PILE DESIGN, INSTALLATION AND BACKANALYSIS OF 54 IN PILES IN GRAVELS AT THE WIRIAGAR DEEP PLATFORM IN BERAU BAY, INDONESIA

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ABSTRACT

The paper reports the design, installation, and back-analysis of six 54-inch driven steel piles in dense sands and gravels at the Wiriagar Deep A platform in Berau Bay, Papua Barat, Indonesia. Pile design required to incorporate the presence of thick gravel layers and the potential interaction of the piles with adjacent well conductors during installation or during drilling activities. Installation design required to consider the lack of industry recommendations for the derivation of Soil Resistance during Driving (SRD) in gravel and selection of fit for purpose piling equipment. Dedicated site investigations were carried out to acquire data that would enable pile capacity and drivability predictions. Geological analysis of the origin and distribution of the gravels provided additional context that was incorporated to the installation engineering studies. Pile design mitigation measures included setting the conductor tip below the proposed pile tip depth and strengthening the tip of the piles. Installation mitigations included full instrumentation and monitoring of the piles during driving and the mobilization of a pile top drilling spread. The pile driving instrumentation data enabled efficient management of the installation programme and limited the requirement to implement the installation mitigations available. A companion paper (Pua et al, 2020) provides a detailed description of the instrumentation programme for the Wiriagar Deep A platform adopted during installation. The data acquired also confirmed the uncertainties previously published for pile driving in gravels and the challenges to characterize their in-situ relative density and behaviour during driving. The pile driving monitoring data suggests degradation of shaft resistance during driving in a similar way to friction fatigue effects. The pile instrumentation data also enabled evaluation of the change of pile capacity with time as re-strikes were performed at selected time intervals and confirm the pile design suitability.

Keywords: piles, gravel, drivability, instrumentation, Tangguh

INTRODUCTION

Background

The Tangguh LNG facilities and associated gas producing offshore platforms are located in a remote site in Berau/Bintuni Bay, West Papua, Indonesia. The site is in an estuary characterised by strong tidal currents, active seabed transport and highly variable shallow soil conditions. In addition, Papua Barat is an area of active tectonics capable of generating significant earthquakes. Shallow soil conditions in Berau Bay typically comprise sands overlying or intercalated with very stiff overconsolidated marine clays with undrained shear strengths that do not increase significantly with depth. Multiple site investigations have been performed to characterise the area and a conceptual geological ground model has been developed covering large sections of the bay and the onshore LNG site. Due to the complexity and variability of the shallow soil conditions, site specific data has been acquired at every offshore site considered. Variable soil conditions and the challenges to achieve representative characterization and design parameters contribute greatly to the challenges of this project.

The Wiriagar Deep A wellhead Platform is a normally unattended installation (NUI) located approximately 30km offshore from Tangguh LNG in Berau/Bintuni Bay. The platform was installed in August 2018 as part of the Tangguh Expansion Project (TEP) that will increase the capacity of the existing Tangguh LNG facility operated by BP Berau on behalf of its partners.

The layout of the existing offshore facilities of TEP including a 3D representation of the WDA structure is shown in Figure 1 that also includes the position of the two new platforms, Roabiba A (ROA) and Wiriagar Deep A (WDA), associated pipelines from WDA to ROA and ROA to the ORF.

