Design and Installation of Mobile Offshore Drilling Unit Mooring Piles using Innovative Drive-Drill-Drive Techniques

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ABSTRACT: The North West Shelf, which is Australia's largest offshore oil and gas province, is well known for the engineering challenges posed by the calcareous seabed sediments present and its exposure to extreme weather associated with tropical cyclones. Mobile and fixed platforms are used to tap into the hydrocarbon reserves and these need to be secured to the seabed using substantial foundations and anchors, which must resist onerous cyclic loads. This paper describes a project where an innovative drive-drill-drive installation approach was adopted to enable the installation of twenty-four anchor piles to moor a mobile offshore drilling unit (MODU) at two drilling sites. The two sites encompassed a wide range of complex seabed geologies, from high strength limestones to uncemented silts, where conventional drag anchors or driven anchor piles would not prove entirely suitable. Installation was carried out using a dynamically-positioned vessel, operated by Fugro, on which an inventive purpose-built drilling tower had been erected specifically for the project. The installation also included development and deployment of an ingenious seabed frame that could grip and release the piles progressively, plus casings to temporarily stabilise the drilled holes from collapsing sands. This project provides an excellent example of how multidisciplinary collaboration can embrace innovative solutions to successfully deliver complex projects safely, on budget and to an accelerated schedule.

1 OVERVIEW

1.1 *Project Description*

The North West Shelf (NWS) is an oil and gas province off the north west coast of Australia. The North West Shelf Joint Venture (NWS JV) represents Australia's most mature oil and gas development. It is centered around existing large offshore natural gas production platforms at Goodwyn and North Rankin, operated by Woodside Energy, which feed onshore processing facilities at the Karratha Gas Plant via long distance pipelines for the production of natural gas for use locally within Western Australia and worldwide export of liquefied natural gas (LNG).

As part of the ongoing development of the NWS JV project, an expansion project involving two additional sites was targeted for exploration and production, to the south west of a hub of existing process platforms. This expansion project, is simply referred to as the Project for the purposes of this paper and comprised two drill centre locations that are referred to as Site A and Site B hereinafter. Production wells at the two sites were required to be drilled by a Mobile Offshore Drilling Unit (MODU) that would be moored by twelve individual pre-installed anchors ar-ranged in four clusters of three anchors at each site.

1.2 *Background*

Typically, MODUs or semisubmersible drilling rigs, are secured using large drag anchors to ensure that they remain on-station and stable during drilling operations. For operations on the NWS, MODUs are, in many cases, required to be safely designed to endure sporadic but severe tropical storms (cyclones), which mostly occur during the summer months. These design conditions result in very large mooring loads, often fluctuating by up to several hundred tonnes on each anchor. The corresponding anchor arrays are typically spread over many hundreds of metres and this distance increases with increasing water depth. Mooring failures have occurred in this region and high levels of redundancy are usually sought to pre-vent potential damage or loss to neighbouring assets.

The NWS seabed geology is renowned for its variability and is dominated by carbonate or calcareous sediments with varying degrees of cementation. Based on a detailed review of the available geophysical and geotechnical data for the Project, it was deemed that drag anchors would not be suitable for many of the proposed anchor locations because of the presence of cemented sediments at shallow depth that would preclude anchor penetration and, thereby, limit the achievable capacities below required levels. Drag anchors previously used in other projects with similar settings have, on occasion, revealed unacceptable performance during installation, which has some-times required unplanned revision of their operational tolerances or remedial actions.

A large variety of alternative anchor concepts were therefore considered instead of conventional drag anchors. This process resulted in the selection of relatively large diameter pre-installed anchor piles as the preferred anchoring solution at both sites. Depending on the precise geotechnical conditions expected at each pile location, it was proposed that these piles would either be installed through drilling and grouting, or via a drive-drill-drive procedure.

The anchor piles were also designed to be installable using an available Fugro operated construction support vessel; one that was associated with ongoing inspection, maintenance and repair (IMR) support of the wider NWS JV project.

2 SEABED GEOLOGY

2.1 *Regional*

The considered Project is located within the Northern Carnarvon Basin, an offshore sedimentary basin covering some 535,000 km^2. Geologically, the Project lies along the northeast trending Rankin Platform at the offshore margin of the Dampier sub-basin. The region has been subject to episodic flooding and subaerial exposure associated with Quaternary and Tertiary glacial-interglacial cycles. Indeed, during the last glacial maximum the eustatic sea level has been estimated as approximately 130 m lower than present (Clark et al., 2009).

Quaternary deposition is dominantly biogenic carbonate sediments with limited terrestrial influx, largely depending on the proximity to the transient coastline and river systems. The shallow geology is correspondingly variable and reflects the complex interplay between cementation, erosion, and weathering during exposure and primary production, deposition, and sediment reworking during inundation.

Sea level fluctuations have also significantly influenced the more local geomorphology. The calcarenite ridges that dominate the region likely formed during a sea level lowstand. Subsequent inundation has resulted in sediment deposition in the troughs between the ridges.

Seasonal cyclones are common along the north west coastline of Australia. Cyclonic storms increase the energy regime resulting in sediment reworking, erosion, transportation and deposition of material coarser than the norm. The seabed would be episodically swept by high and erosive currents. Mainland flooding may also temporarily increase the influx of terrestrial sediments. Terrestrial analogues to this geology can be observed onshore around the Western Australian coastline and hinterland, including exposures of Miocene Trealla Limestone, intermixed with dune systems.

2.2 *Seabed Conditions - Site A*

A representation of the seabed bathymetry at the Project site is presented on Figure 1, with Site A situated in the foreground and Site B in the background. In this image, now submerged paleo-coastal features can be observed at Site A, including a headland, coastal dunes and shorelines, plus a paleo river channel cut-ting around and through them (see the right-hand side of Fig. 1). These features have been essentially locked in time because of the enduring cementation, which would have mostly occurred during long periods of subaerial exposure, resisting later severe erosional effects. It should be noted that the seabed at Site A is exposed to high currents, including those associated with cyclonic storms and in the absence of cementation the observed features would most likely have been obliterated over time.

The subsurface seabed conditions at Site A are illustrated by the schematic on Figure 2, which shows the geophysical data and generic geotechnical units across one cross-section, along with an inferred simplified stratigraphy. One of the major challenges for the Project was the limitation in available geotechnical data with which to ground-truth the geophysical data, which, in-itself, was ambiguous because of the lack of penetration in the relatively hard formation. The associated uncertainty was subsequently man-aged through prudent design assumptions, pragmatic construction contingency methods, backed-up with confirmatory observations during construction.

