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Mitigating the risk of ageing piling equipment and foreign migrant work force by full scale pile testing in Cabinda, Angola

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ABSTRACT

West Africa is a focal area for new oil and gas developments in Africa. Apart from offshore infrastructure there is also some significant pressure to develop infrastructure to support the growing number of personnel and plant required for the oil and gas industry. One such hot spot is the town of Cabinda in northern Angola. A significant risk to developing infrastructure in Cabinda is the availability of plant, skilled work force and achieving quality of construction. At one such development the design required the installation of 600 mm diameter bored piles installed to a depth of 15.5 m to provide the foundation of condominiums for oil and gas industry staff. In the design piles were assumed to carry load in end-bearing and side-friction. Piles were installed using a Chinese manufactured Jintai GPS-15 rig, using migrant labour (meaning imported labour from outside Africa, who are not necessarily remaining in Africa and may not be skilled for the particular task at hand). To test the repeatability of pile installation and the likely pile capacities to be achieved in the saturated marine sand and alluvial clay profile, four (4 No.) full-scale pile tests were conducted and loaded to failure. Piles were instrumented at the pile head and results were back-analysed to verify design assumptions of bearing mechanism and safe load bearing capacity. This paper describes the full-scale testing undertaken and the process and results of the back-analyses. The paper also provides a basis of expectation of pile capacity in Cabinda using locally available equipment and staff.

Keywords: Pile load testing, bored piles, Cabinda, Angola

1 INTRODUCTION

West Africa is a focal area for new oil and gas developments in Africa. Apart from offshore infrastructure there is also some significant pressure to develop infrastructure to support the growing number of personnel and plant required for the oil and gas industry. One such hot spot is the town of Cabinda in Northern Angola. A significant risk to developing infrastructure in Cabinda is the availability of plant, skilled work force and achieving quality of construction. At one such development the design requires the installation of 600 mm diameter bored piles installed to a depth of 15.5 m to provide the foundation of three-storey, reinforced concrete frame condominiums for oil and gas industry staff. To test the repeatability of pile installation and the likely pile capacities to be achieved in the saturated marine sand and alluvial clay profile, four (4 No.) full-scale pile tests were conducted and loaded to failure. This paper describes the full-scale testing undertaken and the process and result of the back-analyses. The paper also provides a basis of expectation of pile capacity in Cabinda using locally available equipment and staff.

2 THE SITE

The site is located next to the ocean outside of the town of Cabinda and is covered by recent beach deposits and a small wetland. The site was investigated in 2012 using 20 No. boreholes with a hollow auger rig and Standard Penetration Testing (SPT) conducted at 1.5 m depth intervals (refusal taken as 60 blows/300 mm penetration). At the time of the pile testing the site had been cleared of most of its vegetation, with only a central clump of palm trees, scattered indigenous trees and short grass remaining. The recent beach sandy deposits vary between fine clayey sand to coarse sands, typically occurring as loose sand at surface, steadily becoming denser with depth. Medium dense and dense conditions typically occur below +1.9 m elevation.

Significant variation was observed in consistency across the boreholes (Figure 1). The “softest” ground conditions relative to other areas on site appeared to be in the wetland zone. From the SPT data it is possible to identify a soft zone located between depths of -1.5 m and -4.0 m. It was expected that this soft zone could be problematic during pile installation resulting in pile borehole collapse. The wetland area is characterised by standing water and is covered by approximately 2 m of high plasticity, organic sandy clay. The groundwater level across the site is located at approximately 0.5 m below ground surface. Notwithstanding the particular wetland area, it is not practical to accurately map varying zones of consistency to specific locations on site. For this reason it was decided to evaluate the data across the site as one data population.

