BP CLAIR RIDGE: INDEPENDENT FOUNDATION ASSURANCE FOR THE CAPACITY OF DRIVEN PILES IN VERY HARD SOILS

K Hampson and TG Evans*
*BP Exploration Operating Company, Sunbury, UK (*formerly)*

RJ Jardine
Imperial College, London, UK

P Moran
Lloyd's Register Energy, Bath, UK

B Mackenzie and MJ Rattley
Fugro GB Marine Limited, Wallingford, UK

Abstract
The second phase of development in BP's Clair Field, Clair Ridge, comprises two bridge-linked piled steel jacket structures: one for Drilling and Production, and a second for Quarters and Utilities. The jackets were installed successfully in 2013 in 140 m of water, about 6.5 km north-north-east of Clair Phase 1 platform. The soil conditions at Clair Ridge are similarly extreme but present a higher degree of variability, both in layering and strength. Despite invaluable experience from Phase 1, the Clair Ridge soils pose challenges for designing and installing driven piles. This paper describes the approach taken by BP's foundation assurance team to provide comprehensive validation of long-term axial pile capacity. The effects of pile slotting, cyclic loading and group action were considered, referring to the soils' mechanical properties revealed by comprehensive geotechnical site investigations. Validation method calibration by driving data back-analysis is also discussed.

1. Introduction

1.1 Background

The Clair Field is located about 75km west of the Shetland Islands, UK. The first development (Clair Phase 1) was sanctioned by BP and its co-venturers Chevron, ConocoPhillips and Shell, in 2001 and production began in 2005 via a single fixed piled steel platform and associated oil and gas export facilities. Evans et al. (2011) present various aspects of the development planning. A second phase of production, Clair Ridge, comprised two bridge-linked piled steel structures: a Drilling and Production (DP) platform and a Quarters and Utilities (QU) platform and was sanctioned in 2011. A drilling template and two instrumented 72-inch diameter docking piles were pre-installed at the DP platform location in mid-2011 and these installations were adopted as driving trials for the main platform piles. The platform jackets were installed in 2013 in a water depth of approximately 140 m. The topsides were added in mid-2015 (QU) and mid-2016 (DP) and are shown in Figure 1. First oil production is expected late in 2017. The detailed design and fabrication of the steel jackets and foundations were performed by Kvaerner Jacket Technology (KJT) and Kvaerner Verdal (KV), respectively. The foundation design was supported by the Norwegian Geotechnical Institute (NGI). The jackets and topsides were installed by Heerema Marine Contractors (HMC).

Figure 1: The Clair Ridge DP and QU platforms

The piled foundations for the Clair Ridge DP platform comprised five 2.74 m (108 inch) diameter, 100 mm uniform wall thickness piles per corner leg, driven to penetrations from 25 m to 40.5 m. The QU platform is founded on three 2.59 m (102 inch) diameter, 100 mm uniform wall thickness piles per corner, driven to between 25 m and 27 m.

Two piles at each of the jacket legs were instrumented with strain gauges and accelerometers, which were monitored continuously during driving. Redrive tests were performed at target penetrations on two instrumented piles from each jacket.

