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Considerations for a Reliability-based Foundations Design in Varying Rock Mass conditions – A Review from the Saint Brieuc Offshore Windfarm

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ABSTRACT: The St. Brieuc Offshore Windfarm site is located within the Massif Armoricain comprising shallow Precambrian rock subject to varying degrees of weathering. The Wind Turbine Generator (WTG) foundations consist of jacket substructures supported by drilled and grouted piles. The main foundation design risks were related to 1) defining a design method to reliably account for the rock mass conditions, 2) accounting for the ground variability considering the presence of faulting zones and a dyke swarm and, 3) accounting for the implications of the adopted installation method into the pile load capacity. To assess the design reliability, the equivalent working stress design factors of safety were assessed to envelope representative scenarios that could arise from the uncertainties associated to these design risks. This study was focused on confirming that in all the foreseeable scenarios, the resulting safety margins would be within acceptable limits. The outcome of this study was crucial during the pile installation phase to substantiate the decisions offshore as the drilling performance was subject to the varying ground conditions with occasional requirement for additional design substantiation work to confirm that the as-built conditions were compliant with the design requirements. Finally, the as-built records on the seabed conditions and drilling activities were assessed and compared against the site ground model to conclude whether the aimed foundation design reliability levels met the target threshold. The findings described in this paper are considered of interest to similar industry projects working towards reducing the Levelised Cost of Energy of offshore Windfarms.

Keywords: Rock mechanics, offshore geotechnics, reliability-based design, drill and grouted piles.

1 GEOLOGICAL SETTING

The site geology is dominated by Neoproterozoic and Lower Palaeozoic rocks of the Cadomian orogenic cycle that occurred approximately between 700My and 540 My. At this stage, the rock sediments were firstly deposited and then heterogeneously deformed by convergent tectonism and transformed by metamorphism. Subsequently, the Hercynian orogeny occurred between 360My and 300My and deformed the rock mass and caused intrusion of igneous dykes into the country rock formations. The alignment of these dykes is predominantly northwest-southeast. Thereafter, the bedrock was exposed to atmospheric conditions and intense meteoric weathering.

The Neoproterozoic rock has been identified as a weakly metamorphosed psammitic rock combined with pelitic horizons, namely Brioverian turbidite, submarine fan deposits. From petrographic analysis, some samples possess tectonic slaty cleavage, tectonic fractures infilled with dolomite and chlorite, and crenulation cleavages, which are indicative of early onset or low-grade metamorphism.

2 ASSESSMENT BASIS

2.1 Design approach

There is limited industry experience in defining a methodology for designing offshore drilled and grouted piles in rock strata, as it is largely dependent on the rock mass conditions and the installation method. The following key ground risk mitigations were implemented to substantiate the pile design:

1) Site data integration studies of detailed geophysical and geotechnical surveys to build a three-dimensional ground model.
2) Definition of a Rock Mass Rating to characterise the varying degree of weathering, fracturing and discontinuities of the rock mass.
3) An onshore pile load test campaign.
4) In-situ drill tests to verify the capabilities of prototype drills.
5) Probabilistic analysis of the geo-mechanical parameters of the terrain units.

2.2 Ground variability

The site ground conditions are highly variable. There are various rock types including Psammites, Pelites, Sandstone, Mudstone, Quartzite and Dolerite in various weathering states, from completely weathered residual soil to fresh rock. Superficial deposits of sands and clays overlay the rock with thicknesses to up to 28m. The site data integration study carried out by Atkins (2018) was focused to:

C1) Delineate a dyke swarm.
C2) Identify faults and shear zones.
C3) Establish stratigraphy to identify depths to superficial deposits, residual soils, rockhead and fresh rock.
C4) Provide an interpretation of the bedrock lithology and geological engineering units.

In addition, the datasets for each WTG location were qualified with a data quality statement together with confidence levels to the interpretations of items C1 to C4 categorised from High, Medium, Low to Very low confidence.

2.3 Installation process

The pile installation comprised the use of a seabed piling template to simultaneously drill a set of three 3100mm diameter sockets with the option to underream the sockets with an extra width of 25mm. The drill tool was equipped with a recoverable casing to prevent from socket collapse while drilling and grouting. The casings were operated by oscillators mounted on a seabed piling template.