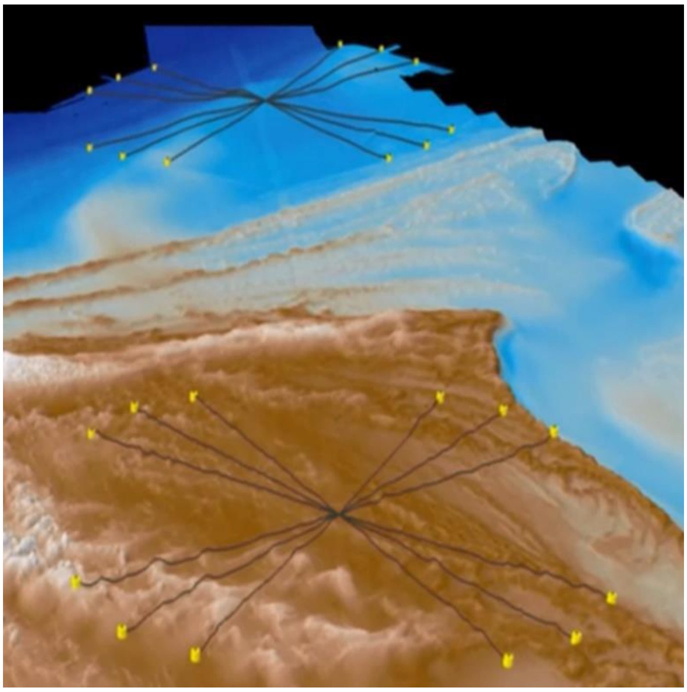

Figure 1. Representation of seabed bathymetry at Project site.

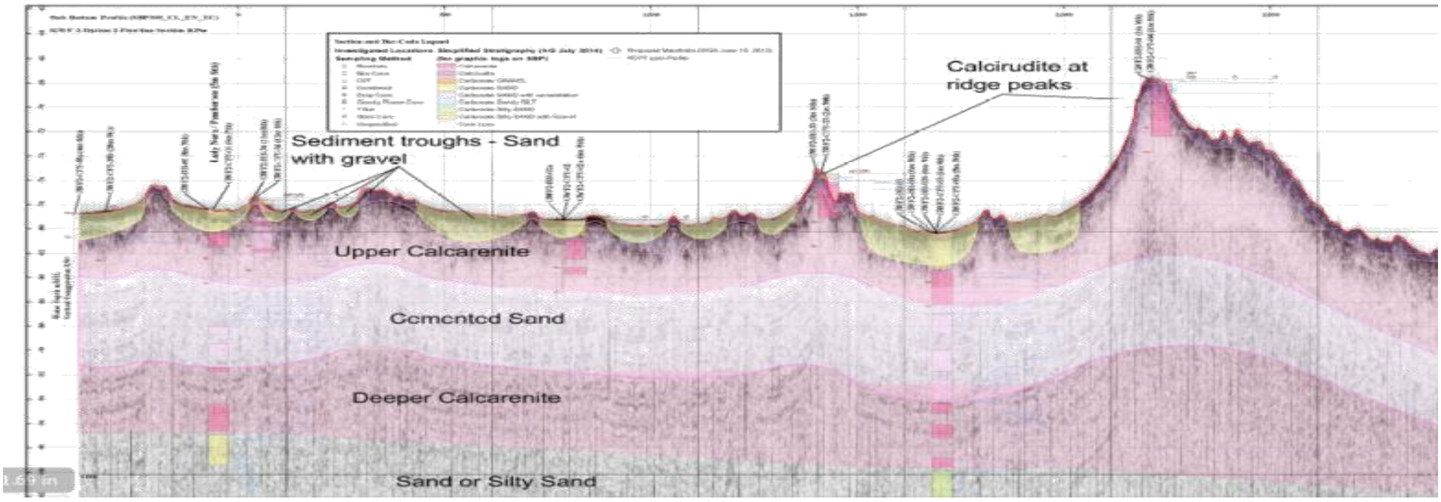

Figure 2. Schematic representation of shallow geology – Site A.

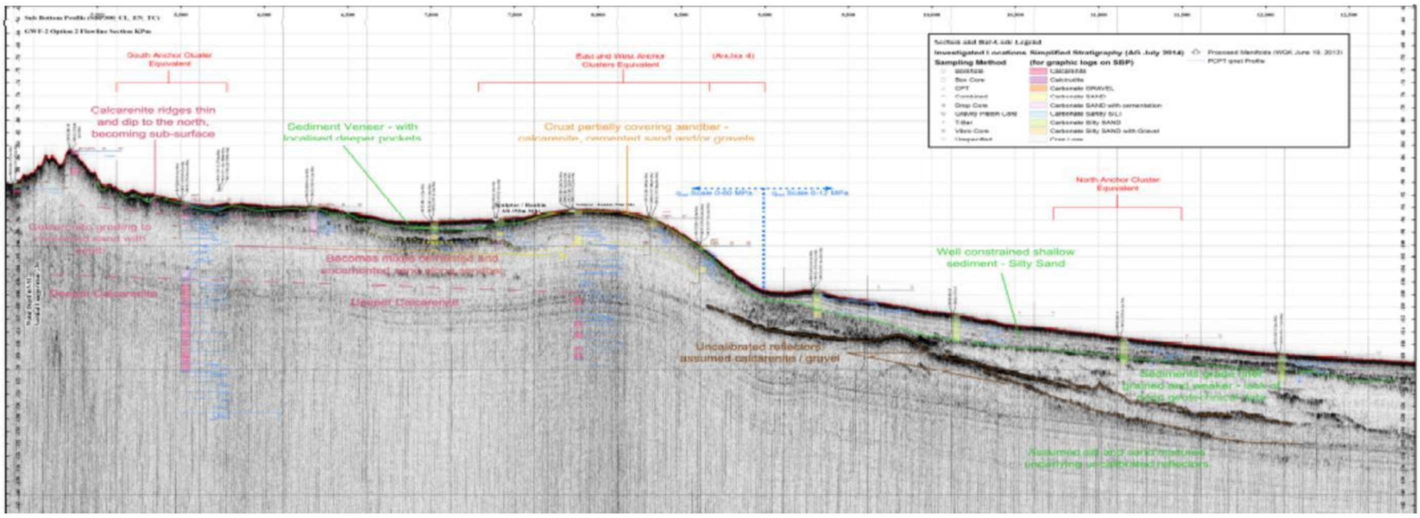

Figure 3. Schematic representation of shallow geology across both Project sites.

The stratigraphy at Site A included layers of cemented sand and calcarenite with pockets of sand above a rock subcrop, which elsewhere outcrops. Measured cone penetration resistances varied considerably across Site A, reflecting a wide range of cementation levels, with refusals occurring where higher strength calcarenite or limestone occurred.

In terms of in-place design, it was pragmatic to adopt a demonstrably conservative stratigraphy consisting of a maximum of 5 m of uncemented sand overlying rock. In terms of installation, it was necessary to consider a much wider range of potential stratigraphies varying from outcropping rock at the mud-line to varying depths of uncemented sand overlying rock.

Water depths across Site A ranged between approximately 70 m and 85 m.