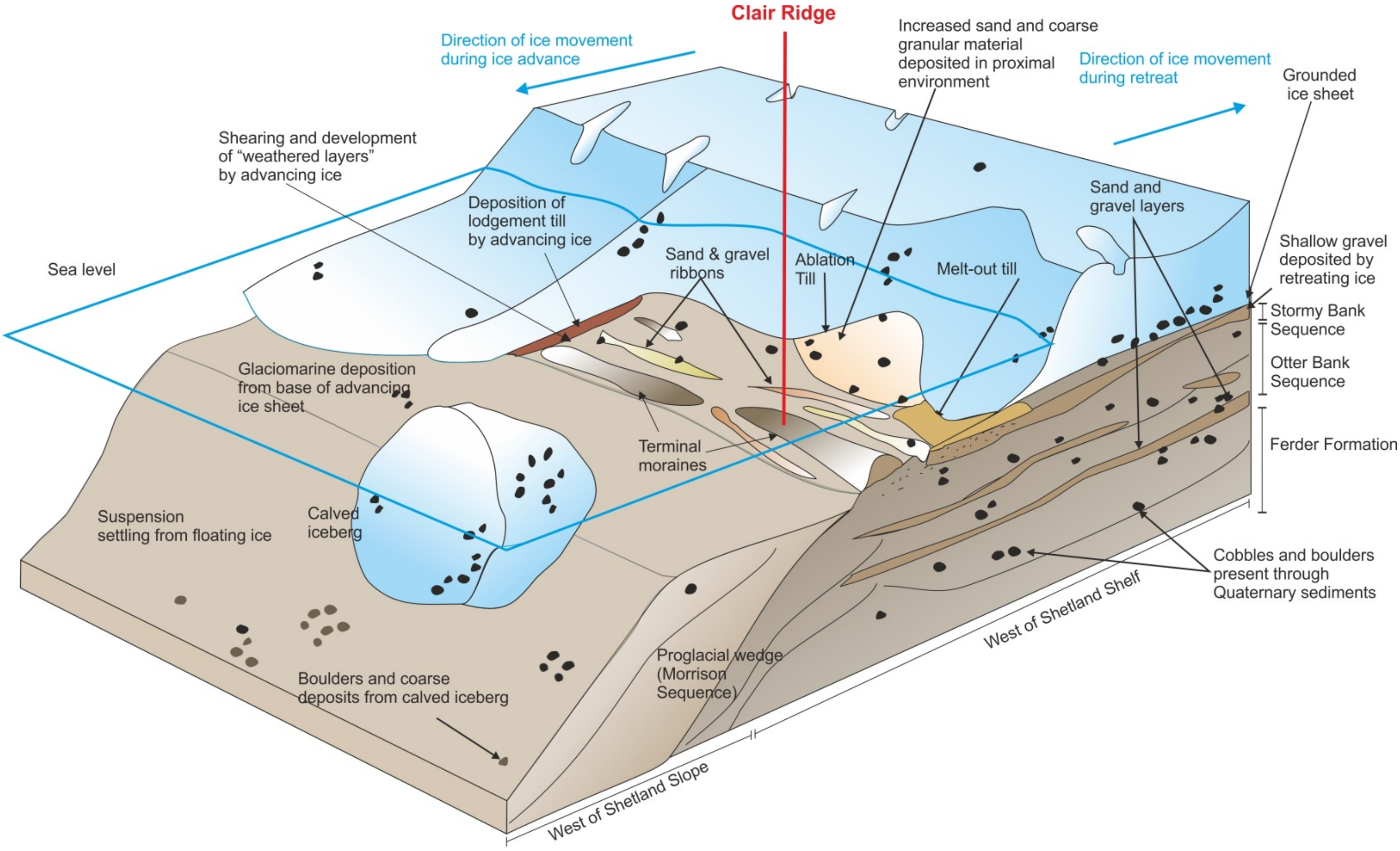

Figure 2: Conceptual summary of glacial processes on the West of Shetland Shelf

1.2 Site Conditions

The Clair Field is underlain by soils that were deposited, compressed and repeatedly sheared by hundreds of metres of ice during successive glaciations. Waxing and waning of glacial ice sheets has largely controlled the spatial and temporal distribution of particular sediment packages. The depositional environments and the associated stress history have had a major effect on the soil properties. Of particular interest are the glacial deposits of the Otter Bank sequence, which occurs extensively on the outer part of the West of Shetland Shelf. These soils comprise interbedded hard clays and very dense sands with gravel to boulder-size igneous rock inclusions. Figure 2 shows a conceptual summary of some key glacial processes on the West of Shetland Shelf.

Shallow geophysical and geotechnical surveys were performed at Clair Ridge in 2009. The soils were found to be similar to those at the Clair Phase 1 platform site, where extremely hard and dense tills were encountered (Aldridge et al., 2011; Jardine et al. 2011); however, greater levels of lateral variability, and thicker sand units were noted.

1.3 Design Challenges

Despite invaluable experience from Clair Phase 1, the soils, cobbles and boulders at the Clair Ridge platform locations still posed challenges for the design and installation of driven piled foundations:

- The ISO 19902:2007 methods for estimating static pile capacity are based empirically on the API pile load test database, reported by Olson (1984) and assume that the embedded length to diameter (L/D) ratios of offshore piles are sufficiently high (typically >15) for axial capacities to be unaffected by lateral and moment loads;
- The Clair tills are much stronger than those included in the API pile test database. In addition, the L/D ratios of the DP and QU jacket piles (9 to 14) fall below the range represented in the database and arguably below the range at which axial and lateral response interactions may be negelected. In effect, there were no validated methods for estimating the pile' axial capacities;
- The Clair Ridge deposits were too laterally variable (see Figure 2) to support a design approach based on average soil properties over the structure's footprint. A leg specific geotechnical data based approach was required;
- While Aldridge et al. (2011) report easier than predicted continuous pile driving at Clair Phase 1, significant set-up was also observed during driving pauses. There was potential for hard driving and possible pile refusal in the event of

- long driving delays occurring at penetrations near to the target depths; and
- Cobbles and boulder-size inclusions posed potential risks to pile installation that might result in (1) hard driving (2) excessive pile damage and/or (3) pile refusal.