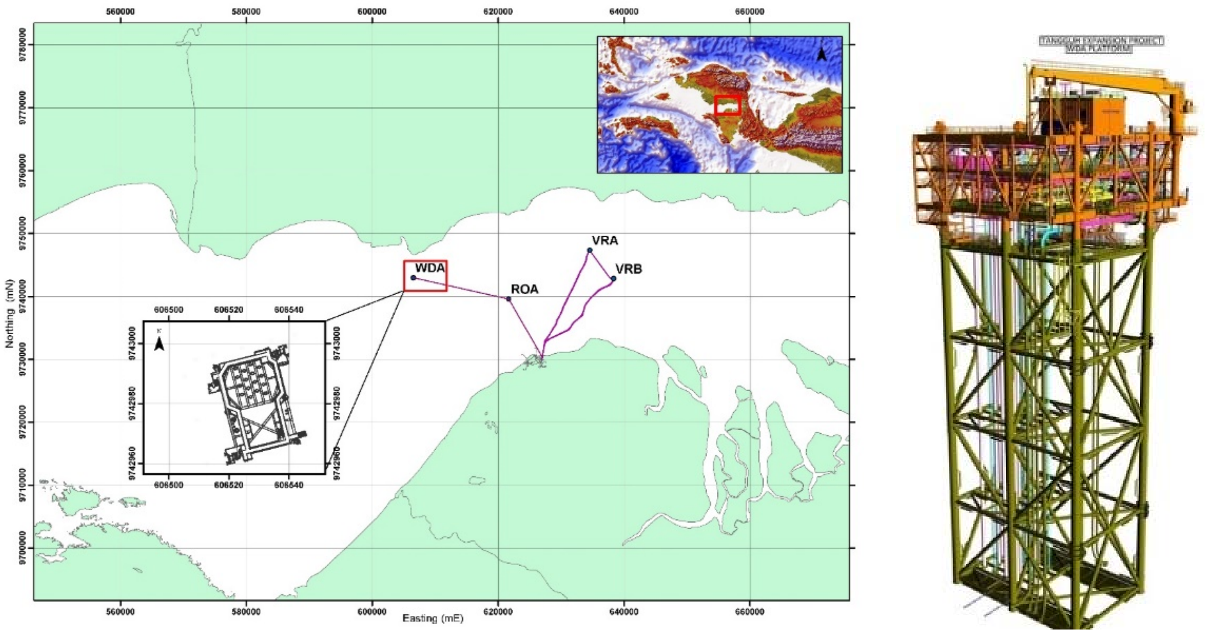

Figure 1 Location of the Wiriagar Deep A and Tangguh LNG Development and 3-D model of the Wiriagar Deep A Platform (approximate dimensions, including topsides: 95m by 30m by 20m, water depth approximately 60 m)

This paper presents an overview of some of the challenges faced to design, install and assure the capacity of large diameter open ended piles driven through the sand and gravel layers present at the Wiriagar Deep site.

The Wiriagar Deep A Fixed Structure

The WDA platform was designed as a six vertical legs steel jacket of approximate weight 2,239t supporting an approximate topside weight of 2,062t. The structure was installed in August 2018 in a water depth of 60m (Lowest Astronomical Tide, LAT) with 14 well conductors also pre-installed in preparation for drilling activities.

Pile and Conductor Details:

Six nos. 1.372m (54in) 70-75mm wall thickness API 2W Gr.60 414MPa Yield Strength outer diameter open-ended vertical steel pipe leg piles were installed to about 72.5m penetration. Final penetration was slightly beyond the target penetration of 72m due to pile installation optimisation considerations. The piles were designed in 3 sections of different lengths welded in-situ with a constant wall thickness of 70mm and a 2m long 75mm increased section at the driving shoe. Fourteen nos. 0.762m OD, 38.1mm wall thickness, API 2W Gr.60 414MPa Yield Strength vertical steel conductors were also installed to 79m penetration. The conductors were driven in 5 different sections with welded in-situ connections. The closest proximity of conductor to pile is a perimeter-to-perimeter distance of 5.21m, with adjacent conductors approximately 3 to 4m apart.

SITE CHARACTERISATION

Geological Setting

The WDA platform site is in the western edge Bintuni-Berau Bay in the Bird's Head region of Papua Barat and lies within a region known as the Bomberai Plains as described by Robinson et al, 1990. The shallow stratigraphy is composed by the New Guinea Limestone Group of Palaeocene to late Miocene age sequentially overlain by the late Miocene to Pleistocene Steenkool Formation and Quaternary alluvial deposits. Sea level changes during the Pliocene to Pleistocene transition generated an area of large hills with large river systems draining into an expansive flood plain system. The higher energy of the river systems resulted in areas of fluvial erosion and deposition of land sediments, typically mixtures of sands and gravels in the banks of those channels. Subsequent episodes of sea level rising resulted in a sea transgression with the river systems submerged by the rising waters creating the present Berau-Bintuni Bay estuary. Previously incised areas with land sediments were overlaid by finer sediments of lower depositional environment. Since the mid-late Quaternary, a dynamic coastal/deltaic environment formed responding to changes in fluvial erosion and sea-level rise. Tidal action has shaped the current day seabed geomorphology of the area and the distribution of shallow Quaternary sediments, typically sands, silts and recent clays.