2.3 *Seabed Conditions - Site B*

Water depths across Site B ranged from approximately 90 m to 115 m. Although the bathymetry at Site B appeared less complex than at Site A (see Fig. 1), being relatively flat and featureless, the underlying stratigraphy was considerably more complex. Figure 3 provides an indication of the complexity of the sub-seabed conditions moving from Site A (on the left) to Site B (on the right).

The considered geology across Site B comprised three main zones: (i) a zone of outcropping calcarenite ridges in the vicinity of the south anchor cluster, (ii) a relic sandbar associated with a palaeo-channel encompassing both the east and west anchor clusters, and (iii) a relatively flat and featureless zone where the water depth exceeds 110 m in the vicinity of the north anchor cluster. This complex geology made for a range of geotechnical conditions. As for much of Site A, sand overlaid rock for several of the anchor locations at Site B, while at some other locations, the depth to rock with any significant strength exceeded the embedment of the anchor. At other locations, the stratigraphy comprised alternating layers of uncemented and cemented strata.

This variability in the geology across Site B necessitated the consideration of separate anchor pile de-signs at the South, East/ West and North anchor clusters.

In general, the strength of the cemented strata at Site B was predicted to be considerably less than at Site A, based on the available data, although exceptions to this were observed during installation, which could have prevented driven pile installation.

3 ANCHOR DESIGN

3.1 *Geotechnical*

The geotechnical design of the anchors was performed in accordance with the global factors of safety for anchor piles for mobile moorings defined in API RP 2SK (API, 2005). A catenary mooring configuration was adopted and unfactored lateral design loads at the anchors were defined for Intact (324 t to 280 t) and Damaged (464 t to 525 t) conditions, where the latter corresponds to a scenario where one anchor-line is assumed to have failed.

The lateral performance of the anchor piles in the cemented sediments was assessed using Fugro's proprietary "CHIPPER" program (Erbrich 2004). This approach and a number of the relevant input parameters have previously been calibrated based on 3D finite element analyses and back-analysis of centrifuge model tests undertaken using weakly cemented carbonate sediments obtained from other sites on the North West Shelf. This analysis technique considers the progressive brittle failure of rock around the pile under the action of cyclic loads, and the correspond-ing degradation of the p-y springs and consequent changes in pile stress. In essence, the level of effec-tive fixity of the pile lowers with on-going loading and the structural design of the pile must account for this.

The lateral performance of the anchor piles in the uncemented sediments was assessed using Fugro's complementary "pCyCOS" program (Erbrich et al 2010). This program is based on a cyclic strength approach and is consistent with state-of-the-art design philosophies for other foundation systems such as shallow foundations and suction caissons. Parameters required to define the pCyCOS lateral transfer (p-y) model were developed from general envelope p-y curves for typical uncemented carbonate sediments from other sites on the North West Shelf with adjustments applied to ensure that the degree of cyclic degradation matched that observed in cyclic simple shear tests performed on seabed samples recovered from the vicinity of the Project location.

The same outer diameter (2.14 m) and wall thickness (50 mm) was adopted for all the anchor piles at both Site A and Site B. Required pile lengths were site specific and ranged between 16 m and 18 m for Site A, and between 16 m and 28 m for Site B, depending on the anticipated stratigraphy. The stratigraphies were later corroborated through construction observations.

3.2 *Structural*

The structural design of the anchor piles and all associated attachments was performed using working stress methods in accordance with API RP 2SK, API RP 2A (API, 2014) and AISC 360-10 2010 (AISC, 2010).

As already mentioned in Section 3.1, given the variability in the soil conditions, four different pile designs were adopted but a common outer diameter and wall thickness was deliberately maintained for each design (see Section 3.1). The Type A and Type B piles were 16 m and 18 m long, respectively and these were adopted at locations where the drilled and grouted installation method was adopted (see Section 4). These pile types included grout tubes attached to the outside of the pile and a stabbing guide at the pile toe (see Fig. 4). The Type C and Type D piles were adopted at locations where the drive-drill-drive installation method was adopted and ranged in length from 24 m to 28 m.

All four pile types were designed with an external fixed padeye situated at the top of the pile. Although a lower padeye can increase lateral pile capacity the surface padeye was adopted to allow future (in-service) inspection, plus prevent potential clashes with rock near the surface during installation and to eliminate uplift forces on the anchor.

The padeye was designed such that the pile was open for relief drilling and a remotely operated vehicle (ROV) could dock with the pile to assist with orienting the pile during installation. Amongst other fac-tors, driving fatigue was accounted for in the overall padeye design.

The structural utilization checks considered the combination of global bending stresses and local stresses generated in the pile at the interface between the relatively soft overlying sand/silt and considerably stiffer and stronger rock. These checks required consideration of the expected range in sand-rock interface depths as well as the potential range of low and high estimates of the respective sand/ silt and rock strengths.

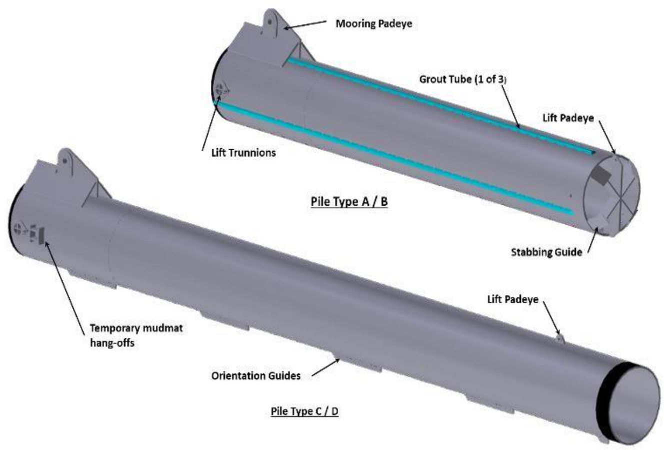

Figure 4. Anchor pile types.

To enable refinement of the structural design, finite element analysis was utilized to check the localised stress at the aforementioned interface as well as at the pad eye (Figure 5).