The drilling records such as the Weight on Bit, Drill Rotary Speed and Torque were continuously monitored during the operations. These parameters were subsequently applied to derive the Mechanical Specific Energy (MSE) (Teale, 1965 and Jones et al., 2023) for making correlations with the design geo mechanical parameters. The main uncertainties related to the pile design were:

1) Socket surface condition to comply with design assumptions on the socket roughness.
2) Socket stability and susceptibility to overbreak, with detrimental effect to the grouting integrity.
3) Susceptibility to rock smearing.

3 METHODOLOGY

3.1 Site data assessments

The site conditions were characterised via site integration studies from two main site investigation phases (Atkins, 2018). The borehole data comprised rock coring to varying depths from 20m to 57m below seabed. These depths were established in situ based on continuous feedback from the vessel laboratory to identify the depth to rockhead and degree of weathering as the core runs were recovered on board. There was typically one borehole per WTG location, with additional PCPTs to confirm superficial seabed conditions. The geotechnical data was integrated together with high-resolution seismic reflection and refraction datasets, to substantiate the understanding on the spatial distribution of rockhead and mapping of geo hazards.

3.1.1 Ground model rating

The confidence levels C1 to C4 defined in section 2.2 were assigned a risk rating (R_L) and a weight factor (W_f) for deriving an overall Ground Model Confidence Rating (GMR):

$$GMR = 100 - \Sigma\, [R_L]_{Ci} \times [W_f]_{Ci} \qquad (1)$$

$(R_L)_{Ci}$ ranged from 0 to 1 in correlation with the confidence from high to very low levels for each item Ci = 1 to 4 accordingly. The weight factors $(W_f)_{Ci}$ were applied from engineering judgement to account for the implications of the item Ci into the pile design.

From an initial assessment of the GMR across all the WTG locations, there were seven of them that were relocated to up to 40m away from its original locations to avoid proximity to dykes and faulted zones and improve their GMR for the pile design phase.

3.1.2 Rock Mass Rating

The rock mass rating (RMR) system was defined following Bieniawski's (1989) classification system and core logging in line with ISO 14689 (2017), based on categorising the following key parameters:

- Uniaxial compressive strength of the rock core samples.
- Rock quality designation (RQD).
- Spacing of discontinuities.
- Condition of discontinuities.
- Orientation of discontinuities, adopting a rating to account for the discontinuities effect in relation to the stress path of principal stresses from axial loads (Zhang, 2017) and accounting for greater risk of overbreak for rock masses presenting increased degree of sub-vertical fissures.

The GMR typically varied from 50% to 100% across the WTG locations and based on engineering judgement, these were subsequently correlated with a ground model factor (γ_{GMR}) varying from 1.0 to 1.1, for the purposes of assessing the sensitivity of these effects into the design margins on pile lengths.

3.1.3 Geotechnical clusters

Using the ground model terrain unit maps (Atkins, 2018), the WTG locations that were subject to similar GMR conditions were classified to be within the same geo-cluster. The site conditions from each WTG location of the same geo-cluster were then reviewed to confirm consistency of geo-mechanical parameters and RMR. The data correlation per geo-cluster was useful for assessing the existence of potential unforeseen conditions, such as, spatial varying conditions of rock weathering or localised zones presenting higher degree of fracturing, inherent of the site wide geological conditions, but possibly omitted from a single borehole data per WTG location. E.g. to account for the case that a borehole at a WTG location logged the mechanical parameters within the upper bound, but not representative of the conditions to account for within the whole jacket footprint.

As part of the design basis, the selection of the design rock mass Young's Modulus for each WTG location (Em_{WTG}) was based on engineering judgement considering the GMR and the WTG site specific geo-mechanical rock properties.

In addition, based on the Bayesian analysis approach (Phoon and Ching, 2015) for updating geotechnical parameters, a geo-cluster factor (γ_{GCL}) was derived at each WTG location to account for the differences between the WTG specific data and the wider dataset from the geo-cluster.

$$\gamma_{GCL} < 0.5\ (Em_{GCL} + Em_{WTG}) / Em_{WTG} < 1 \qquad (2)$$

where Em_{GCL} is the value derived from the Normal Distribution for the lower quartile Q1, with mean μ_{GCL} and standard deviation σ_{GCL} derived from the rock mass Young's modulus of the geo-cluster dataset.

3.1.4 Socket roughness

The design method to determine the pile shaft resistance (Qs) was defined from an onshore pile load test campaign (Manceau et al., 2020) scoped to assess the upper bound rock mass conditions present across the windfarm. In addition, the Monash roughness model (Seidel, 2001) and the ICP method (Jardine et al., 2005) were used to substantiate the Qs scenarios for lower bound rock mass conditions.