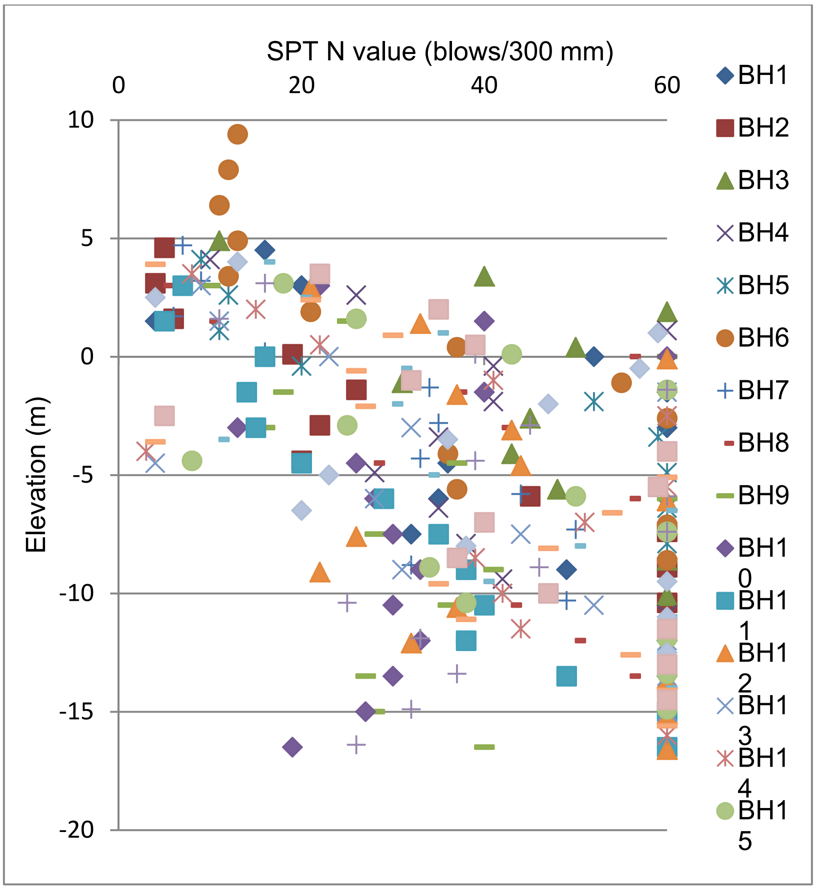

Figure 1. SPT data indicating variability in consistency

2. THE FOUNDING SOLUTION: BORED PILING

Since the initial geotechnical reports in 2012 it was clear that some kind of deep foundation was needed to support the three-storey concrete frame structures. Bored piling is often used in the Cabinda area and was selected as a suitable means of foundation. The geotechnical consultant at the time indicated pile capacities of up to 1700 kN/pile. This was translated into the structural design to coincide with a vertical working load requirement for design of up to 1700 kN per pile, associated with an estimated ultimate vertical load capacity of 4500 kN. The design anticipated 1200 No. of 600 mm diameter reinforced concrete piles installed to a depth of 16 m below pile cap level. Top of pile (T.o.P.) level is at +5 m elevation.

Very often in Angola the plant available for pile installation is old and derelict. At this particular site in Cabinda it was no different. The plant proposed to be used comprised of a derelict Jintai GPS-15 piling rig. The rig is manually operated, with the locus of control for pile installation residing entirely with the operator. The piling rig and drilling tool are shown on Figure 2. There was a very real concern about the state of the plant, the competence of the work force, the repeatability of pile installation and the reliability of the pile design in relation to how the pile could be installed. On this basis it was decided to conduct a series of full-scale pile load tests to verify the design and installation procedures.

Figure 2. Piling rig (Jintai GPS-15) and drill bit used for test pile installation

3 TEST PILE INSTALLATION

The test pile installation was done by the Chinese piling contractor using the plant he intended to use on site. The instrumentation and pile head monitoring was conducted by a third party Chinese contractor. A total of four (4No.) test piles were installed across the approximately 400 m x 200 m site.