Gravel layers geological characterization

A desktop geological characterization study was performed to try to determine the origin and characteristics of the gravel deposits present at the Wiriagar site. The aim of this study was to determine the probability of encountering large cobble or boulder size materials in the gravel layers that could have a significant effect in pile installation operations.

As described in detail in AG (2013), the estimated age of the deposition of the gravel layers of interest could not be determined accurately. Only few assumptions could be made based on inference from the applicable sea-level curve. The bottom of the third gravelly horizon corresponds to an elevation of -120.4m to -121.7m LAT, which coincidentally correlates relatively well with the sea-level during the last glacial maximum approximately 20,000 years ago (about 120m below actual sea level). However, the presence of a 30m thick over-consolidated clayey deposit overlying the gravelly horizons, and additional information from nearby Vorwata A location question this hypothesis. Carbon dating was carried out at the nearby Vorwata A location. This dating confirmed an age of 41,000 years before present proving that at Vorwata A the upper clay layer within the "channel" predates the last sea level low. Based on this, it was estimated that the studied gravelly horizons could have been deposited during the penultimate glacial maximum between 145,000 and 125,000 years ago. However, there is not enough evidence to constrain this age and the studied gravelly deposits could be even older. The ages of the studied deposits were determined to be likely of Quaternary age, but the particles may be reworked lithoclasts of up to 500Ma years old rocks of the Kemum Block further to the north of Berau Bay.

The slope and the material recovered from the cores and analogues from other current rivers in West Papua indicate that bed-load transport for the gravelly material by was performed by braided river and the material was deposited in a coastal braid delta environment. Typically, the gravel horizons in this environment may extend laterally from a few dozen meters up to a couple of kilometres as confirmed by the interpretation of geophysical data available in the area. This study confirmed that based on the back-calculated slope angles gravelly horizons would have been deposited in <0.5m deep braided rivers discarding therefore the possibility to encounter large cobbles or boulder that could pose a risk to pile driving operations.Figure 1 shows examples of gravel sample recovered from two boreholes at the WDA site.

Figure 2 Photograph of sample from borehole WDA-2A-S at 39.45m below seafloor annotated with possible rock types and mineralogy, and sample from WDA-9-CS at 60.5m. (AG, 2013)

Geotechnical site investigations

Offshore geotechnical site investigations at the WDA Platform site were performed from a dedicated geotechnical drilling vessel in two separate campaigns. Initial fieldwork started in September 2011, and a supplementary investigation followed in September 2013 that provided additional data in the sand and gravel intervals. In total, nine soil investigation locations relevant to the platform piles were acquired at this site. Fieldwork comprised geotechnical borehole sampling, cone penetration tests (CPTs) (with 10cm^2 and high capacity 5cm^2 cones), seismic cone tests and in-situ dissipation tests. Subsequently, advanced static and dynamic laboratory analysis was performed in several geotechnical laboratories. Figure 3 shows an overview of the test locations in the context of the platform legs and pile locations.

