East Anglia ONE
Offshore Windfarm

Economic Impact of ICP-05 Design Method

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1 Introduction

1. ScottishPower Renewables (SPR), part of the Iberdrola group, is developing the East Anglia ONE Offshore Windfarm (EAONE). In June 2014, its Development Consent Order (DCO) was secured for the project, followed by the award of a Contract for Difference in February 2015. The project achieved a positive Final Investment Decision (FID) by February 2016 and the construction and installation works are being carried out between 2018 and 2020.

2. This document provides an overview of the different activities and EAONE work packages affected by the use of the geotechnical design method developed by the Imperial College London (Jardine et al, 2005), ICP-05, and the impacts on the project as a whole.

3. EAONE is the first area to be developed within the East Anglia zone and covers an area of approximately 300 km^2 being located in the south of the zone. The export cable route extends for about 122km, being 85km from the EAONE offshore substation to the landfall at Bawdsey and 37km onshore to the national grid substation at Bramford.

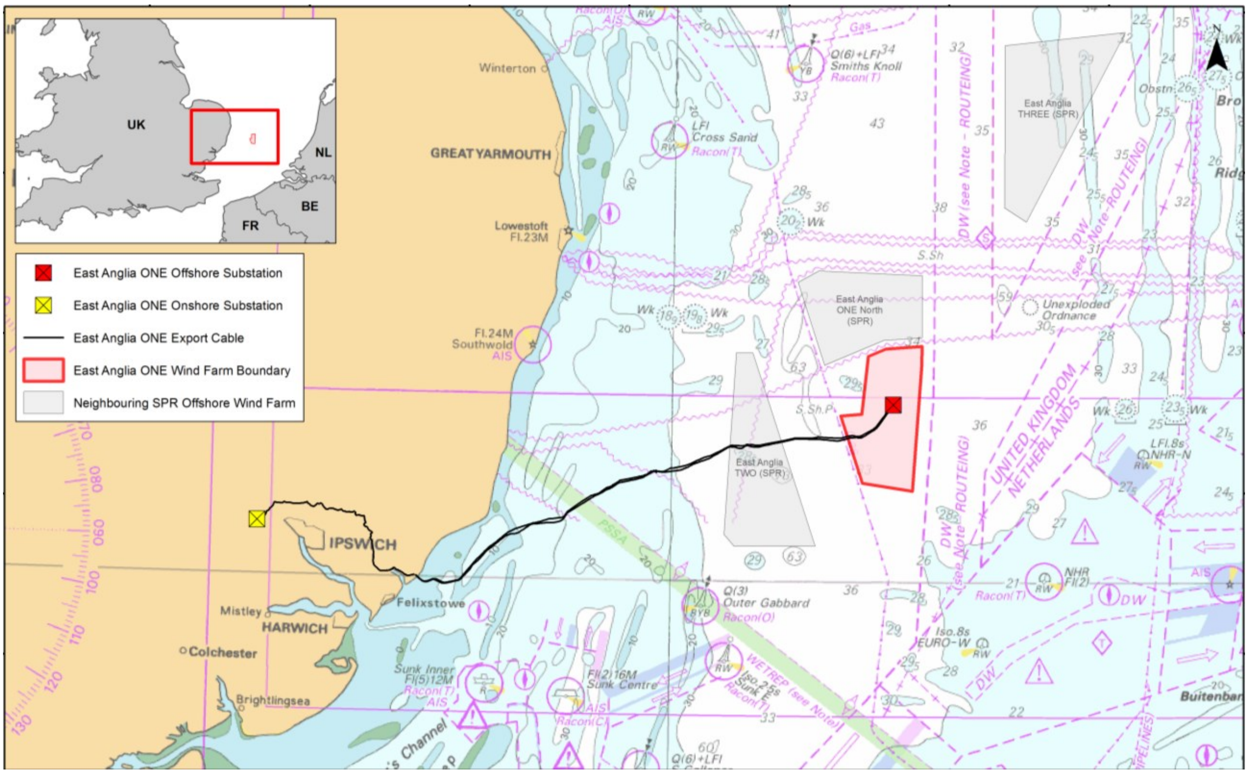

Figure 1-1: East Anglia ONE project (red line) and export cable route

4. The site is formed by 102 7MW turbines (WTG) supported by 3 legged jackets sitting on 3 x 2.5m diameter piles each jacket. The hub height is approximately 90m above sea level and the rotor diameter is 154m.