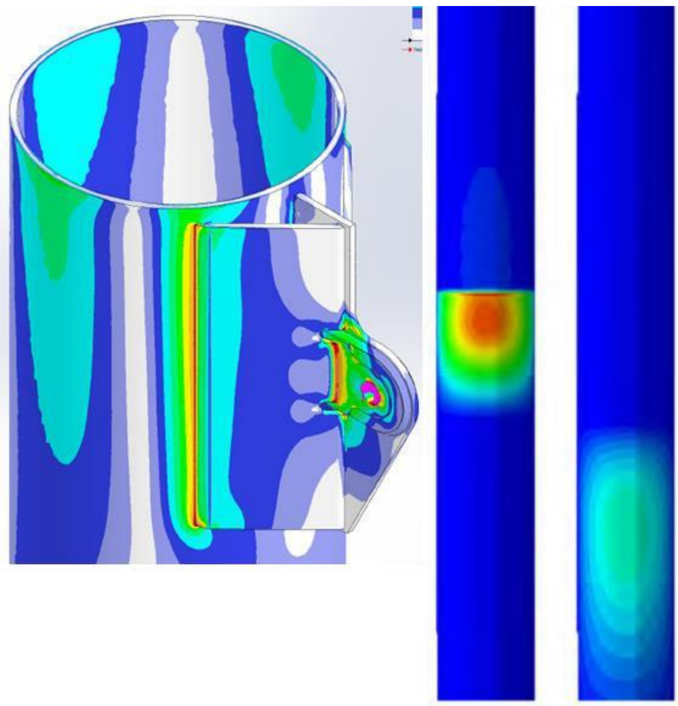

Figure 5. Finite Element Analysis: Pile padeye (left) and radial pressure distribution on the pile (right)

4 INSTALLATION

4.1 *Methodology and Installation Risk Management*

The installation methodology, in combination with the anchor design, was chosen to eliminate all foreseeable installation risks, which were identified during formal HAZID and constructability workshops. Many of these risks were addressed using the Safety-in-Design philosophy promoted by Woodside. In general, this led to increases in pile size to compensate for design uncertainty, which was deliberately moderated by the adoption of an observational approach during construction to confirm the design assumptions. However, this inevitably led to some increases in the levels of construction risk, both in terms of technical aspects and operational safety. Ultimately, a pragmatic process was adopted that sought to balance construction risks with design assurance.

The main technical construction risks that could adversely affect anchor performance included:

- Pile instability during initial installation. This could lead to toppling failure and damage to the construction equipment.
- Pile refusal during driving. Obviously, the design embedment had to be achieved to meet the functional specifications. Premature refusal, leaving a free-standing pile, could also present a vulnerability to the installation vessel, which required management.
- Driving damage. Excessive driving leads to damage to the pile and reduction in its fatigue life and, in the extreme, structural collapse of the pile toe.
- Hole collapse. Collapse of the bored hole could prevent the pile achieving the design embedment and could potentially cause a pile to become stuck.

Following detailed interdisciplinary assessments of different potential installation methods and equipment spreads, the drilled and grouted and drive-drill-drive installation methods where selected to deal with the full range of diverse ground conditions anticipated across the two sites.

The drilled and grouted installation method was adopted at anchor sites where rock occurred at or near seabed level and continued over the full embedment of the pile. Specifically, this installation method was adopted at all the Site A anchor locations and for the South anchor cluster at Site B, where out-cropping calcarenite ridges were identified. This method involved the drilling of an open hole in which the slightly smaller diameter piles could be inserted, with the drilled hole-pile annulus subsequently filled with grout following the pile installation. This method enabled installation of the piles without the need for a large driving template. However, stabilization was required over a limited depth of surficial uncemented material to prevent hole collapse where sands and gravels could run into the hole and trap the drilling equipment. A temporary drilling caisson was therefore adopted at sites where more than 2 m of surficial sand was anticipated, to prevent such soil run-in.

The drive-drill-drive installation method was adopted at anchor sites that featured more than 5 m of uncemented material at the surface. Specifically, this installation method was adopted at the East, West and North anchor clusters at Site B. This method initially involved driving of the pile to a pre-defined acceptable blow count, or the target penetration depth. In cases where the limiting acceptable blow count was attained during initial driving, relief drilling was performed through the centre of the pile before a second stage of driving was undertaken to install the pile to the target penetration depth. To facilitate the use of this technique, a relatively flat and level seabed was required for the landing of the driving template.

4.2 *Equipment*

4.2.1 *Installation Vessel*

The anchor installation was performed from the Southern Ocean, a Fugro operated dynamically positioned (DP2) installation vessel (see Fig. 6).

Figure 6. Installation vessel – Southern Ocean

4.2.2 *Drilling Equipment*

Both installation methods employed on the Project, utilized a flooded reverse circulation (RC) drilling method. This uses air injected above the drill bit to provoke water circulation and therefore cuttings return up through the centre of the drill pipe, before being discharged approximately 30 m below sea surface level. Figure 7(a) presents a schematic of the RC drilling concept.

Figure 7(b) presents a photograph of the purpose-built drilling tower installed on the Southern Ocean. The main components of the drilling system comprised:

- Bottom Hole Assembly (BHA), including the drill bit, centralisers and weight cans/ elements to keep the drill string in tension.
- RC drill pipe to connect the BHA to the power swivel and to provide air supply for lifting cuttings out of the hole.
- Power swivel to provide torque during drilling
- Drill tower to provide torque reaction during drill-ing.
- Vessel crane for supporting the top drive and controlling the weight on bit during drilling.

Two separate BHAs were employed on the Project, one for each of the two installation methods. The BHA, shown on Figure 8 for the drilled and grouted pile installation method, consisted of a cross-over piece from the drill pipe, stabilizers, air injection plate and the drill bit.

During drilling operations, the vessels crane was operated in Auto Tension mode to isolate the drill bit from the vessel motions and to allow accurate control of the weight on bit, while the vessels dynamic positioning (DP) system maintained position over the lo-cation.

The drilling casings required for hole stabilization when surficial uncemented sediments were expected to extend deeper than 2 m were provided in lengths of either 2 m or 4 m (see Fig 14).

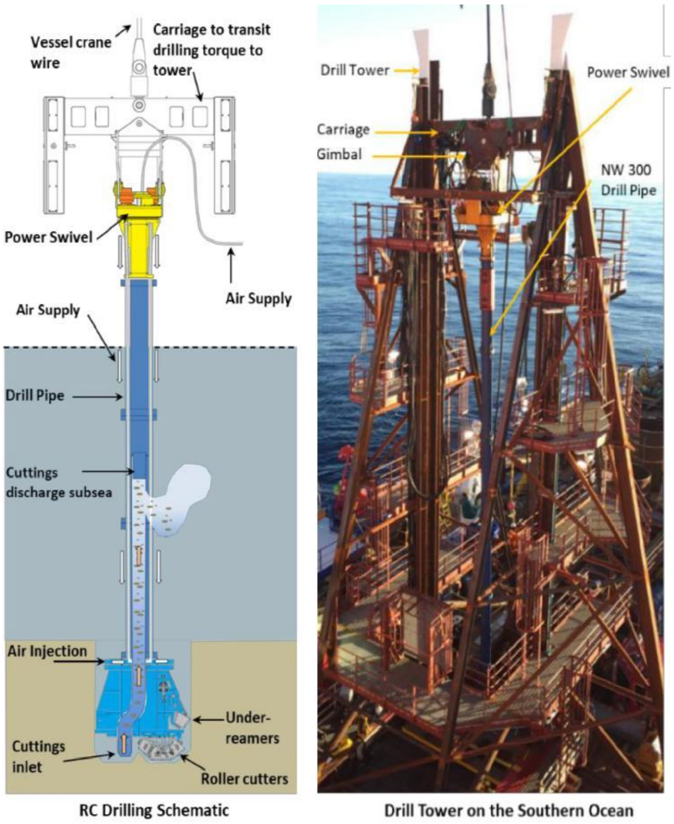

Figure 7. (a) Reverse Circulation drilling (b) Drill tower on the Southern Ocean.