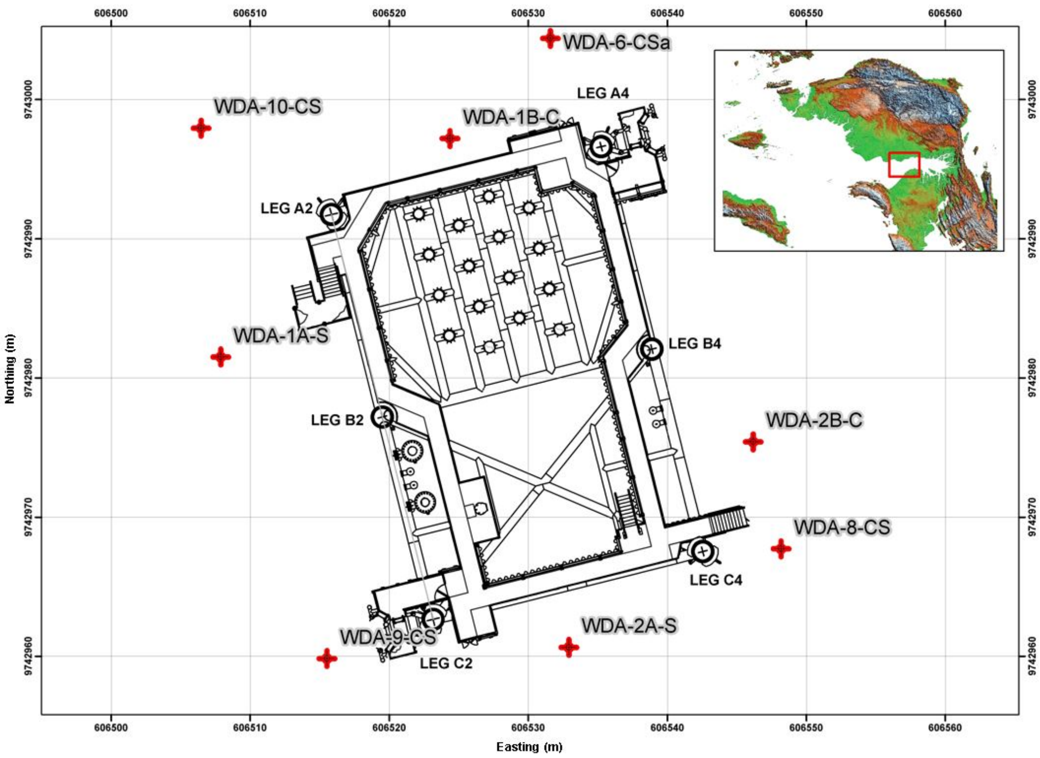

Figure 3 WDA Platform Plan View and Sample / PCPT Borehole Locations

Table 1 provides a summary of the site-specific geotechnical investigation performed for the design of the WDA platform's foundations.

Table 1 Summary of Site Investigation at WDA platform site

Year of Site Investigation	Borehole	Penetration Depth Below Mudline (m)	Data Acquired (-)	Notes
2011	WDA-1A-S	76.4	47 PIS, 9 Hammer and 11 WIP	
	WDA-1B-C	77.4	43 PCPTs, 5 SCPTs and 5 DSSs	Seismic Cone and Dissipation
	WDA-2A-S	145.7	31 PIS, 14 Hammer and 58 Samples	
	WDA-2B-C	146.0	66 PCPTs and 8 DSSs	Dissipation
2013	WDA-6-CS	2.8	1 PCPT	
	WDA-6-CSa	85.5	30 PIS, 47 PCPTs, 11 WIP	5cm2 PCPT
	WDA-8-CS	85.0	30 PIS, 40 PCPTs, 15 WIP	5cm2 PCPT
	WDA-9-CS	85.0	36 PCPTs, 15 WIP	5cm2 PCPT
	WDA-10-CS	65.0	30 PCPTs, 1 WIP	5cm2 PCPT

Note: PIS: Piston Sampler, WIP: WIP sampler, SCPT: seismic PCPT, DSS: cone dissipation test

Design Stratigraphy and Engineering Design Soil Parameters

The shallow stratigraphy at the WDA platform location can be summarized as a dense surficial sand layer to 0.7-2.2m below seabed underlain by firm to very stiff overconsolidated clay to 33m below seabed, followed by alternating layers of dense sands, dense gravels and hard clays. Table 3 details the interpreted soil stratigraphy adopted for foundation design at the WDA platform location. The main geotechnical design parameters plots including design profiles are shown in Figure 4.