The Clair Ridge Project design team applied a 'best-endeavours' approach to the assessment of axial pile capacities, similar to that adopted during Clair Phase 1 (Evans et al. 2011). The process involved using the industry standard ISO procedures for developing the base case designs and validating those solutions using other robust, physically reasonable approaches. An independent foundation assurance team (IFAT) was appointed by the Project to validate the DP and QU jacket piled foundation solutions. The IFAT team comprised specialists from BP, Imperial College, Fugro and Senergy. This paper describes the technical approach taken by the team, with a particular focus on the comprehensive validation of long-term axial pile capacity.

The team was also asked to carry out independent predictions of pile driveability to develop pile installation acceptance plans and to monitor the pile installations. However, detailed descriptions of these activities are outside the scope of this paper.

2. IFAT Design Assurance

2.1 Strategy

Given the design challenges associated with the Clair Ridge platform foundations, a key objective of the IFAT work was to confirm that the base case design had sufficient theoretical reserve capacities to accommodate minor installation problems without necessarily requiring remediation. The design robustness criterion set by the Clair Ridge Project was that the tolerable utilisation factors (UFs) for maximum loaded single piles and maximum loaded pile groups were not greater than 0.9 in axial tension and compression.

2.2 Basis for Approach

The approach taken by IFAT was based on the following convictions concerning the Clair Ridge soils and the design of open-ended tubular driven steel piles in such soils:

- The hard clay tills encountered at the platform locations are similar to those encountered at the Clair Phase 1 platform;
- The soils are not cemented and their high strengths are due to mechanical processes only;
- The soils are sufficiently variable to require leg-specific pile designs;
- The ISO procedures used to estimate the base case axial pile capacities would be conservative for Clair Ridge, because it does not fully capture the positive biases for low aspect ratio piles and for clays with high yield stresses observed in open-ended tubular steel pile test databases;
- Methods that are based on sounder soil mechanics principles and/or are a better statistical fit to the subset of pile test data that most closely match the Clair Ridge pile geometries and-soil conditions would give more reliable and less conservative characteristic pile capacities than the ISO method;
- Characteristic pile capacities may be reduced by post-peak softening of unit skin friction along the pile shafts and should therefore be considered in design; and
- Project-life operational factors such as local and general scour, gapping/slotting under cyclic lateral loading, pile group interaction and axial cyclic loading may reduce the basic characteristic capacities of the Clair Ridge piles. While these operational factors should be addressed, it may be overly conservative to apply them all to characteristic pile capacities derived by the potentially overconservative ISO method.

2.3 Validation Process

Figure 3 illustrates the validation process adopted by the IFAT team. The characteristic axial compressive and tensile pile capacities represented the expected capacities that a single isolated pile would develop if loaded quasi-statically, with no other factors taken in to account. Operational factors were those as expected to arise during the 40-year life of the platforms and which may reduce pile characteristic capacities: (1) general scour of near-surface sands, (2) gapping/post-holing and other damage in the hard clays due to lateral cyclic loading, (3) pile group interaction and (4) axial cyclic loading. Operational static axial compressive and tensile capacities were then obtained by multiplying the corresponding characteristic pile capacities by the operational factors in the exact sequence shown on Figure 3.

2.4 Validation Methods

Two different validation methods were adopted to estimate the long-term characteristic axial capacities of the DP and QU platform piles. A third level of validation was obtained by calibrating the two predictive methods with site-specific pile driving data. :

- Validation Method 1 (VM1) adopted the ICP effective stress method for driven piles in sands and clays (Jardine et al., 2005), which has been calibrated for range of soils, including those with glacial origins;

- Validation Method 2 (VM2) was based on the ISO method, but considered the statistical bias of the method for open-ended, low aspect ratio, piles in heavily overconsolidated clays; and
- Validation Method 3 (VM3) was a calibration of the VM1 and VM2 predictive procedures by comparison of the long-term characteristic static pile capacities estimated for pre- installed 72 inch template docking piles with those extrapolated from driving records using signal-matching techniques.