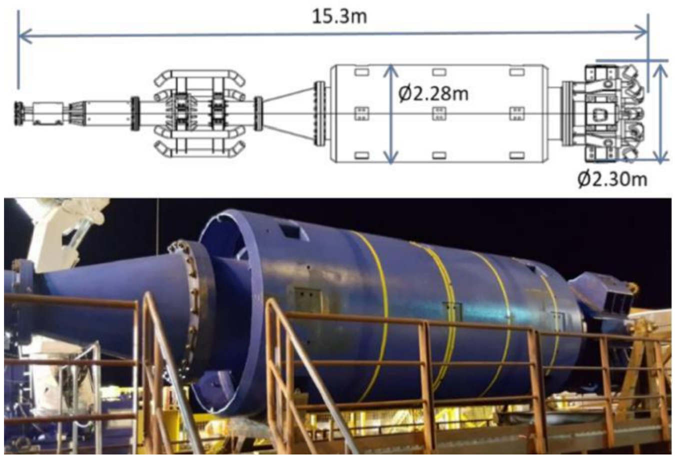

Figure 8. Bottom Hole Assembly – Drilled and Grouted Installation Method.

4.2.3 *Pile Guide Frame*

For the drive-drill-drive installation method, a 14 m x 14 m Pile Guide Frame (PGF) was used to maintain the vertical stability of the pile, hammer and follower during driving (see Fig. 9). The PGF was also fitted with custom designed gripping jacks to enable the pile to be held during relief drilling operations, thereby preventing the pile from running onto the BHA and trapping the drill string downhole. The PGF was fitted with two independent gripping systems to ensure 100% redundancy.

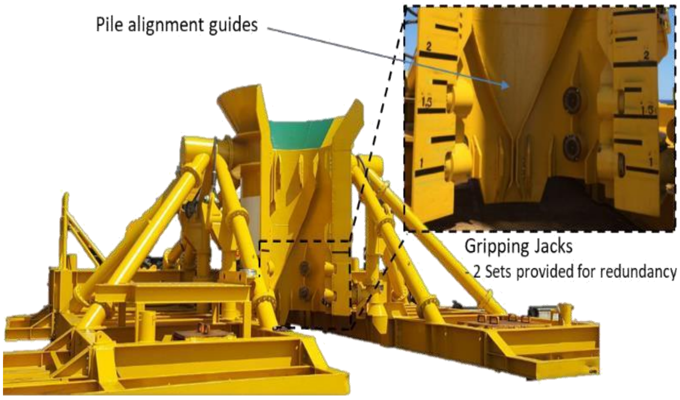

Figure 9. Pile Guide Frame with gripper jacks

4.2.4 *Pile Driving Spread*

A 280 kJ hydraulic subsea hammer was used to drive the anchor piles to depth for the drive-drill-drive installation method. A follower was employed to transmit the driving energy from the hammer to the pile. The hammer and follower are shown on Figure 10.

Although the grippers in the PGF were not able to prevent free-fall of a driven pile, as a contingency, the piles included protrusions that were designed to en-gage with purpose built mudmats to arrest any free-fall should that occur.

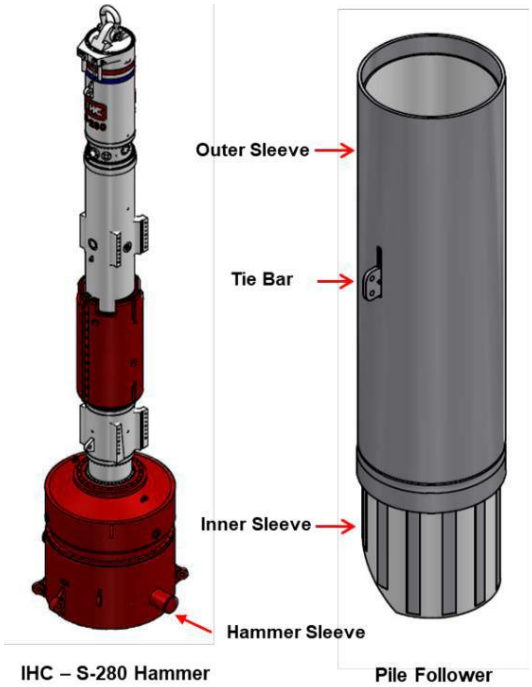

Figure 10. Pile hammer and follower

4.3 *Drilled and Grouted Piles*

4.3.1 *Site identification*

Prior to installation of the drilled and grouted piles, it was first necessary to identify the thickness of (any) surficial uncemented sand expected at each anchor location. Site-specific seismic refraction data was obtained at the anchor locations (see Fig. 11) prior to installation in an attempt to mitigate potential risks associated with collapse of uncemented sediments into the drilled hole. The colours shown on the seismic refraction profiles on Figure 11 represent the seismic velocity and this is used as an index for sediment strength and induration.

This data allowed the anchor locations to be adjusted, within the allowed installation tolerances, in order to target areas with less than 1.5 m of surficial uncemented sand. In areas where thicker layers of surficial sand could not be avoided, the depth of uncemented material indicated by the geophysical data assisted in determining the required length of casing to be used.

An additional installation challenge for the Project was the rugosity of the seabed. This was both in terms of the stability and levelness of the construction equipment and also the alignment of the chain connected to the anchors and the loads imposed on it and those transferred to the anchor. This aspect was effectively managed by targeting optimal positions for all the anchors, whilst considering chain alignment.

4.3.2 *Installation Process*

Fifteen of the anchor piles were installed using the drilled and grouted technique. Of these, drilling casings were required at three locations to prevent loose sand from running into the hole. Figure 12 depicts the drilled and grouted pile installation process for an anchor location that did not require a casing for stabilization of the surficial sand/ uncemented material.

4.3.3 *Drilling*

The drill tower and pile length were designed to enable drilling to be completed in one stroke of the drill tower, to prevent the need to add drill pipe while downhole. The maximum drilled hole depth during the project was 17.5 m.

The BHA and drill string were marked to enable measuring of drill depth subsea and a bullseye was used on the top stabiliser for checking the drill string verticality.

The initial collaring or initiation of the drilled hole typically took around 30 minutes but from there drilling generally progressed relatively quickly, at rates of up to approximately 2 m – 3 m per hour. The drill bit and BHA were designed to achieve drilled holes with an inclination of less than 1 degree to the vertical. Figure 13 presents a series of images showing the progression of the drilling process from initiation to completion of the hole.