Table 2 Interpreted soil stratigraphy and design parameters– WDA Platform Location

Layer	Depth (m)		Description (-)
	Top	Bottom	
1	0.0	1.0	Surficial loose calcareous SAND with some loose fine gravel
2	1.0	34.0	Stiff to very stiff CLAY with black peat lenses
3	34.0	38.2	Dense to very dense fine SAND
4	38.2	40.0	Very dense fine SAND with gravel
5	40.0	43.0	Hard CLAY
6	43.0	48.2	Dense SAND and GRAVEL with clay lenses
7	48.2	50.0	DENSE fine GRAVEL
8	50.0	52.0	Hard CLAY
9	52.0	61.0	GRAVEL with fine sand and clay lenses, (clean at intervals)
10	61.0	63.1	Hard CLAY
11	63.1	65.9	Dense to very dense silty fine SAND
12	65.9	66.9	Interlayered silty SAND and very hard CLAY
13	66.9	70.0	Dense sand
14	70.0	74.9	Sandy gravel
15	74.9	79.8	Interlayered sands and clays
16	79.8	84.0	Dense SAND
17	84.0	85.5	Hard CLAY

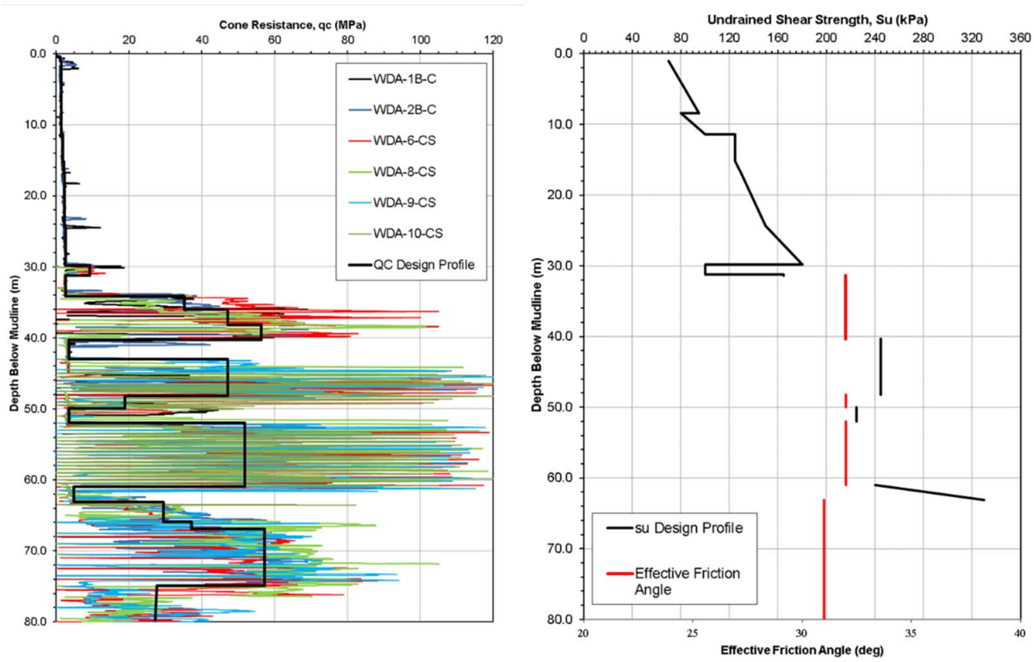

Figure 4 WDA Design cone resistance, undrained shear strength and effective friction angle

PILE DESIGN AND DRIVEABILITY ANALYSIS

Pile Axial Capacity and Driveability Design Basis

Given the uncertainties on the axial capacity response of open-ended steel piles in gravel materials a review of published case histories was performed. Jeanjean et al (2015), analysed PDA instrumentation and CAPWAP results of open-ended piles driven in gravel demonstrating that shaft friction and end bearing capacity of gravel layers can be lower than calculated by adopting current API RP 2GEO guidance for granular soils. They concluded that limitations in site investigation techniques do not allow to adequately characterize the in-situ relative density of this material and recommended to treat them as equivalent very loose sand layers. This conclusion is similar to other case records for similar diameter open ended piles in gravels as described in Fugro-McClelland. (1992), Schneider et al., (2007).