The full-scale load testing was done to ASTM standards (ASTM D1143-81, 1994: Standard test method for piles under static axial compression load). The objective of the pile load testing was to load 600 mm diameter, 16 m long single piles in compression to a maximum load of 4500 kN or failure (whichever occurs first). The test piles were installed as bored piles using the same equipment planned to be used for construction. The pile testing equipment utilises a very old set of mechanical equipment with no electronic checks. The piling operator had total autonomy and control as to how the piles were constructed.

The pile installation process is summarised as follows: (1) A 600 mm diameter borehole is drilled under bentonite slurry using a temporary casing to a depth of 16 m below pile cut-off-level. It was the intention to clean the bentonite as it is circulated, but this process was not conducted to any satisfactory standard at the time of the test pile installation. (2) Upon reaching the desired depth, the drill is lifted out of the borehole and the reinforcement cage was placed into the hole using a crane and manual labour. The reinforcement cage is fitted with a permanent 600 mm outer diameter steel casing (approximately 0.5 m long) to protect the upper portion of the pile and to provide a firm loading area for the test. This arrangement is only for the test piles and was not carried through to the permanent piling works. (3) After centralising the reinforcement cage, a concrete funnel is assembled. This comprises a steel pipe section that is fitted to a funnel-shaped element where concrete is poured into. The pipe and funnel system is lowered into the hole before concrete is poured. Concrete is now poured into the funnel, expelling the bentonite in the borehole and creating the pile. (4) The pile is allowed to cure for a minimum of 28 days and the concrete cube strength values measured are shown in Table 1.

Table 1 Test pile concrete cube strength (laboratory Chiazi, Cabinda, Angola)

Test Pile Number	Minimum Cube Strength (MPa)	Maximum Cube Strength (MPa)	Average Cube Strength (MPa)
E1	41.3	46.3	44.0
E3	42.5	47.9	45.9
E4	43.2	47.4	44.8
E5	38.8	47.0	42.8

(5) Following curing, a load test frame and kentledge comprising 500 tons of steel reinforcing stacked onto a level surface and bound together, was constructed as shown in the figure below. Settlement is measured using 4No. linear variable differential transducers (LVDTs) referenced to a reference plate. Load is applied using a hydraulic jack and pressure is measured on a 100 MPa Bourdon-type pressure gauge with 1MPa resolution (approximately 129 kN). The influence of the load arrangement on the jack was limited by setting the jack into the ground by approximately 1 m and by having the load beam spaced wide apart (approximately 5 m). Readings from the pressure gauge is taken manually, while settlement is measured to 0.01 mm resolution via electronic data collection system. Load is increased or reduced using an electrical motor controlled by a manual switch. The entire test setup was shaded in an effort to shield the instruments from changing atmospheric conditions. (6) A load-unload sequence of 8 No. equal loading steps to 4500 kN and 4No. equal unloading steps to zero was proposed by the contractor to conform to ASTM standards to achieve the estimated ultimate failure load of 4500 kN.

Figure 3. Kentledge (reinforcing steel stacked to 500 tons) setup completed

4 FULL-SCALE TEST RESULTS

4.1 Back-analyses

A prediction of the ultimate vertical compression capacity of the pile was made using the method described in Brown et al. (2007). Although the title of this document refers to continuous flight auger (CFA) piles, the document is intended to present the state-of-the-practice for design and construction of CFA piles, including those piles commonly referred to as augured cast-in-place (ACIP) piles, drilled displacement piles and screw piles. Since the piling system used on the current site utilised a bored drilling method under slurry it was assumed, for the purpose of the back-analysis, that similar analysis methods to CFA piling would apply to the estimation of load. The method estimates both the shaft capacity (with likely maximum mobilised side shear) and the end-bearing capacity. For shaft capacity two methods are proposed, namely the so-called Federal Highway Administration (FHWA) method and the method by Coleman and Arcement (2002).