For the three anchor sites where greater than 1.5 m of surficial sand was expected, drilling casings were employed. In each instance, an initial depression in the seabed was created by open hole drilling and then the casing was inserted when it was evident that run-ing of the surficial material was preventing the advancement of the hole. The casing generally progressed downward into the seabed with the continuation of drilling to approximately the point that the top

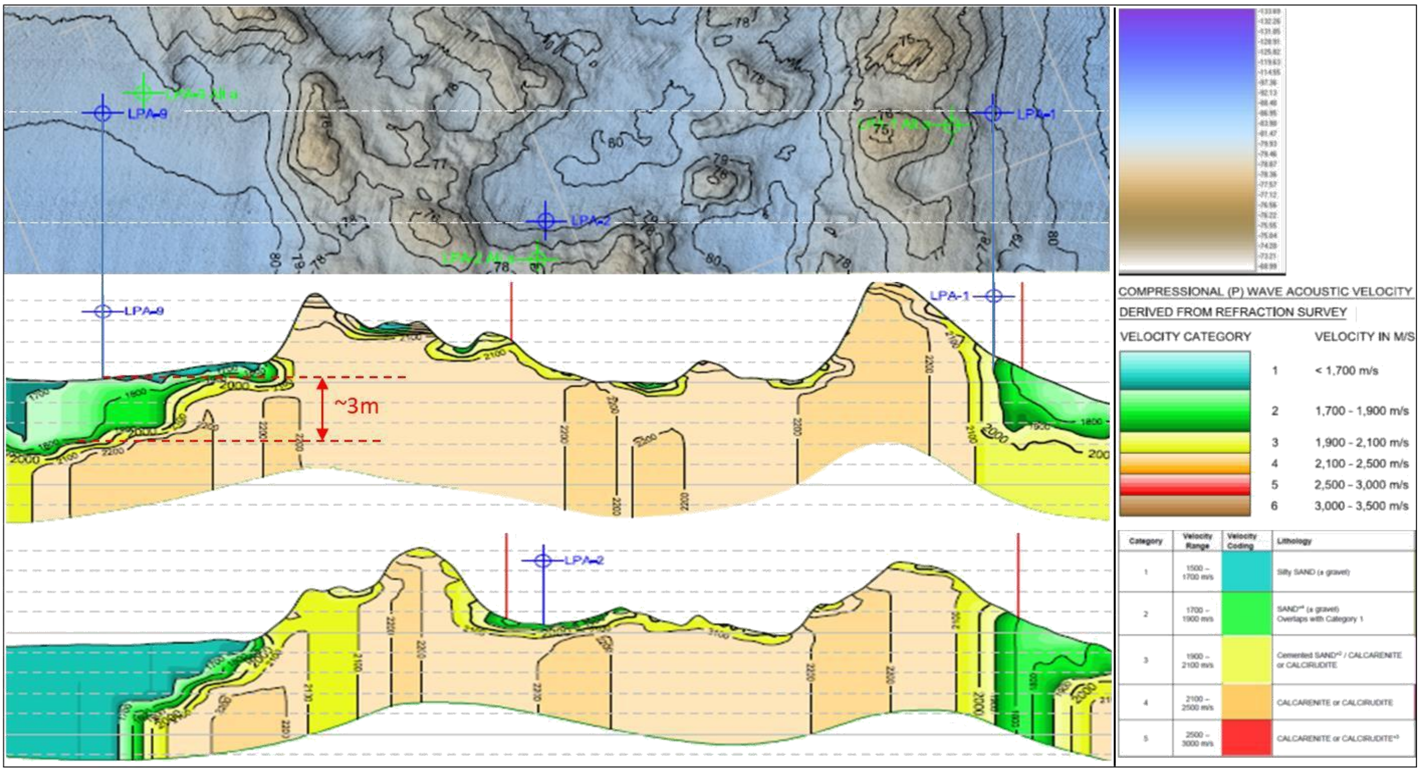

Figure 11. An example of Geophysical data used for installation planning. E.g. LPA-9 original location (in blue) was identified as having approximately 3m of uncemented material over rock.

of the casing was situated at or about mudline level. Where required, a clump weight was used to assist installation of the casing and to maintain casing verticality. Figure 14 presents images showing the drilling process at locations where a drilling casing was employed.

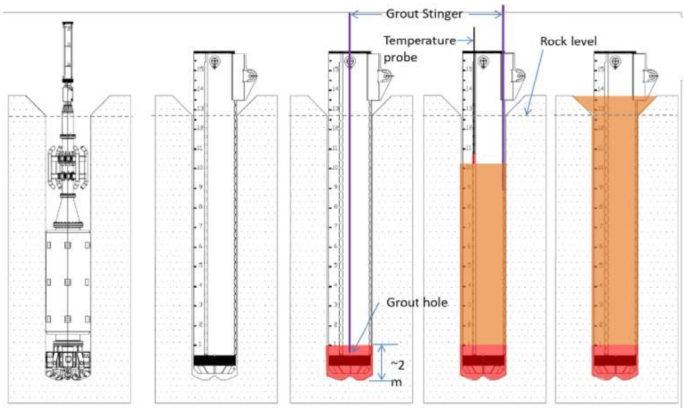

Figure 12. Drilled and grouted pile installation process.

Figure 13. Images showing progression of drilling process

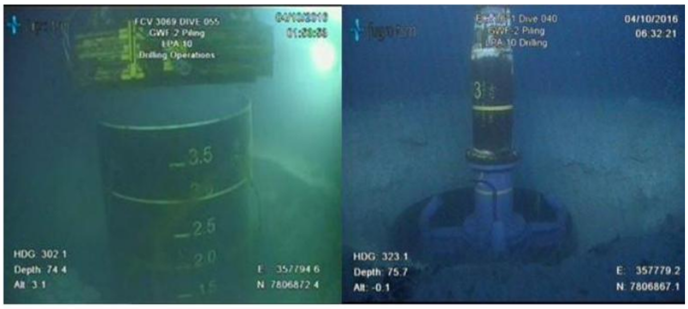

Figure 14. Drilling process incorporating drilling casing.

4.3.4 *Pile Installation and Grouting*

Following completion of the drilling, the piles were installed into the pre-drilled holes using the vessels crane (operated in Auto Tension mode). During pile insertion, an ROV was docked to the anchor padeye to assist with alignment and orientation of the anchors, as shown on Figure 15.

The drilled and grouted piles each featured three grout stinger tubes welded to the outside of the pile to facilitate grouting of the drilled hole-pile annulus. The piles were grouted using a 2-inch flexible hose downline and grout stinger made from 2-inch steel drill pipe.

Figure 15. Pile insertion into drilled hole

Figure 16. Grouted pile.

Two different grout mixes were used. First a grout plug was set at the bottom of each hole, and for this a higher density grout (SG of 1.75) was used with the objective of achieving a 1 MPa compressive strength within 8 hours to effectively seal the base of the pile.

The annulus was subsequently grouted with a low-density grout mix (SG of 1.67), which was designed to flow around the pile without restriction. Grout ad-ditives were also used with the annulus grout to re-duce the effect of bleed as the grout set.