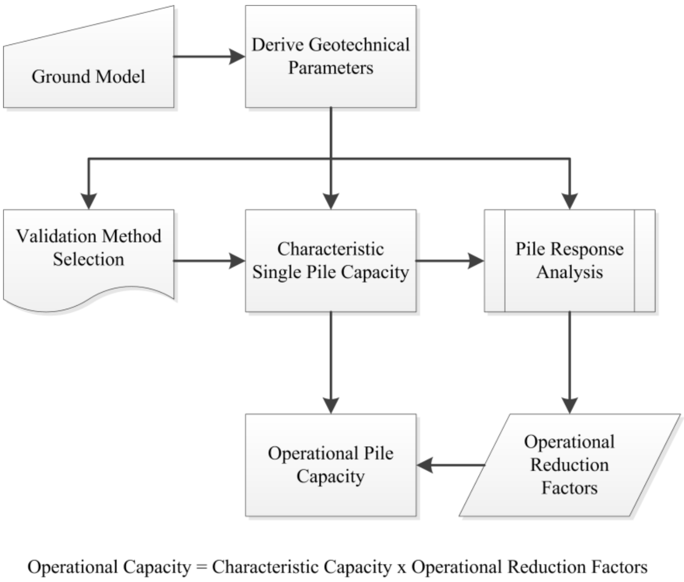

Figure 3:Axial pile capacity validation process

3. Geotechnical Parameters

3.1 Site Investigation

Combined piezocone (CPTu) and deep boreholes were performed at each corner of the DP and QU platforms, along with one further deep borehole at the drilling template location, giving nine deep boreholes in total. High quality samples were recovered from each sampling borehole. Static pile bearing capacity analyses using ISO 19902:2007 were performed during the course of the investigation to ensure that each borehole depth exceeded the recommended pile penetration by ~3 diameters plus 6 m or to a minimum of 45 m below seafloor (BSF). Based on this approach, a single borehole was terminated at 60 m BSF with investigation at all other platform corners being terminated at depths from 45 m to 50 m BSF. Boreholes were performed using both PQ coring and API rotary drilling techniques. Eight further shallow boreholes were performed (one per leg location) to facilitate mudmat design in the variable soil profiles.

Extensive series of offshore and onshore laboratory tests were performed on samples recovered from the investigation. These series included advanced stress path triaxial, cyclic simple shear, and interface shear testing and were similar to that reported by Aldridge et al. (2011) and Jardine et al. (2011). All parameters required for input to the ICP method and cyclic analysis were carefully measured by comprehensive laboratory testing.

3.2 Integrated Ground Model

IFAT's work was informed by a shallow geological and geotechnical engineering ground model for Clair Ridge that was developed following a detailed assessment of the available geophysical, geochronological, geological and geotechnical data for the Clair Field.

3.3 Design Soil Profiles and Parameters

The Clair Ridge soils were found to be highly laterally variable. Soil profiling and parameter selection were therefore performed on leg-by-leg basis. Generally this process was straightforward and based on the available data. In some cases a holistic approach, based on seismostratigraphic unitisation from the engineering ground model, was required. This was particularly useful in cases where only short CPTu strokes could be achieved in the hard soil conditions. Figure 4 summarises some key geotechnical parameters for one of the leg locations investigated.

Location-specific cone factors were derived for clay layers to account for regional depositional and post-depositional effects. Notably high N_{kt} values, between 18 and 33, were required to find s_u values from net cone resistance. The soil:steel interface friction angles δ measured according to ICP method procedures were also found to be relatively high with δ_{ult} ranging from 26º to 28º in the clay units and a limiting value of 28.8º being assigned to the dense sand units.

3.4 Stress History and Yield Stress Ratio

The stress history of the soils at Clair Ridge required careful attention. Whilst the exact nature and extent of previous glacial events was uncertain, it is clear these formations have experienced significant loading and unloading phases, shear deformation and possible weathering. To assess the effect of these processes, the relationship between yield stress ratio (YSR), vertical effective stress and undrained shear strength was investigated within a rational framework, as outlined by Jardine et al., (2005).

Existing oedometer based methods for prediction of yield within dense glacial tills often produce poor quality correlation or a poorly defined yield point. For this reason an apparent 'prior preload' concept was introduced. The holistic approach adopted for prediction of vertical effective yield stresses σ'_{vy} was calibrated directly against laboratory strength data (Figure 4a) for each specific clay layer. The apparent degrees of preload were considered across each jacket's footprint. An iterative method was then used to calibrate a final YSR = $\sigma'_{vy}/\sigma'_{v0}$ profile against depth from CPT and laboratory strength data at each leg location (Figure 4b).