Following load capacity estimation, the pile settlement results were back-calculated using the method presented in Das (1995). This method requires estimation of the soil modulus along the length of the pile, the modulus at the toe of the pile and the mobilised side-shear at a particular point in time. Although the method is based on elasticity, the mobilised condition, taking into account non-linearity of the soil is provided by the combination of SPT estimation of soil modulus using CIRIA 143 and the iteration of mobilised side shear. Values of mobilised side shear are iterated until the predicted and observed pile settlements are similar. The vindication of the settlement calculation for the purpose of back-analysis is when the mobilised side shear value approaches the maximum side shear value at ultimate load capacity calculated using Brown et al. (2007) in the first stage of the back-analysis. By using the pile settlement measured at each load sequence, the distribution of end-bearing and shaft load distribution was estimated.

The following core parameter assumptions were made to employ the methods described above: (1) Young's modulus, E', of the soil material was estimated from SPT values along the length of the pile; (2) Pile Length taken as 15.5 m, discounting the upper 0.5 m of the 16 m pile; (3) Pile diameter = 0.6 m; (4) Groundwater level below T.o.P. = 0.5 m; (5) Soil unit weight (wet) = 20 kN/m^3; (5) Poisson's ratio = 0.3; (6) Frictional resistance distribution = 0.67; (7) Point Load settlement influence factor, I_{wp} = 0.85; (8) Frictional resistance settlement influence factor, I_{ws} = 3.81.

The back-analyses achieved the following for each test pile: (1) The estimated ultimate vertical load capacity; (2) The estimated working load capacity for design. In this instance a global factor of safety of 2.0 is assumed. The lower factor of safety is based on the fact that there will be four (4 No.) full-scale test piles available on a fairly small site that would provide information required for design. (5) The estimated shaft:end-bearing load carrying ratio and therefore the mechanism of load carrying for design.

4.2 Summary of Results

The measured and back-analysed results of the four test piles are shown in the table below. It is notable that the results of test pile E4 are significantly different to what was observed for the other test piles. This test pile failed before the second load increment could be fully applied.

Table 2 :Pile test results and back-analysis comparison (Note: Pile E2 was not installed)

	Test Pile E1	**Test Pile E3**	**Test Pile E4**	**Test Pile E5**
Closest Borehole [a]	BH18	None (BH13 and BH14 in vicinity)	BH7	BH11
Depth of installation	16 m	16 m	16 m	16 m
Pile diameter	600 mm	600 mm	600 mm	600 mm
Pile head elevation [f]	5.0 m	5.0 m	5.5 m	4.5 m
Maximum applied load (as per conversion between pump load and applied load in kN)	3937.5 kN	3937.5	940.5 kN [Note 3]	3937.5 kN
Estimated Ultimate Load (UL) Capacity,	2252 kN	2813 kN	563 kN [c]	2813 kN

	Test Pile E1	Test Pile E3	Test Pile E4	Test Pile E5
[Shaft stress] [b]	[7.9 MPa]	[9.9 MPa]	[2.0 MPa]	[9.9 MPa]
Estimate Ultimate Pile-Soil Shear Capacity [b]	59 kPa (closest fit using Brown et al., 2007 with Coleman and Arcement, 2002)	80 kPa (closest fit using Brown et al., 2007 with FHWA method)	16 kPa [g]	81 kPa (closest fit using Brown et al., 2007 with FHWA method)
Estimated Shaft:Base Load Carrying Ratio at UL [b]	79:21	84:16	Unknown	78:22
Measured Pile Head Movement at UL [d]	12.3 mm	13.7 mm	2.9 mm < Pile Head Movement < 39 mm [e]	34.5 mm
Back-calculated pile head movement at UL using Das (1995)	12.7 mm	13.7 mm	3.6 mm	35.0 mm

Notes:

a. As per ground investigation report.
b. Results evaluated as per Brown *et al.* (2007) and Das (1995).
c. Test pile E4 failed while attempting to apply the 1125 kN load step. The value of 563 kN as an estimate of UL relates to the only readable load before catastrophic failure occurred and may not be a true reflection of the UL for this test.
d. The estimated pile head movement is taken as the measured pile head deflection during pile load testing.
e. Due to the unexpected catastrophic failure of the pile during application of the second load increment it is not know exactly how much movement was undergone at UL.
f. Pile head levels were surveyed. For the purpose of back-analysis the pile head elevation was taken as 5.0 m for all the test piles.
g. Side shear capacity follows estimation of mobilised side shear during iteration of the settlement calculation using Das (1995).