5. The southern North Sea is a relatively shallow sea. Within the EAONE site, water depths are recorded at average depth of around 44 m LAT, with a maximum depth of 53 m LAT and a minimum depth of 31 m LAT. The majority of the area is in water depths ranging between 40-45 m LAT while shallower waters are normally associated to the presence of sand waves.

6. Geotechnical engineering was undertaken with the support from Offshore Wind Consultants (OWC) and the WTG foundation design was carried out by Ramboll. The cyclic degradation assessment was performed by Fugro

Geoconsulting (Fugro) in consultation with Professor Richard Jardine at Imperial College London (ICL) and OWC and subsequently and subsequently approved by Ramboll for design.

1.1 Soil Conditions

7. The stratigraphy at the EAONE site was found to be generally dipping towards the northeast with some variations in thickness of the different geological units. The stratigraphy of the area (to the depth of interest) is mainly formed by medium dense to very dense sands (occasionally clayey and silty) and clay layers (part of the Westkapelle Ground Formation). The soils encountered are mainly part of the following geological formations: Holocene, Yarmouth Roads, Smith's Knoll and Westkapelle. Occasionally thin clay layers from the Brown Bank Formation could be encountered on the western edge.

8. The following geological and geotechnical engineering units have been identified on the site and were used as the basis of all design works carried within the project:

Geological Units		Engineering Units		Description
HOL	Holocene	1HOL	Holocene	Marine sediments
BB	Brown Bank	2BBC	Brown Bank Clay	Sandy silty Clay
		2BBS	Brown Bank Sand	High strength to very high strength Clay
YR	Yarmouth Roads	4Y2C	Yarmouth Roads II Clay	Very high strength Clay
		4Y2S	Yarmouth Roads II Sand	Dense to very dense fine to medium Sand
		4Y1S	Yarmouth Roads I Sand	Dense to very dense very silty fine to medium Sand
SK	Smith's Knoll	5SMK	Smith's Knoll	Medium dense to very dense fine to medium slightly silty Sand
WK (b_W1C)	Westkapelle (Base WK I Clay)	6W2S	Westkapelle Ground II Sand	Medium dense very silty fine to medium Sand
		6W1C	Westkapelle Ground I Clay	Very high to extremely high very silty Clay
		6W1S	Westkapelle Ground I Sand	Very dense very clayey silty fine to medium Sand

Table 1-1 – Summary of Ground Model and Engineering Units in EAONE

1.2 Design Methodology

9. The geotechnical design method followed for the foundation pile design was the ICP-05 method. The background for using such method is clarified in this section.

10. The design methods for prediction of axial pile resistance for driven tubular steel offshore piles originally considered for the EA1 foundation design were:

- API RP2GEO (2011);
- DNV (1992);
- **ICP-05 (Jardine et al, 2005);**
- UWA-05 (Lehane et al, 2005);
- NGI-05 (Clausen et al, 2005); and
- Fugro-05 (Kolk et al, 2005).

11. The API and parallel ISO documents set out ‘Main-Text’ sections that specify well-established simple design methods for piles driven in both sands and clays. These methods call for basic geotechnical parameters that are gathered in routine offshore site investigations. DNV (1992) presents methods for predicting the axial resistance of piles driven in sands and clays similar to those included in the API ‘Main-Text’ and Commentary, with similar methods recommended for accounting for pile length effects in clays. An early CPT method for driven piles in sand is mentioned in DNV (1992); however this method was not recommended for use in design due to a lack of relevant pile load test data that was available at that time.