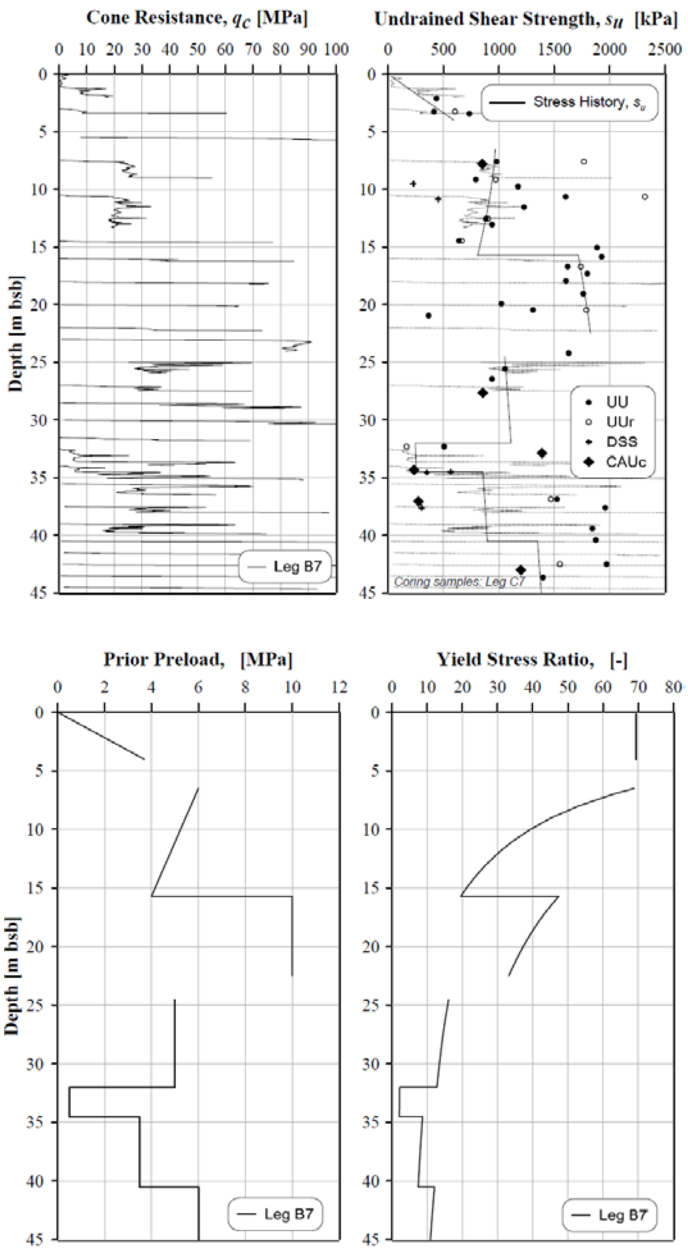

Figure 4: Variation of (a) measured q_c and s_u and (b) iterated prior preload and YSR for one DP platform leg location

3.5 Validation Method Parameters

The unified approach to parameter selection allowed a consistent set of design parameters to be applied to estimate characteristic pile capacities using both VM1 (effective stress method) and VM2 (modified total stress method). The approach also allowed a more realistic assessment of soil strength and YSR profile with reference to previous geological loading and stress history of the clay layers. The q_c values used for sand layers were interpreted from CPTu data alone. This was also carried out within a holistic framework for each seismostratigraphic unit. The application (where possible) of a standardised set of parameters across VM1 and VM2 eliminated subjectivity in the dataset and allowed a more rational approach to method comparisons.

4. Characteristic Axial Pile Capacities

4.1 Validation Method 1

VM1 adopted the ICP method for driven piles to provide leg-specific estimates of characteristic axial compressive and tensile capacities, based on the geotechnical parameter selection process described above.

All ICP pile calculations indicated that the DP and QU platform piles would behave in an unplugged manner under static loading at final pile penetration. In view of the erratic q_c data and soil layer variability, the q_c profiles used for estimating unit pile base resistance were chosen more conservatively than those adopted for calculating pile shaft capacity. This approach is recommended by Jardine et al. (2005; 2011) and discussed in detail by Jardine et al. (2015) for similar circumstances and leads to a lower risk of piles terminating in a weaker-than-expected layer. Although not significant for the glacial clay units, the effect of strain softening was considered implicit in VM1, since δ values were assigned based on interface ring shear testing to residual states.

4.2 Validation Method 2

The primary objective of VM2 was to establish whether any conservatism is inherent in the ISO method when considering the subset of the API pile test data which most closely match the Clair Ridge conditions. This analysis was employed to establish statistical bias factors for direct input into pile shaft capacity calculations.

The pile test database subset considered 21 high quality tests on open-ended piles in overconsolidated clays from the test data collated for API (Olson, 1984) and by Imperial College (Chow, 1997). The approach takes advantage of positive statistical biases for high YSR clays and piles with low aspect, L/D* ratios, where D* is the equivalent diameter of a solid pile. Figure 5, which shows the relationship between the ratio of measured to estimated shaft friction and L/D*, illustrates the aspect ratio bias. For the final pile penetrations at the DP and QU platforms, shaft friction bias factors were assessed to be in the range 1.20 to 1.30. A single bias shaft friction factor for sand layers was adopted directly based on the data published by Lehane et al (2005).