Grouting was performed using a bottom up method with the outlet of the grout stinger submerged approximately 1 m into the grout during filling. The level of the grout was determined using a temperature probe deployed down a separate grout tube from the stinger. A photograph of a grouted pile is shown on Figure 16.

4.4 *Drive-Drill-Drive Piles*

4.4.1 *Installation Process*

The drive-drill-drive installation method, as depicted on Figure 17, was adopted at locations where more than 5 m of uncemented material was expected. Consequently, this method was employed at nine of the anchor locations, all of which were situated at Site B.

This method was used because it allowed the pile to essentially ‘self-case’ the loose material during self-weight penetration and initial driving, while also enabling driving through any underlying cemented layers. This method also had the added benefit of al-lowing drill string to be safely added with the BHA in the pile without the risk of heave motions of the BHA causing hole collapse.

4.4.2 *Initial Driving*

Each of the drive-drill-drive piles was installed through the PGF described in Section 4.2.3, to ensure vertical stability and to also align the pile heading. The PGF employed a gripping system to enable the pile to be held during relief drilling to prevent the pile running onto the BHA and trapping the drill string downhole.

Following landing of the PGF, the pile was in-serted through the PGF and allowed to penetrate un-der its own self weight. The hammer and follower were then positioned on top of the pile with driving commencing once any additional self-weight embed-ment had ceased.

In order to drill out below the pile tip, the BHA would first need to be lowered through the pile with-out getting stuck, which could occur in the event of an ‘extrusion buckling’ failure, such as occurred at Woodside’s Goodwyn A platform (Barbour and Er-brich, 1993; Senders et al, 2013). There was only very limited scope for any pile tip distortion during driving, since there was only a 20 mm wide annulus separating the inside of the pile and the BHA (assuming both were perfectly circular in plan).

It was therefore essential that the pile would not deform significantly during the initial driving stage and hence appropriate analyses were required to ad-dress this risk and to assist with decision making dur-ing installation. A series of analyses were therefore

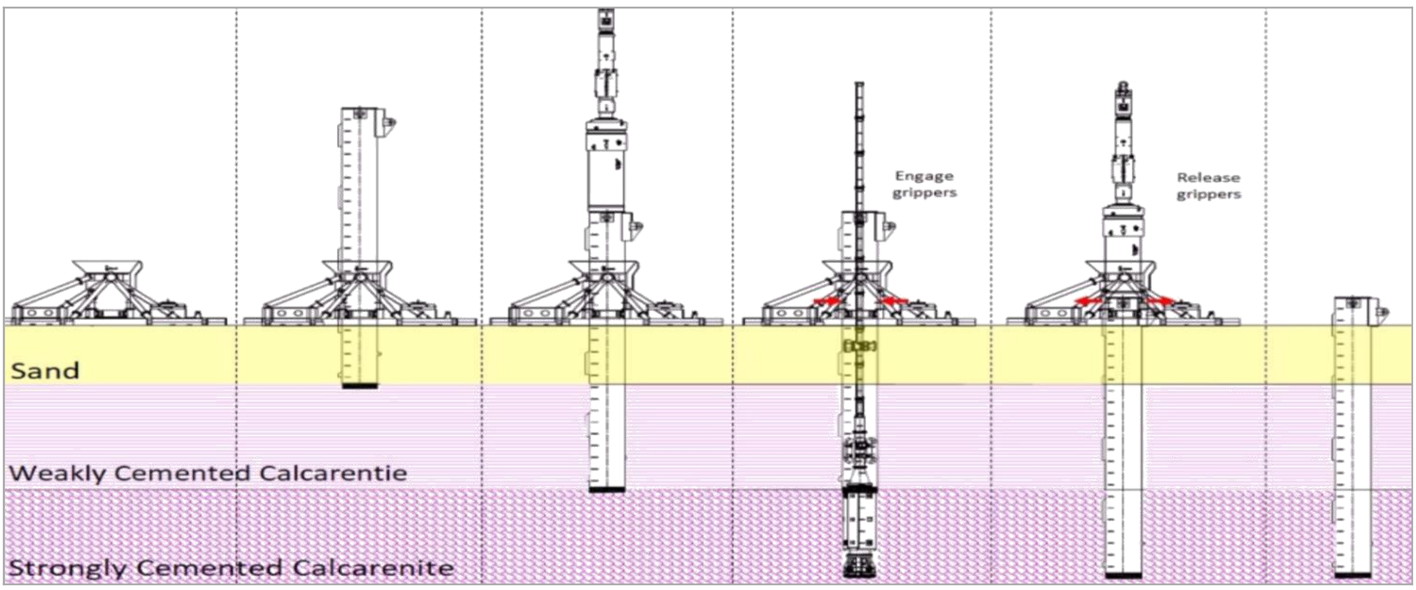

Figure 17. Drive-drill-drive installation process.

performed to assess the potential deformation of the pile tip during driving, using an algorithm known as BASIL; Erbrich et al. (2010). This explicitly models the extrusion buckling failure mechanism. The results of these analyses revealed the maximum number of hammer blows that could be applied to the piles while preventing deformation of the pile tip beyond the point where it would not be possible for the drilling BHA to fit through the bottom of the pile. This was used to define an effective 'refusal' criterion for the piles. Consequently, the initial driving phase was per-formed until either this criterion was met, or the pile achieved its target penetration depth.

The BASIL model was also extended to allow the tip deformation to be modelled during the secondary driving phase, once the hole had been successfully drilled below the tip. This modification included explicit modelling of inclined tip forces (due to asymmetric tip bearing failure as a result of the pre-drilled hole below) and also involved application of a limiting pile radius (the hole radius) at which soil pressures would be assumed to apply.

4.4.3 *Relief Drilling*

In circumstances where pile refusal occurred during initial driving, the hammer and follower were recovered and the gripping system on the PGF engaged to prevent additional movement of the pile. The relief drilling BHA was then deployed through the top of the pile using a guide cone as shown on Figure 18.

The BHA adopted for relief drilling was only capable of advancing a maximum of 7 m past the pile toe without significantly increasing the possibility of the BHA becoming trapped in the hole in the event of hole collapse. Hence, it was possible that multiple relief drilling passes may have been required to achieve the pile target penetration depth.