12. The API methods have been applied for many years in offshore projects. However, there is widespread recognition that these pile simplified design models do not capture many features of behaviour accurately. In particular, it is well known that axial resistance measurements made with the sand methods are subject to considerable bias and scatter; see for example Jardine and Chow (1996; 2007). A range of alternative methods has been developed to address this shortfall. Four such approaches (that are generically termed ‘Modern CPT methods’) are recognised in the Commentary section of the API RP2GEO (2011) document as being fundamentally better and these are now being applied in a wide spread of offshore applications, particularly in the North Sea.

13. The four ‘Commentary’ approaches for assessing axial resistance in sand comprise the ICP, UWA, NGI and Fugro methods. The methods address key features seen in field experiments made with the Imperial College Instrumented Piles (ICPs) at Labenne and Dunkirk where the local distributions of shaft radial and shear stresses were studied in detail. As detailed by Jardine et al. (2005) and Jardine and Chow (2007) these experiments proved that the conventional approach of assuming constant K values gave a very poor fit to the observed field radial effective stress behaviour.

14. The ICP approach was the first to be proposed and it had been validated (rather than calibrated) against a database of 81 high quality static pile tests in sand by Chow (1997) and in an updated and extended database by Jardine et al (2005), covering diameters between 0.1 and 2.0m, lengths from 1.8 to 47m and sand average relative densities between 31 and 100%. They found that the ICP method more than halved the API/ISO method’s considerable scatter (expressed as a CoV ˜ 0.6 in API calculated/measured capacity) and avoided its systematic skewing with regard to relative pile length (L/D), pile diameter (D) and relative density (Dr).

15. It should also be noted that the ICP-05 approach gives the best predictions for the offshore static tension pile test conducted in dense North Sea sands at the North Sea Leman BD complex (Jardine et al 1998), in multiple tests in dense North Sea sand at Dunkirk, Northern France (Jardine et al 2006) and for the EURIPIDES tests conducted (on 760mm diameter open steel piles of lengths up to 47m) at Eemshaven, close to the German-Dutch border, as described by Kolk et al (2005). All three sets of tests involved North Sea sands comparable to those present at the EAONE site. The case histories, references and validation checks are explained fully by Jardine et al (1998), (2005a) and (2006). In contrast, the API method was found to be highly overconservative in estimating the resistances proven in multiple high quality static tests at the EURIPIDES and Dunkirk sites.

16. There was less widespread acceptance of the ICP-05 clay method, which is effectively the same as the Jardine and Chow (1996) approach. However, this method was demonstrated to give far greater reliability than the API main text method through rigorous database studies involving almost 100 pile load tests by Jardine et al (2005). This approach applies effective stress principles, rather than a total stress method, and requires specialist interface ring shear tests and careful consideration of clay Yield Stress Ratios (YSR) and Sensitivities (St) determined from an appropriate site investigation in order to apply the method safely. The ICP-05 clay method has been successfully applied by Shell in 14 case studies reported by Overy (2007) and in major recent oil and gas projects including the West of Shetland Clair field fleet of structures (see Hampson et al; 2017) and several jackets for EDF’s Southern North Sea Cygnus development. The ICP-05 clay approach was also applied for the Borkum West II wind-turbine project (Merritt et al 2012).

17. Based on the above discussion, the ICP-05 sand and clay methods have been considered the preferred methodology for assessing design axial pile resistance for the EA ONE project. The ICP-05 procedure provides the following advantages:

- Full compatibility between the sand and clay methods;
- Best statistical performance of pile resistance predictions against relevant load tests, so providing the best balance between economy and service reliability for any given set of WSD or LRFD factors;
- Extensive track record of application to offshore pile foundation design in the North Sea for both oil and gas and offshore wind projects
- Updated approaches for addressing cyclic loading, as set out by Jardine et al (2012)
- The linked research into the beneficial effects of pile ageing, as identified by Jardine et al (2006)

2 Site Investigation Works

18. Apart from a reconnaissance geotechnical site investigation (SI) carried out in 2009/2010 which included a few boreholes within the EAONE site, the final SI strategy was based on 2 separate campaigns, the first taking place in 2014 prior to concept and FEED design (41 seabed CPTs and 24 boreholes within the wind farm) and the second after FID (30 seabed CPTs and 27 boreholes) to enable the detailed design works. The first campaign also enabled a detailed laboratory testing scope to be carried and the correlation with the geophysical data resulting on the development of a detailed 3D Ground Model and the first assessment of global parameters for the identified engineering units.