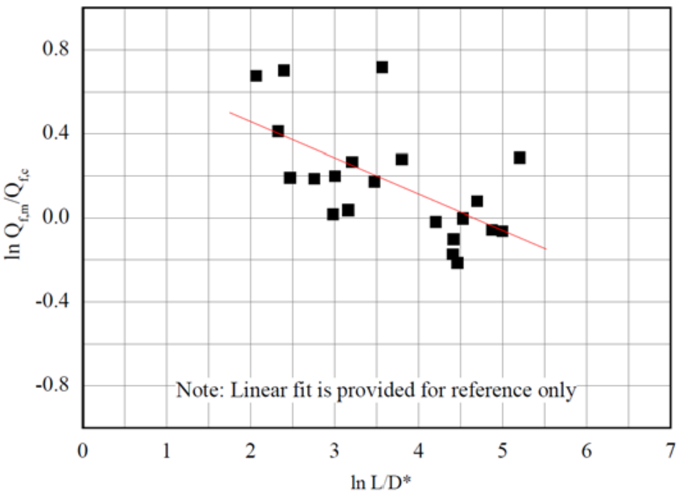

*Figure 5: Variation of API pile test database trend with L/D**

Site-specific triaxial and interface ring shear tests showed that the glacial clay units do not exhibit pronouncedly brittle behaviour; the characteristic pile capacities derived by VM2 were not reduced for strain softening.

5. Operational Reduction Factors

5.1 Global Scour

Variable thicknesses of loose to medium dense Holocene sands were observed at the seabed at both platform locations. Erosion of these soils was considered by including 1 m of general scour when calculating operational pile capacities.

5.2 Pile Gapping/Slotting

The low plasticity, hard-clay tills encountered at Clair Ridge are susceptible to large plastic displacements and it was considered that gaps may form around piles under repetitive lateral loading. Their relative effect on axial capacity would be exacerbated by the relatively short piles (L/D ~ 9 to 14). Gapping will result in a reduction of axial shaft friction through loss of contact near the tops of the piles and possibly from cyclic softening of a zone of soil below the gap that may be disturbed by cyclic loading that does not reach the limit required for gapping to develop. The effects of gapping under lateral loading were evaluated by performing beam-column analyses, using cyclic lateral load transfer curves (p-y curves) and soil yield criteria which were back-analysed from lateral load tests in very stiff glacial tills at Tilbrook Grange (Long et al., 1992). The beam-column results were used to estimate the depths of gaps and disturbed and softened zones that may form around the DP and QU Platform piles under operational (cyclic) conditions.

Shaft capacities were reduced for gapping effects by assuming zero skin friction over the gap depths and by decreasing unit skin friction in the underlying disturbed zones by applying depth-dependent damage reduction factors. For the most heavily laterally loaded piles of the DP platform, gap depths up to approximately 6 m BSF, were predicted. Partial damage reduction factors that reduced to give no cyclic damage at 13 m BSF were applied below the gapping depth.

5.3 Pile Group Interaction

It is normal to reduce pile design capacities for group effects in onshore civil engineering but this is not common practice in the offshore construction industry. Group effects can be beneficial for piles driven into sands but they are invariably negative for driven piles in clays. The numbers of piles in each group for the DP and QU jackets are relatively small but they are closely spaced and driven through predominantly clayey soils. Group effects may therefore be significant.

Recent evidence for the reduction in operational axial pile capacities due to group interaction has been obtained from tests carried out on piles driven through clay-silts in Northern Ireland (Lehane and Jardine, 2003; Lehane et al., 2004). The effects of pile group interactions were assessed using a simplified method of overlapping and compounding shear stress fields developed from the results of this research work. Using this procedure, average reductions of shaft friction of the order of 20 % for the DP jacket pile group, and 9 % for the QU jacket pile group were derived.

5.4 Axial Cyclic Loading

Different approaches were adopted for assessing the potential effects of cyclic loading on axial pile capacities for VM1 and VM2. In each case, the assessment was based on 35 hour characteristic/unfactored pile loads for a 100-year storm, assuming that cyclic degradation only affects pile shaft friction. The cyclic degradation models used for VM1 and VM2 were derived from cyclic direct simple shear (CSS) tests. The approach applied for VM1 was consistent with that described by Merritt et al. (2012) and Rattley et al. (2017) for recent off-

shore wind farm projects. A total stress accumulative strain approach was developed for VM2.
Table 1 summarises the final predicted shaft capacity degradations under cyclic loading associated with the 100-year storm for the DP and QU platform jacket piles.