As per the project’s structural Basis of Design, piled foundations were designed in accordance with API RP2GEO recommendations incorporating the CPT methods for sands detailed in the commentary section. Initial studies evaluated the opportunity of optimizing the pile lengths by tipping the piles in Layer 9, interpreted as a very dense gravel. The proximity of the conductors to the piles and potential disturbance along the pile shaft or below the pile toe was also considered. The nearest conductor offset (OD 762mm) to a leg pile is 6.8m (centre to centre). As this distance is greater than eight (8) OD and in accordance with API 2GEO, no conductor group effect for loads and deflections were considered for foundation design interaction. However, the effect of drilling activities beyond the conductor shoe on pile axial capacities were studied in detail. These studies indicated that if certain potential conditions developed during drilling, mitigations would be required to avoid or at least minimize any effect to the piled foundations.

As the WDA platform and topside structural design further developed, pile head reactions were refined resulting in a net increase of axial and lateral pile loads. The overall foundation and conductor design philosophy was re-evaluated and the option of piles tipping in the gravel interval was discarded. The estimated axial capacity of a pile in tipping in gravel was marginal and the uncertainties in the operative axial capacity could not be reduced. The final API RP2GEO ultimate axial pile design capacities and the pile/conductor lengths are shown in

Table 3. Given the residual risk associated to the effects of well drilling beyond conductors on the shaft friction and end bearing of the piles, it was decided to set the conductor tip at least 7m (approximately 5 OD) below the tips of foundation piles in an interbedded hard clay and dene sand layer.

Table 3 WDA Platform design pile axial capacities and design pile/conductor lengths

Design Condition	Pile Length (m)	Compression (kN)	Tension (MN)	(Conductor Length) (m)
API RP 2GEO Axial Capacity	72.0	47.4	28.2	79.0

Once pile and conductor lengths were confirmed, studies were performed to assess their installation in the field. The total effort required to drive an open ended pile comprises the combined effects of the soil resistance to driving (SRD) (evaluated as the sum of the static pile tip resistance and total shaft friction along the pile), increases in pile resistance due to viscous rate effects, and dynamic increases due to inertia (Byrne et al., 2011).

In order to understand variability and sensitivity to the SRD prediction method adopted, a comparison of SRD profiles by different authors was performed. The methods compared were Toolan & Fox (1977), modified Stevens et al. (1982), Alm & Hamre (2001) and Schneider & Harmon (2010). The drivability analyses that followed evaluated several hammer sizes, scenarios of limited hammer efficiency and delayed restarts. The results of the drivability analyses concluded that two hammers were suitable to perform the installation of these piles; Menck's MHU-800 and MHU-1700. Given the residual uncertainties in installation in gravels and the probability of encountering obstructions it was also decided to mobilise a large diameter drilling spread in case soil plug removal was required to achieve target pile penetration. The implementation of a drive-drill approach was subject to the results of the pile instrumentation during field operations.

PILE INSTALLATION AND MONITORING

Monitoring instrumentation

Due to uncertainties associated with the installation of large diameter piles through thick gravel layers, continuous pile monitoring was implemented in all pile and conductor segments. The objectives of the instrumentation campaign were:

- De-risking early refusal in gravel layers
- Optimization of the installation sequence
- Determination of soil-plug removal was required
- Verification of axial pile capacity, short and long term through restrikes
- Assurance on pile tip integrity
- Assurance on installation hammer performance

Pile monitoring was carried out by attaching instruments near the pile top to measure strain and acceleration during hammer impact. A detailed description of the instrumentation programme is provided in the companion paper (Pua et al, 2020).

Pile installation behaviour

The initial self-weight penetration of the piles during installation with the internal lifting tool ranged between 10 m to 23 m. All the piles further penetrated when the MHU1700 hammer was placed on the pile top. Final self-weight penetration ranged between 22 m to 29.0 m. The average self-weight pile penetrations with internal lifting tool and hammer is presented on Figure 5.