19. The borehole termination depths during the 2014 SI were assessed real-time based on parameters assigned to the main layers, based on the findings from the reconnaissance survey. Although the parameters assigned were conservative enough to ensure the boreholes were carried out into sufficient depth, it is estimated that about **15% of borehole drilling/sampling/testing cost and programme** was saved on the first campaign when compared against a fixed borehole termination depth or an assessment based on the API 2A-WSD main text. The investigations costs for sites such as EAOne can be expected to range between £8M and £14M.

20. Considering the results and developments following the 2014 SI and ICP-05 assigned as the geotechnical foundation design method, a final geotechnical site investigation was deemed necessary after completion of a cost-benefit analysis and its aim was to increase the confidence in the ground conditions across the site and obtain all the necessary information to inform the design and the installation of the piled foundations, whilst complying with the requirements of the Certifying Authority.

21. Preliminary analyses of the axial pile capacity of the WTG foundation piles, based on the ICP-05 method and the global parameters developed after the 2014 SI campaign, showed a required embedment depth of 45 to 60m. As a result, all piles across EAONE were expected to extend to the Westkapelle unit since the other geological units were encountered at shallower depths, i.e. within the shaft. Based on the aforementioned design methodology 2 main drivers were identified for the design and installation of foundations, as follows:

- The engineering units within Westkapelle especially the top and the bottom of 6W1C since a pile tipping in clay develops considerably lower base resistance than a pile tipping in sand.

- The strength (q_c) of 6W1S which appears to be highly variable depending on the location of the foundation within EAONE (a very dense sand layer was indicated to be present in the north-western part of the site).

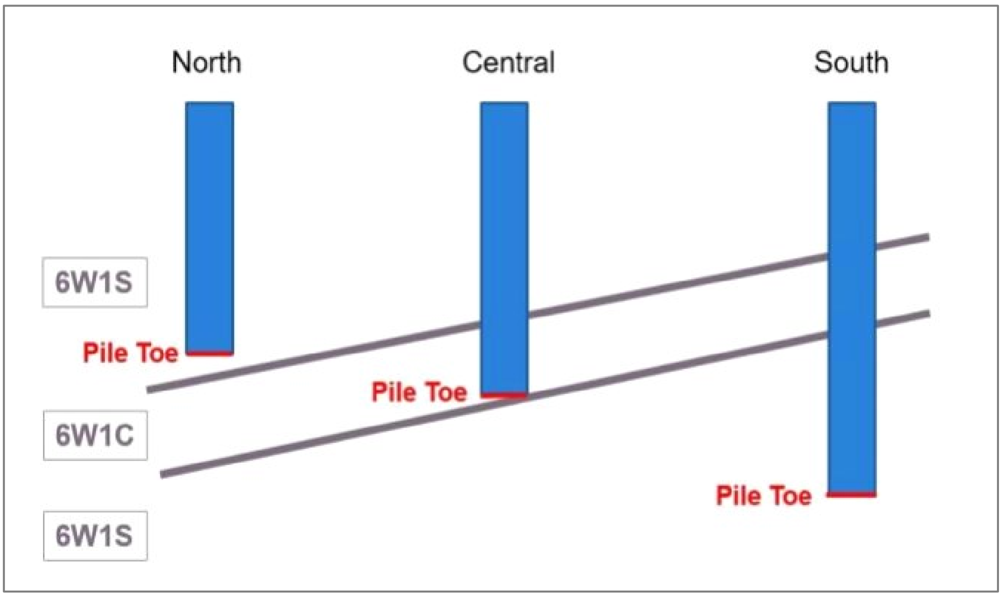

Figure 2-1: Illustration of pile tipping layer across EAONE

22. The SI was carried starting by a seabed CPT phase (20 ton thrust) at all WTG locations with no CPT or borehole data from the 2014 SI which was expected (and generally achieved) to have good penetrations (average penetration of 35m). The site geotechnical model was then updated considering the level of confidence on the Ground Model and identified top of engineering units with the CPT push, local parameters (particularly q_c for the assessment of the borehole depth based on ICP-05) and particular driving risk due to presence of very dense sands from the 6W1S engineering unit.