Table 1: Summary of cyclic degradation predictions

Platform	Predicted cyclic degradation [%]	
	VM1	VM2
DP	6 to 20	5 to 13
QU	10 to 15	8 to 10

6. Operational Axial Pile Capacities

6.1 Summary

Figure 6 shows the VM1 and VM2 characteristic, biased (VM2), and operational axial pile capacities in compression at 25 m pile penetration for an example jacket leg case. Although the methods started from different technical perspectives, the final operational capacities predicted by VM1 and VM2 were in good agreement.

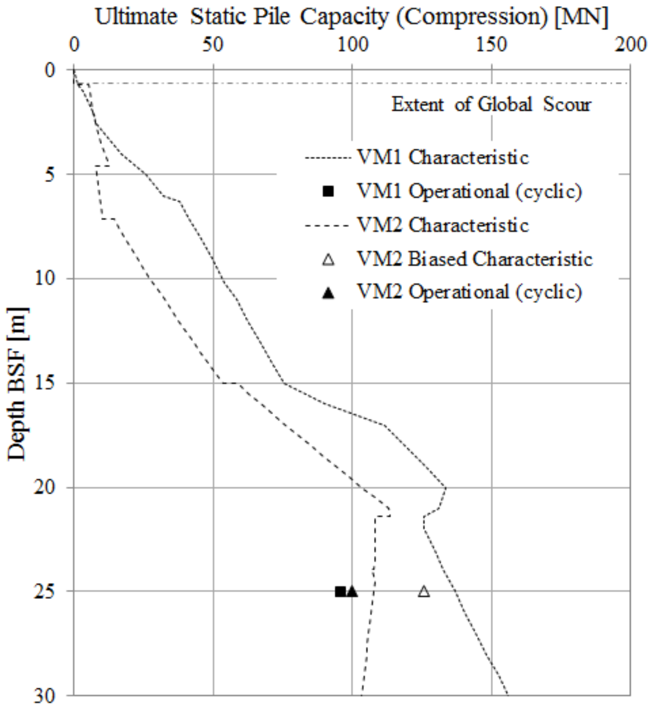

Figure 6: Example pile capacity profile showing VM1 and VM2 characteristic and operational compression capacities

Across all jacket legs, piles driven to base case target penetrations were estimated to have operational axial pile capacities sufficient to resist the axial loads derived from the 1-in-100 year ULS in-place jacket analyses. The utilisation ratios (URs) in axial tension and compression, expressed as the ratios of the partial pile resistance factors specified by the Clair Ridge project to the calculated pile resistance factors, were estimated to be less than 0.9 for individual piles and for each pile group.

7. Validation Method 3

7.1 Approach

The third validation method, VM3, involved calibrating VM1 and VM2 by comparing the axial compressive capacities estimated by the two methods for the pre-installed 72 inch docking piles with those inferred from these piles' driving records.

The docking piles were monitored during installation allowing signal-matching analysis for predictions of static soil resistance during driving (SRD). From each of these analyses, back-figured distributions of friction mobilised along the pile shaft are available, in addition to overall back-figured static shaft and tip resistance. The VM1 and VM2 methods were applied to predict the long-term characteristic static axial pile capacity for the 72 inch docking piles based on soil profiles and parameters derived on a location-specific basis using the same approach described earlier for the platform piles. No operational reduction factors were applied. The VM1 and VM2 predictions were then compared to the installation static SRD values at various penetrations corresponding to different L/D* values. As expected, the shaft capacities at end of driving fall below the anticipated long-term capacities and the comparison identified the amounts of post-driving set-up required to achieve the long-term capacities predicted by VM1 or VM2. The required values were then compared with set-up rates observed from the full pile installation dataset available for the Clair Field.

7.2 Required Pile Set-up

VM3 indicated that, for the 72 inch docking pile case at final penetration, pile set-up of approximately 38 % would be required to achieve the pile compressive capacity predicted according to VM1. A lower required set-up value of 2 % was inferred for VM2. These required pile set-up values increased to 76 % for VM1 and 15 % for VM2 when only the shaft friction component of the predicted and back-analysed static pile resistance was considered.

Back-figured pile shaft capacity set-up data from previous pile installations in the Clair Field, at time t was compared to the piles' immediate post-installation shaft capacity (i.e. at t = 0) leading to the dataset presented in Figure 7. The trends from upper and lower bound set-up projection curves, developed from the expressions of Bogard and Matlock (1990), are also plotted. The set-up factors required for VM1 and VM2 fall below those indicated by the lower bound curve and were considered likely to be achieved.

Later monitoring of the Clair Ridge DP and QU jacket pile installations in 2013 indicated static SRD at final penetration within the range expected. Although not reported here, the piles' set-up data plotted between the curves shown on Figure 7 and so further supported the IFAT capacity predictions.