The instrumentation data acquired indicated uniform driving behaviour of piles and conductors with limited deviation from average values of blowcount, maximum driving stress and back calculated SRD. Simplified CASE Method approach (RMX with Jc= 0.5) allowed continuous comparison of the predicted SRD with the back calculated value from driving data. This comparison identified that the overall driving resistance was slightly overestimated during design. Back calculated SRDs values were generally close lower bound predictions, except for pile C4. The record of blowcount/interval and hammer energy in Figure 5 show a general overestimation against predictions. In particular, the blowcount in the gravel layer between 51 and 62m was lower than expected for the energy assumed in the initial drivability analyses.

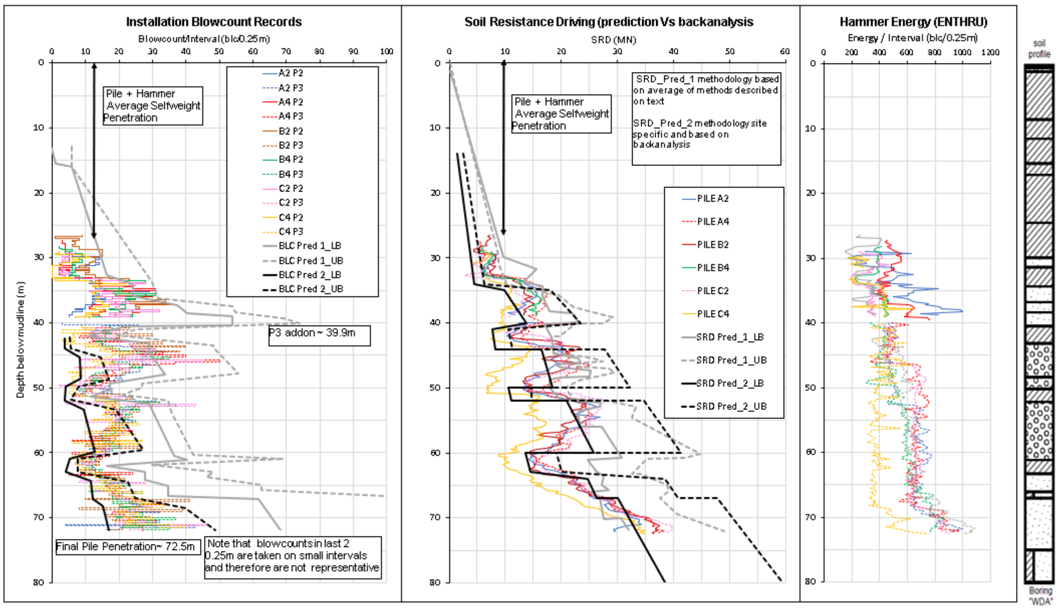

Figure 5 Composite plot of SRD back-calculation, blowcount and hammer energy for all piles. See annotations for SRD methodologies adopted

Soil plug measurements for all piles are presented in table confirming the expected unplugged behaviour of the piles during driving, although some partial plugging was observed in pile C4.

Table 4 Pile measured plug elevations

Leg	Measured, L, m	H, m	Soil plug, m	Elevation corresponding to mudline
A2	69.60	70.01	-0.41	0.41 m above mudline
B2	76.30	70.01	6.29	6.29 m below mudline
C2	68.20	70.01	-1.81	1.81 m above mudline
A4	75.40	70.01	5.39	5.39 m below mudline
B4	71.75	70.01	1.74	1.74 m below mudline
C4	82.30	70.01	12.29	12.29 m below mudline

Note Soil plug distance negative indicates soil plug elevation above the mudline and positive indicates soil plug elevation below mudline

Driving behaviour in gravel layers

Installation records and instrumentation data in Figure 5 indicate that driving resistance in the gravel layers can be characterized by the following behaviours;