23. Subsequently after the completion of each borehole, started around the termination depth of the seabed CPT at the same location and terminated at average depths of 57m due to the termination depth tool with refined parameters, and comprising mainly down-the-hole CPTs with the occasional sample, the geotechnical model was updated and revisited to ensure that the new knowledge was built into the pile design. The ultimate limiting factor for the number of boreholes was to define when an additional borehole would become more expensive than the potential additional steel or the increased driving risk of not having data to depth at a given location. Further information on this approach can be found on Adamopoulos et al. (2017).

24. This method allying the 3D Ground Model to a refined and appropriate geotechnical design method such as the ICP-05, together with strong collaboration between all parties, enable the project to save an estimated 20% on the full costs of the second campaign when compared against a full coverage of boreholes throughout the WTG locations and extension to depths based on API main text method.

3 Cyclic Degradation Assessment

25. With the confirmation of using the ICP-05 as the geotechnical design method for the EA1 foundation piles, a cyclic testing and degradation assessment strategy was also developed as a joint effort between Fugro, ICL, OWC and SPR and subsequently adopted by Ramboll. The approach followed was the ICP methodology asset out by Jardine et al (2012) and Aghakouchak et al (2015). The work was undertaken in conjunction with Professor Jardine.

26. Given the type of foundations planned for EAONE, the majority of the cyclic testing was performed using Cyclic Direct Simple Shear (CSS) tests. Cyclic Triaxial (CTXL) tests were scheduled to provide comparison to the measured simple shear response and ensure that the results of the simple shear tests were consistent with general expectation of the dense sand

soils being tested. The sample orientation was defined to replicate as reliably as possible the stress conditions in a soil element adjacent to a pile shaft as illustrated on **Figure 3-1**.

27. To allow interpretation of the test in the context of the cyclic model, the maximum consolidation stresses within the sand units were calculated to represent the maximum effective radial stress that might be expected to be generated at each weighted unit depth during driving of an open-ended pile with the dimensions of the EAONE piles. These effective radial stresses were calculated according to the ICP-05 design method for equalised effective radial stresses at the pile wall, but with the parameter h, which represents the tip depth of the pile relative to the soil depth, fixed to a value of 0.5 m. Cyclic pre-shearing was applied, under constant normal stress conditions and in a stress-controlled manner, to replicate the effect of cyclic action during pile driving. Pre-shearing has been shown to increase the cyclic shear strength of sands and silts (Andersen, 2009) and was expected to reduce the impact of sample reconstitution. Creep intervals were applied on preloading (or preconsolidation) and following pre-shearing or following application of average shear stresses.

28. The consolidation stress regime for the clay samples was specified to broadly replicate the expected horizontal stress history of the clay unit, allowing for the effect of pile installation on stress conditions in the soil adjacent to the pile shaft. The consolidation regime involved initially consolidating the test specimen under a normal stress which is approximately equal to the estimated maximum horizontal effective stress to which the clay is expected to have been subjected during their entire geological history. The test specimen was then unloaded to a normal stress equal to the estimated in situ horizontal effective stress, calculated based on available in situ and laboratory data and the sample depth below seafloor. Lastly the specimen was reconsolidated under a normal stress approximately equal to the expected radial effective stress adjacent to a pile shaft (Jardine et al., 2005) following installation by driving.