The BHA was fitted with under-reamers to enable the diameter of the hole in front of the pile toe to be increased to further reduce the driving resistance. The under-reamers could be deployed at two different settings, each with slightly different gauge diameters,

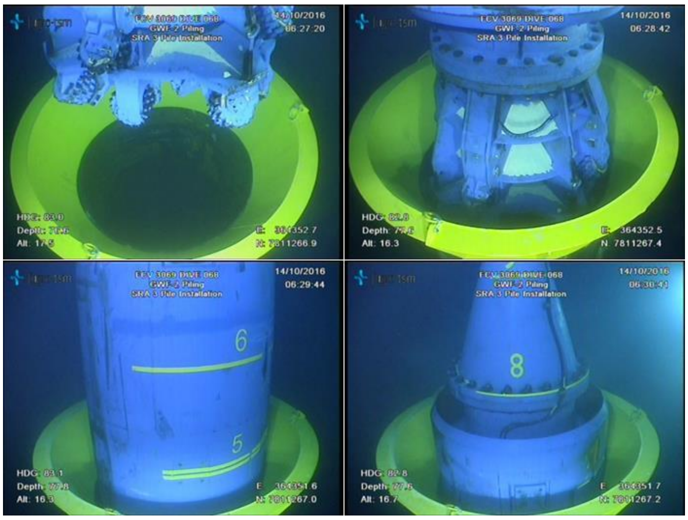

Figure 18. BHA assembly passing through guide cone.

depending on the ground conditions encountered. Use of the under-reamers was considered as a contingency only, if secondary driving following relief drilling proved unsuccessful.

4.4.4 *Secondary Driving*

Following completion of a relief drilling pass (up to a maximum of 7 m in front of pile toe), the BHA assembly and guide cone were recovered and the hammer and follower were repositioned on the pile. From there, additional pile driving was performed until the pile reached the target penetration depth or subsequent refusal occurred.

4.5 *Key Installation Observations*

Given the variability in the ground conditions and the contingencies provided for by the installation methods, an observational approach was adopted during installation of the piles to verify the predicted ground conditions and to aid with decision making for the installation of subsequent piles. Some of the key observations from the installation process are discussed here.

The driving resistance during initial driving was generally found to correlate well with the available geophysical data. Two examples of this are shown on Figure 19, where 'refusal' during initial driving occurred at or near a prominent reflector.

Relief drilling after initial refusal was found to reduce the driving resistance (total energy per quarter meter) to between approximately one third and one quarter of the resistance prior to drilling (see Fig. 19). Therefore, without the use of the drive-drill-drive technique, a very large hammer (and consequently heavier piles to accommodate the fatigue loading and risk of pile collapse) would have been required. This

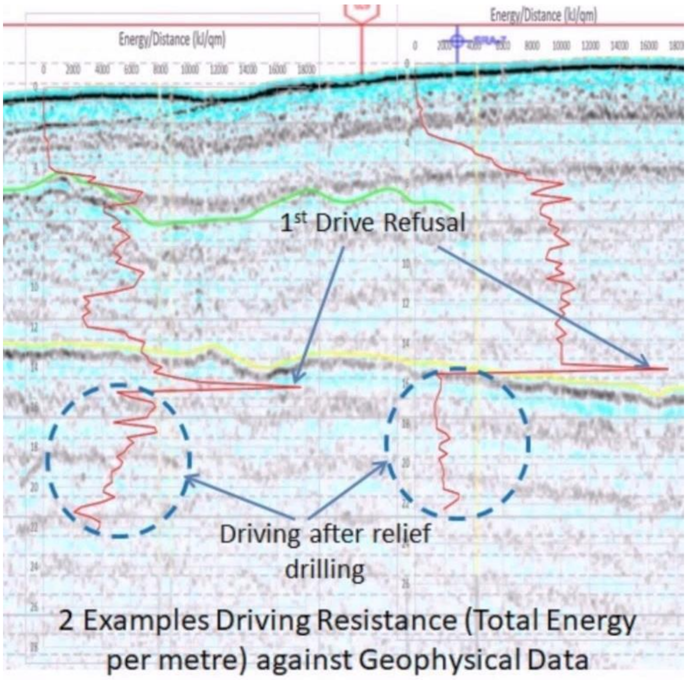

Figure 19. Examples of comparison of driving resistance with geophysical data.

would have required a much larger installation vessel and equipment and also resulted in greater, and possibly prohibitive, acoustic impact on the marine fauna. Hence, the drive-drill-drive technique provided a cost-effective installation solution that successfully mitigated the risks associated with premature pile refusal.

Generally, only a single relief drilling pass was re-quired to reduce the driving resistance to the point where the pile could be driven to its target depth. However, at two anchor locations, two relief drilling passes were successfully performed.

At the East and West anchor clusters at Site B, the presumed weakly cemented material demonstrated higher driving resistances than initially anticipated. However, at one location, degradation in the cementation of this material during relief drilling in advance of the pile toe caused some issues with hole collapse. This was mitigated at subsequent locations by ensuring that the pile was initially driven deeper (and thus to a higher 'refusal' criteria) during the initial driving phase prior to relief drilling. The assessment of the suitable depth for initial driving was aided by the available geophysical data.

Drilling rates in the underlying strongly cemented calcarenite at Site B (indicated by the yellow line on Fig. 19) were significantly slower than observed in the outcropping calcarenite at Site A. This reflects the expectation that the deeper calcarenite is likely to have been subjected to more sub-aerial exposure and is therefore predicted to be stronger.

4.6 *Verification of Ground Conditions*

Given the limited regional geotechnical data and a lack of site specific geotechnical data, installation observations were used to verify the ground conditions against those assumed in design. This was achieved in a number of ways, including:

- Review of drilling records (for both installation methods).
- Visual observations of the characteristics of the drilled hole (for the drilled and grouted method) – see Figure 20.
- Visual observations of the drilling debris at the dis-charge location and at the seabed.
- Review of pile driving records (for drive-drill-drive method).

The drilling debris was composed of fragments of calcarenite and limestone, which was formally assessed and considered to be benign to the environment, being of an identical mineralogy to the outcropping rock and uncemented sediments.

Figure 20. Visual observations of drilled hole.

5 CONCLUSIONS

The successful installation of 24 large anchor piles, which were up to 2.14 m in diameter and 28 m in length was achieved across a wide range of geologies, from high strength limestones to uncemented silts, and required the use of a strategic combination of driving and drilling techniques. Installation was carried out using a Fugro operated dynamically-positioned construction support vessel, on which an in-ventive purpose-built drilling tower had been erected. Some of the adopted drive-drill-drive installation methods had never previously been adopted in the region; in particular, the use of the vessel's tension-con-trolled crane to support the drill-string within a purpose-built derrick. This innovative process also included development of an ingenious seabed frame that could grip and release the piles progressively, through which drilling could be performed, plus drilling casings to stabilise the drilled holes.

The Project was successfully completed despite the limited availability of location-specific geotechnical data and insufficient time for conventional site investigation and laboratory testing. The acquisition of additional geophysical refraction data was adopted as a fit-for-purpose approach that allowed the Project team to successfully understand the incredibly variable seabed conditions which helped to optimise the pile design, develop pragmatic installation strategies and cross-check encountered seabed conditions with design assumptions.

An integrated multi-disciplinary team worked together to achieve the ultimate project goals of safely delivering secure mooring arrays for the MODU, avoiding any negative impact to the original well drilling plan and the environment. This project demonstrates how multidisciplinary collaboration can embrace innovative solutions to successfully deliver complex projects safely, on budget and to an accelerated schedule.

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