was expected that refusal driving may occur at one of

Table 3. Site C soil conditions

From (m)	To (m)	Description
0	1.5	Loose to medium dense calcareous sand
1.5	3.9	Stiff to very stiff 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

the gypsum layers. Detailed lateral pile analyses indicated piles driven to at least 9 meter penetration would have sufficient lateral capacity, so it remained for DMS measurements to prove compressive and tension capacity at such shallow penetrations. Since very shallow 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 would be driven on a batter angle. As such, the pile toe may only be in partial contact with the rock surface which would create a condition with additional non-uniform pile toe stresses. 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 500 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 CAPWAP capacity estimates ranged from 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 meters just slightly above the gypsum rock. This pile was then allowed to set for nearly 18 hours. Restrike testing on this pile indicated a CAPWAP estimated tension pile capacity of 6.7 MN, or a factor of safety of 1.5 for the design tension loads. 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 calculation is typical for such calculations.

Based upon the DMS results it was shown that that platform piles had obtained the required compression

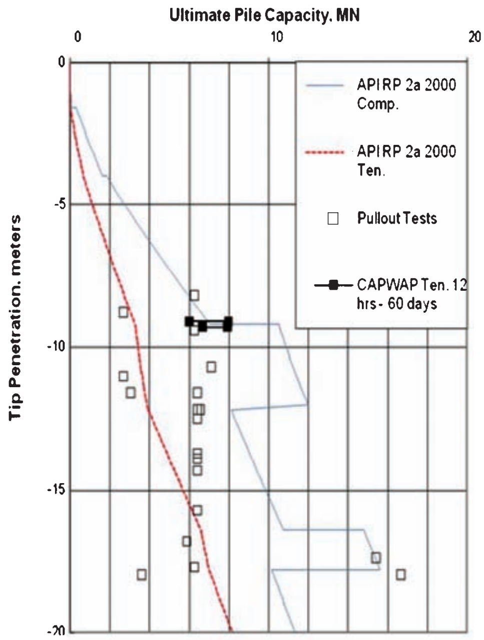

Figure 5. Pile capacity from API prediction, DMS results and pile pull out tests.

and tension capacities. Fig. 5 displays the results for compression and tension pile capacity along with the predicted pile capacity from the static pile capacity calculations. In addition, results from nearby pile pull out tests are also provided which show relatively good agreement with the DMS results. The final analysis needed to accept the platform piles was to show 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 meters.

3.4 *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. Based on drivability analysis and experience, a 3 meter long × 51 mm wall thickness driving shoe was provided. 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 as shown in Table 4.

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 pile damage during hard driving. The Menck MHU 500 hammer energies were, therefore, reduced to 50% or lower to prevent pile toe damage as discussed above. Unexpectedly hard driving was encountered at a pile penetration of 18.5 meters even though a

Table 4. Site D Soil conditions at Site D

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	V. 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

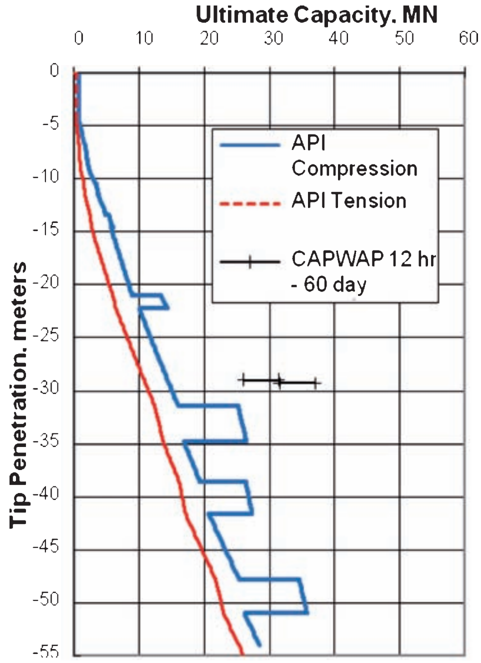

Figure 6. Site D Pile capacity from API and DMS results.

gypsum layer was not indicated at this depth. The blow count increased to 670 blows per 0.25 meter (800 blows per foot) 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 500 hammer energy was increased to approximately 80% of the maximum energy and the piles were driven to a final penetration of 29 meters 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 17 to 26 MN based upon CAPWAP analyses. Two of the piles were restruck after waiting periods of 17 and 31 hours. Based upon these restrikes, skin friction setup factors of 1.4 to 1.5 were estimated along with 60 day capacities ranging from 28.5 to 39.6 MN as shown in Fig. 6. However, considering the soil conditions encountered, the setup analysis may not be appropriate for these conditions. 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 large change in the end bearing resistance would be encountered. It is more likely that during restrike driving a higher hammer energy setting resulted in a higher end bearing activation.

4 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 pile sections 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 hammer 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 four examples provided here show that DMS testing can and should be used for evaluation and acceptance of offshore piles. At three of the four platforms pile lengths were shortened or pile acceptance was provided when refusal driving was encountered. For the Site A platform, the desired ultimate pile capacity was obtained based upon restrike tests performed after waiting 4.5 days. This waiting period was substantially longer than can normally be expected, but it helped to show that 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 driveability 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 driveability analyses to estimate the expected stress maxima either at the pile toe or along the pile shaft as a function of measured pile top stress. Pile toe stresses while driving to rock have 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.

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