4.3 Discussion of Test Pile Results

4.3.1 "Normal" pile conditions

Good agreement was obtained between measured values and back-analysed values to such an extent that the ultimate failure load and development of load between shaft and end-bearing could be defined. The methodology proposed also allowed the back-analysis of test pile E4 in order to assess the likely failure mechanism observed. From the back-analyses it is concluded that: (1) The test results for test piles E1, E3 and E5 are considered representative of the contractor's "normal" piling installation process. (2) The combination of methods proposed by Brown et al. (2007) and Das (1995) provide a good estimate of test pile conditions using the ground information from the closest boreholes to the test piles. As a realistic estimate the method proposed by Brown et al. (2007) in combination with estimating the maximum side shear using Coleman and Arcement (2002) provided the closest estimation of ultimate load capacity.(3) Ultimate load capacity of the piles varied between 2252 kN and 2813 kN. The difference between the expected loads is believed to be due to the variable conditions at the base of the pile, including consistency and base contact achieved (affecting end-bearing potential) and the mobilisation of side shear. The maximum side shear estimated during loading varied between 59 kPa and 81 kPa and is believed to be more closely linked to the repeatability of the installation process and to a lesser effect the variability in ground condition. This is postulated because a larger variation may have been expected had it been linked strongly to variation in ground conditions in relation to the data shown in Figure 1. For design purposes a value of 59 kPa was proposed. (4) The ultimate load was achieved at settlements of approximately 2.1 % to 3.5% of pile diameter (typically 12 to 21 mm). (5) At ultimate load the back-analysed shaft:end-bearing load ratio achieved values ranging from 84%:16% and 78%:22%. (6) At loads of approximately 1126 kN (the second load step in the test sequence) the load ratio in all three test piles was 99%:1%, which means that the piles carried the load primarily in shaft friction. At 1687 kN load, the test piles are estimated to carry between 3.5% and 13% of the load in end-bearing. As an indicative value, end-bearing is considered negligible below 1126 kN load.

4.3.2 Progressive shaft and base failure: Test pile E4

The failure of test pile E4 was at first unusual since there is no grounds to expect significantly different ground conditions to what was expected across the rest of the site. Back-analyses using the methods described earlier and using the ground conditions of borehole BH7 predict an ultimate load capacity of 2470 kN. The average E’ of the ground profile using CIRIA 143 is predicted as 50 MPa along the length of the test pile, while the maximum side shear capacity is predicted to be 59 kPa, using the method of Coleman and Arcement (2002). The pile response, however, did not support these parameters.

Using Das (1995) to fit the settlement achieved showed that a peak side shear of only 16 kPa may have been mobilised during application of the first loading step (563 kN). The predicted side shear : end-bearing load ratio would already have been in the order of 83% : 17%, which, based on the results of the other test piles, would have indicated a situation where shaft side shear capacity was fully mobilised and the pile was resisting any additional load in end-bearing. By the time the full 940 kN load was applied to pile E4, full side shear failure would have been activated and the pile would have been carrying load in end-bearing only.

Using the method of Brown et al. (2007) the maximum end-bearing available (under the expected ground conditions) would have been 743 kN. This corresponds well with the fact that total failure (depicted by a continuous increase in pile head movement with no further increase in load) was observed at 940 kN.

Upon researching the installation of test pile E4 it was reported that the contractor had significant difficulty in installing the pile due to collapse of the borehole. The exact depth of the borehole was not recorded. The borehole was redrilled and installation only succeeded later that evening.