29. Further details on the test specification can be found on Rattley et al. (2017).

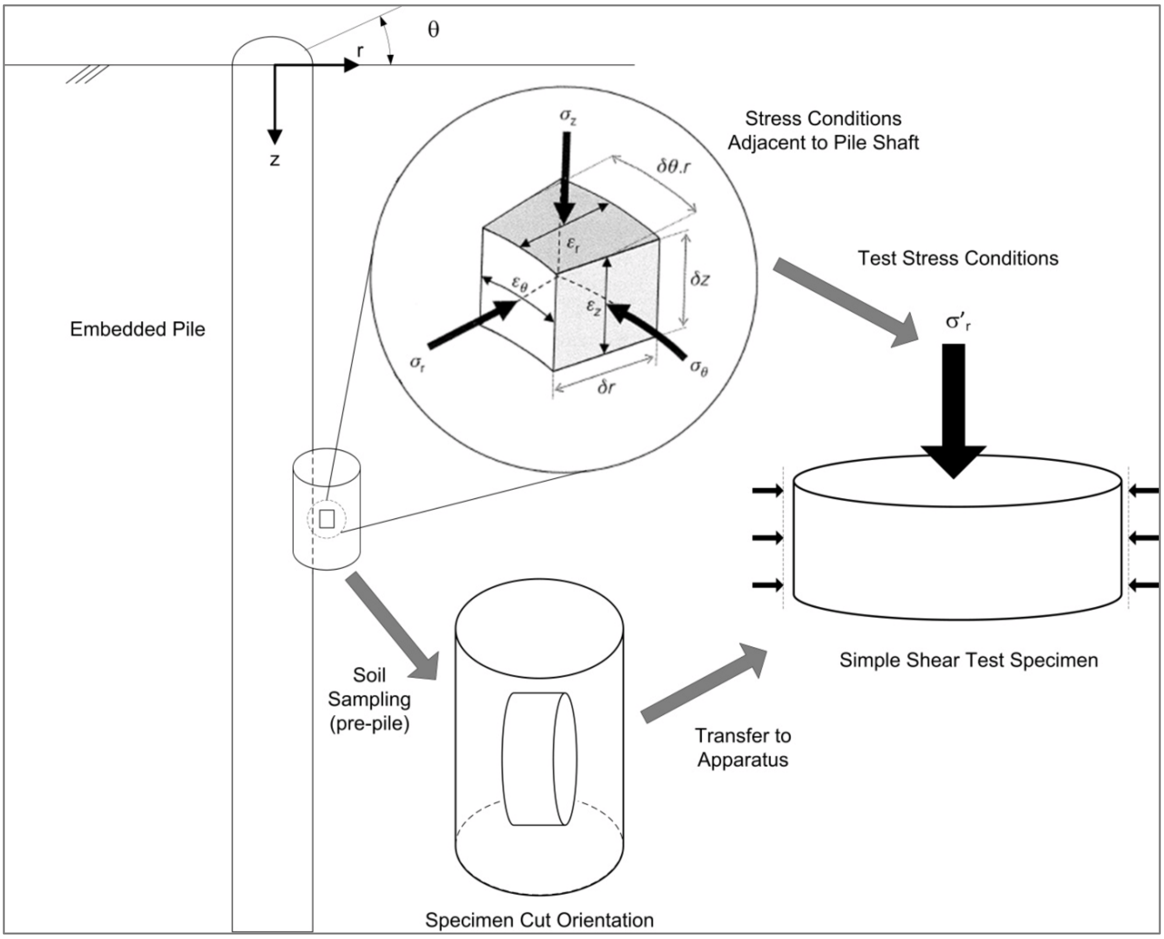

Figure 3-1: Schematic of stress conditions in a soil element adjacent to a pile shaft as translated in the direct simple shear test (see also Sim et al., 2013)

30. Following the successful execution of the cyclic testing (split between the 2014 and the 2016 SI), the interpretation model was developed in line with the selected design methodology. The model was based on the assessment of the model parameters (A, B, C) from the CSS to quantify the effective stress changes expected under a range of cyclic conditions. Reduction in effective stress is expected to increase as a function of the number of applied cycles (N) for cyclic stress amplitudes above a given threshold.