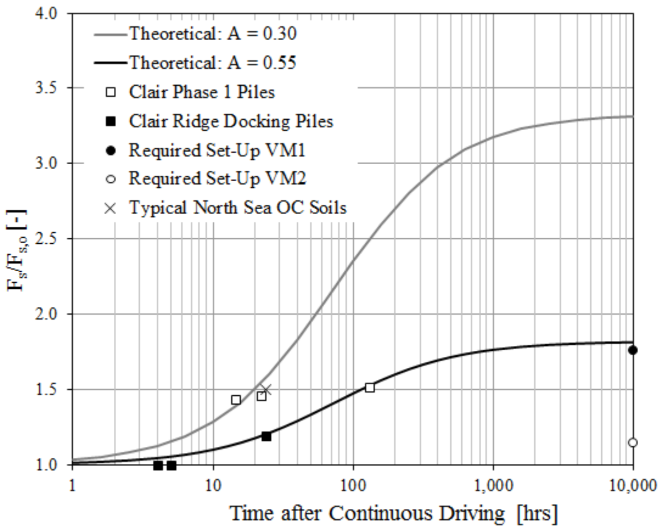

Figure 7: Clair-specific pile shaft friction set-up curves, where A is the ratio of immediate to long-term shaft capacity

8. Conclusions

1. Driven pile design for the two Clair Ridge jacket structures had to consider the exceptionally hard and variable glacial soils encountered, the piles' relatively low L/D* ratios and the significant cyclic loading levels anticipated in the West of Shetlands location.
2. The primary design work was supplemented by an independent foundation assurance team that considered three alternative pile validation methods and addressed the impact on axial capacity of a series of additional factors, including cyclic axial and cyclic loading that are often neglected.
3. The IFAT's independent analyses, supported by advanced site investigations, indicated that the final pile designs would be fit-for-purpose under the anticipated design storm conditions.

9. Acknowledgements

The Authors gratefully acknowledge the permission of BP and their co-venturers Chevron, ConocoPhillips and Shell to publish the work described in the paper.

10. References

Aldridge TR, Carrington TM, Jardine RJ, Little R, Evans TG and Finnie I. (2011). BP Clair Phase I – Pile driveability and capacity in extremely hard till. *Frontiers in Offshore Geotechnics II*, Perth, Taylor & Francis Group.

Bogard JD and Matlock H. (1990). Application of model pile tests to axial pile design. *Offshore Technology Conference*, Houston, OTC

Chow FC. (1997). *Investigations into displacement pile behaviour for offshore foundations*. PhD Thesis, Imperial College, London.

Evans TG, Finnie I, Little R, Jardine RJ and Aldridge TR. (2011). BP Clair Phase I – Geotechnical assurance of driven piled foundations in extremely hard till, *Frontiers in Offshore Geotechnics II*, Perth, Taylor & Francis Group.

ISO 19902:2007, *BS EN ISO, Petroleum and natural gas industries – Fixed steel offshore structures*, International Standards Organisation.

Jardine RJ, Aldridge T and Evans TG. (2011). Offshore foundation engineering in extremely dense glacial tills West of the Shetland Islands. *Proc. 15th ECSMGE*, Athens, pp 879-884.

Jardine RJ, Chow FC, Overy RF and Standing JR. (2005). ICP design methods for driven piles in sands and clays. Thomas Telford Ltd.

Lehane BM and Jardine RJ. (2003). Pile group tension cyclic loading. Field test programme at Kinnegar, N. Ireland. Research report 101, Health & Safety Executive.

Lehane BM, Jardine RJ and McCabe BA. (2004). Response of a pile group in clay to first time one-way cyclic tension loading. *Advances in Geotechnical Engineering. Proc. Skempton Memorial Conference*, Thomas Telford.

Lehane BM, Schneider JA and Xu X. (2005). CPT based design of driven piles in sand for offshore structures, GEO:05345, The University of Western Australia.

Long MM, Hamilton J, Clarke JC and Lambson MD. (1992). Cyclic lateral loading of an instrumented pile in over-consolidated clay at Tilbrook Grange, *Proc. Recent Large Scale Fully Instrumented Pile Tests in Clay*, ICE, London.

Olson RE. (1984). Analysis of pile response under axial loads, Report to API.

Merritt A, Schroeder F, Jardine R, Stuyts B, Cathie D and Cleverly W. (2012). Development of pile design methodology for an offshore wind farm in the North Sea. *Proc 7th Int. Conf. on Offshore Site Investigations and Geotechnics*, SUT London, pp 439-448.

Rattley MJ, Costa L, Jardine RJJ and Cleverly W. (2017). Laboratory test predictions of the cyclic axial resistance of a pile driven in North Sea soils, *Proc. 8th Int. Conf. Offshore Site Investigation and Geotechnics*, SUT London.