- Initial driving resistance was higher than expected
- Driving resistance during continuous driving was lower than calculated by industry accepted methodologies for granular soil
- It was observed a decrease of driving resistance with increasing penetration. Assuming that the end bearing capacity should remain relatively constant with depth, the reduction of resistance with depth can be attributed to loosening due to driving induced vibrations or “equivalent driving fatigue” due to reducing shaft friction with penetration as described in Alm & Hamre (2001)

Pile set-up

A series of re-strike tests were conducted on several piles on intervals of 30 hour and 30 days after initial driving. In the first restrike test carried out 30hrs after end of driving, the pile penetrated about 10 cm after about 20 blows with average measured ENTHRU of about 800 kJ. A second restrike test was carried out on the same pile (A2) after a delay of nearly 30 days. The hammer was set at full rated energy with an average measured ENTHRU of about 1,400 kJ. The pile penetrated about 50 cm after about 67 blows. The calculated rate of increase in of capacity with time as detailed in Figure 6 was comparable with other data from other open-ended piles of similar diameter driven in Berau Bintuni Bay.

Dynamically measured vs predicted pile capacity

All the foundation piles were installed to a final penetration of about 72.5 m, slightly deeper than their design penetration. The back analysis of instrumentation data analysed by specialist consultants and reviewed independently indicated that as expected, the piles are acting plugged in the long-term static loading condition. The CAPWAP analyses from the restrike test conducted after a delay of about 30 days gave a soil resistance of 48 MN as shown in Figure 6. This interpreted CAPWAP dynamic soil resistance compared favourably with the design ultimate axial pile capacity of 47 MN based on API RP 2GEO recommendations. The dynamic pile capacity measurement and other considerations were used to assess the adequacy of the foundations.

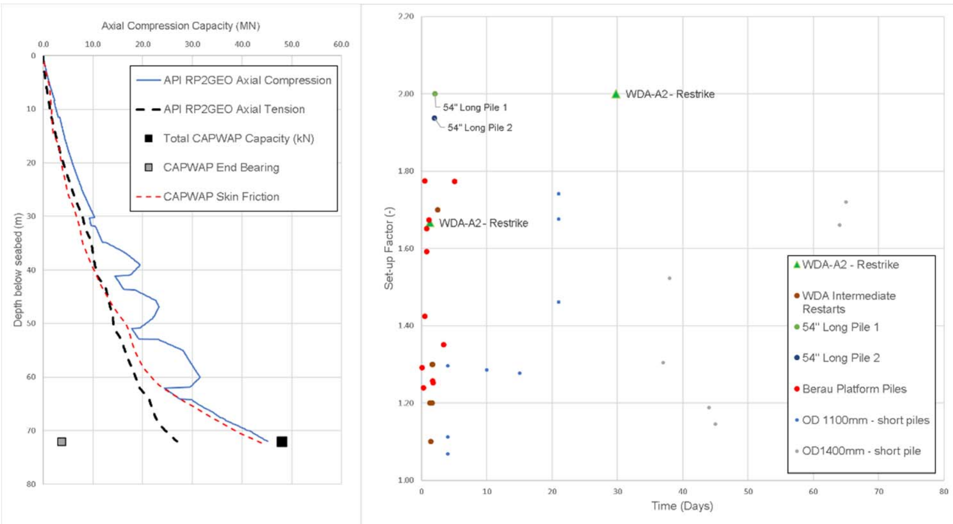

Figure 6 Comparison of ultimate compression capacity prediction from API RP 2GEO and CAPWAP estimated capacity from 30-day re-strike and calculated rates of set-up for piles driven in Berau/Bintuni

CONCLUSIONS

This paper provides a case record in an area of the industry where experience is still limited. The paper confirmed the uncertainties in characterizing the in-situ relative density of gravel can lead to significant reservations at the time of carrying out pile axial capacity and installation design. By implementation of mitigations, the risks associated with these uncertainties can be effectively mitigated. The data collected during installation allowed for significant optimization of the installation programme. It also provided additional assurance and verification of the pile design adopted, despite the complexities of the soil conditions. These learnings support the de-risking of platform location selection in other locations in Berau Bay.

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