31. The assumption for the Metastable cycling is that when τ_{cy}/σ' is less than $tan(\delta)$, the proportional loss of effective stress ($\Delta\sigma'/\sigma'_0$) is nearly independent of the average shear stress applied and can be expressed in terms of τ_{cy}/τ_{rzf} and number of cycles (refer to Section 9 of Jardine et al, 2005), having the A, B, C parameter defining the rate of effective stress reduction under cyclic loading.

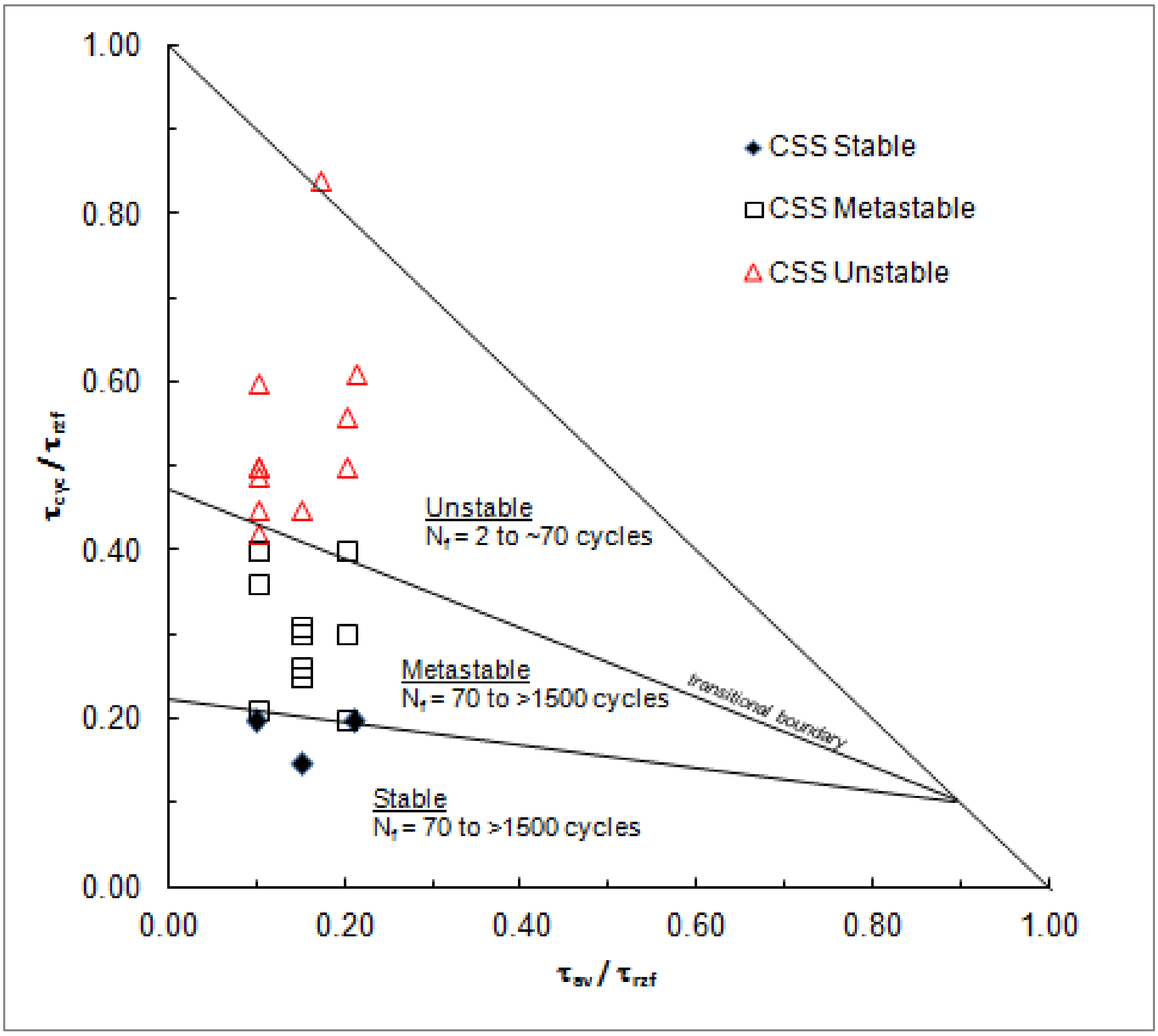

Figure 3-2: Illustration of potential cyclic stability zones in a normalised cyclic interaction diagram