Considering the low estimated ultimate side shear value achieved and the low overall resistance of the pile in relation to the other test piles, it is believed that pile E4 experienced a failure in side shear initially, followed by base failure during the second load sequence. The cause of this is believed to be the collapse of the sidewall and the prolonged opening of the borehole, which may have caused a softening of the wall of the pile hole and possibly some loose material remaining in the pile hole. It is also possible that during the drilling operation a soil-bentonite “smear” may have formed that significantly reduced the sidewall friction. This statement can however not be proven, but seems likely in view of the very low side shear achieved. It is further possible that the pile may not have achieved full end-bearing potential due to the collapse; again, this statement cannot be proven, other by observing the overall low pile capacity. These findings were considered to be very significant as they point to the importance of quality and repeatability of the pile installation and proved fears in this regard that resulted in full-scale testing in the first place.

4.4 Estimating the Working Load for design

The back-analyses of the four test piles provided a firm basis for estimating a safe working load for the piles to be constructed on site. Utilising all the data available on site a “lower bound” design line was defined. This line represents a judgement of likely lower bound conditions in relation to E’ and side shear to be expected across the site. Utilising the design line to estimate E’ from SPT values, a maximum side shear of 59 kPa and a factor of safety of 2.0 for base and side shear, it was concluded that for detail design confirmation the following would apply: (1) Pile length = 16 m (measured from t.o.p); (2) Pile diameter = 0.6 m; (3) Pile type = Bored pile installed under bentonite using a Jintai

GPS-15 rig; (4) Ultimate pile load capacity (UL) = 2252 kN (end-bearing and side shear); (5) Working Load (WL) = 1126 kN; (6) Design soil modulus along the length of the pile = 13.5 MPa; (7) Estimated settlement at WL = 7 mm (proposing 5 to 10 mm for design); and (8) Minimum 28-day concrete cube strength = 32 MPa.

4.5 Considerations for construction

Apart from providing a basis of verification of design some important considerations emerged for construction. The pile installation method is paramount to the repeatable and successful installation of a load-bearing pile. This was illustrated very eloquently by the failure of test pile E4. The following issues were found to affect repeatability of installation and were put forward for inclusion into the Quality Assurance plan for site: (1) It was reported that a number of breakdowns occurred during test pile installation (up to 60% downtime was mentioned). Equipment need to be in a reliable working order; (2) Only trained and experienced staff shall be used and staff shall not be changed for the duration of the contract to ensure that they gain experience of the particular site. (3) The process of circulating bentonite into a soil sump where soil is allowed to settle out of the bentonite/soil mix is not acceptable. The bentonite shall have properties as described in BS8004 (or similar) and shall be properly quality controlled, for instance to BS8004 (section 6.5.3.8) or similarly approved project specification. (4) The process of drilling, installing the reinforcement cage and concreting must be a streamlined operation with the borehole drilled quickly and efficiently and the hole not being in contact with bentonite for too long. This may affect the pile-soil interface. At working load the pile is relying in total on side shear. Any reduction in side shear will necessarily have an effect on pile capacity. (5) Pile borehole collapse was observed in at least two of the four test piles and a soft layer occurred at approximately -1.5 m to -4.0 m elevation in a number of boreholes. Mitigation measures, such as temporary casing needed to be considered to ensure borehole stability during pile installation.

5 SUMMARY AND CONCLUSIONS

This paper describes the full-scale testing undertaken and the process and results of the back-analyses. The paper also provides a basis of expectation of pile capacity in Cabinda using locally available equipment and staff. It was found that the methods described in Brown et al. (2007) could effectively be used on the basis of traditional borehole and SPT ground investigation to back-analyse pile response of bored piles in mostly submerged, sandy beach deposit profiles in Cabinda, Angola. Strict control on pile installation time, proficiency and competency of staff with proposed piling equipment and control of fluids, such as bentonite, used during pile installation need to be applied and compiled into the Quality Assurance processes of the site. Very often in remote parts of Africa, as on this particular site, old equipment may be proposed for use. The client and client's representative should be insistent on well-maintained equipment and trained contractor staff.

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