32. The impact of the model developed from the CSS tests by considering τ_{cy}/τ_{rzf} as broadly analogous to the pile shaft load ratio Q_{cy}/Q_s applied to a pile driven in uniform soil. In this case Q_{cy} is the cyclic load component and Q_s is the pile shaft

resistance, and the relative loss of pile shaft resistance can be expressed as a similar function of cyclic load amplitude stated in ICP-05 and Jardine and Standing, (2012):

$$\frac{\Delta Q_s}{Q_s} = A.\left(\frac{Q_{cy}}{Q_s} - B\right).N^C$$

33. The parameters derived for each soil unit were as per Table 3-1:

Formation	Soil Type	Model Parameters		
		A	B	C
Yarmouth Roads	Sand	-0.06	0.19	0.56
Smith's Knoll	Sand	-0.03	0.20	0.64
Westkapelle	Sand	-0.07	0.18	0.61
	Clay (LP)	-0.34	0.22	0.22
	Clay (HP)	-0.34	0.34	0.22
LP = lower plasticity				
HP = higher plasticity				

Table 3-1 – Summary of cyclic model parameters derived for EA1

34. The outlined approach was applied to estimate the possible reductions in pile shaft capacity due to the axial load cycling expected for the WTG jacket piles for a synthetic 50-year, 3-hour peak storm event and typical pile design dimensions.

35. In conclusion this approach resulted in estimated values for single unit reduction in axial pile shaft capacity due to cyclic loading of less than 5% for the soil units present at EAONE. This, compared to the original 20% reduction considered during preliminary design stage following common practice, is estimated to have been translated into a reduction on average pile length of about 5%.

4 Steel Order

36. The steel order has mainly been directly affected by the application of the ICP-05 design methodology when compared to the use of the API main text and the improvement on the cyclic degradation assessment.

37. Following the same line of thought as outlined in Section 2, the estimated benefits of the use of the methodology on the pile length are around 15% and the cyclic degradation assessment has added an about 5% further improvement on the pile length.

38. Considering approximate pile steel supply and fabrication costs within the offshore wind industry of around £3.500 / m and the average reduction on pile length of about 10m (20%) when compared to the API main text design methodology, it is estimated that savings in the order of **£11.0M** are achievable on projects of a similar scale to EA1. To this estimated amount the additional savings on handling and transport should be added (estimated around 2% due to reduction in pile weight and dimension, increase in transport and load-out efficiencies) which are estimated to add up to about £200k-£300k.

5 Installation

39. The reduction on pile length resulting from the use of the ICP-05 method also had a clear obvious impact on the pile installation time which can be considered as the same relative improvement as the pile length (20%). For a project a similar dimension of EA1 the potential savings resulting from reduction of driving time due to optimised pile length are estimated to be in the range of £3.0M-£5.0M. This not only has significant benefits on the direct costs (vessel and driving time) but also on the consenting as it results on less driving time and less hammer blows, which considering the increasingly demanding consenting scrutiny, is considered of very high value for the project.

40. Additionally reducing the pile length translated into less penetration of the piles into the Westkapelle very dense sands and high strength clays, minimising therefore the risk of refusal which also resulted in savings on expensive contingency measures (larger hammers or drilling tools).

6 Conclusion and Look-Ahead

41. After completing the successful installation of all the EAONE piles (WTG and OSS) it is considered that the use of the ICP-05 design methodology played a very significant part in its success. The 10 dynamic load tests performed during WTG pile installation, with an approximately 1-day restrike, proven that the required capacity has been achieved and also indicated that further potential additional reductions might be possible when considering the ageing and set-up effects in the soil identified by Jardine et al (2006) in their field tests in Dunkirk.

42. In summary the ICP-05 method is considered highly reliable for North Sea sands and clays and will be continuously considered where relevant on future offshore developments within the Iberdrola offshore renewables ever growing portfolio.

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