The key geotechnical parameters for defining the shaft friction were the rock mass stiffness (Em) and the socket roughness. The term socket roughness refers to how the rock socket surface profile compares to an idealized cylindrical profile. The greater the asperity of the socket surface, the greater the roughness of the socket beneficial to facilitate dilation at the rock-grout interface. The adopted design roughness bounds were substantiated via assessing the available site-specific data:

- Caliper and acoustic televiewer at 8 Geobor S boreholes (Fugro, 2018).
- Multisensor core logging at 4 Geobor S core runs at representative geo units (Geotek, 2018).
- Detailed description of rock fractures for each core run according to ISO 14689 (2017).
- Downhole ROV inspections carried out at the initial phase of the pile installation campaign (Bertossa et al., 2023).
- Compressional and Shear wave logging (PS logging) for correlation of rock mass conditions across geological units (Fugro, 2018):
  - Downhole logging at 15 borehole locations.
  - Measurements from Pundit tests at the laboratory across the 62 WTG locations.

3.2 Pile axial load capacity margins

The pile design was carried out using DNVGL-ST-0126 (2021) as the governing standard and supplemented by DNVGL-RP-C212 (2019) and API RP2 GEO (2011) guidelines. The stratigraphy and geo material properties adopted as a design basis were based on engineering judgement and a qualitative assessment on the implications of level of confidence C1 to C4 corresponding to each WTG location.

The pile sizing was defined from applying the following design criteria:

- A load factor (γ_L) equal to 1.35 to account for functional and environmental loads at ultimate state limit (DNVGL, 2016).
- A reduction factor (γ_M) of 1.25 applied to the ultimate pile axial capacity (DNVGL, 2018).
- A strength reduction factor (γ_{CYC}) of 1.25 applied to account for cyclic degradation effect.
- The contribution of end bearing capacity was neglected due to the uncertainties associated with cleanliness of the pile toe at the time of grouting.

Rearranging the above factors into a working stress design factor of safety (API, 2002) including the effect of cyclic degradation for pile sizing, the resulting factor of safety as the design basis FOS_{DB} is equal to:

$$FOS_{DB} = \gamma_M\ \gamma_L\ \gamma_{CYC\,(DB)} = 2.11 \qquad (3)$$

This margin is within the range of 1.8 to 2.7 (Stacey, 2007) from offshore industry experience using WSD methods. Likewise, the recommendations by the API (2002) indicate that the pile penetrations should satisfy design and operational conditions with minimum factors of safety equal to 1.5 and 2.0 accordingly, plus additional allowance for cyclic loading effects.

3.2.1 Minimum pile depth in competent rock

The locations subject to shallow high strength rock were designed to mobilise shaft friction to up 660kPa, as they require short embedment depth to satisfy the required design capacity. Nevertheless, as a design criterion, the pile design lengths were limited to a minimum embedment depth of 18m below the seabed, i.e. approximately a minimum L/D ratio of 6, irrespective of the shaft capacity. This criterion was set to guarantee sufficient anchor length below the point of fixity as well as to account for uncertainties from the interpretation of the geophysical data on the depth to terrain units within the jacket footprint.

Likewise, in some cases, there was stratigraphy that comprised high strength rock units overlain by weak rock and residual soils. In this instance, subject to the shaft load transfer path over the pile length, the pile penetrations were set to ensure a minimum embedment depth of 5m within the higher strength rock stratum.

3.3 Sensitivity case study

The design uncertainties on the adopted pile lengths due to the ground variability aspects discussed in section 2.2 and installation effects from section 2.3, were assessed via a sensitivity analysis case (SC) adopting the following criteria:

a) Implementing model factors (CFMS, 2020) to account for possible varying ground conditions within the foundation footprint. This was derived as the combined effect of the ground model factor γ_{GMR} and the geo-cluster factor γ_{GCL} as defined in sections 3.1.1 and section 3.1.3 accordingly.

b) Adopting a revised design basis to represent best estimate load and installation conditions based on the final design and construction method reports:

   i. A modified factor $\gamma_{CYC(SC)}$=1.12 was derived from updated cyclic stability diagrams using the final design load cases.

   ii. With confirmation on the type of drill tool, comprising full-face cutters to facilitate suitable socket base conditions, and subject to the relative stiffness between the rock mass and the pile, a proportion of the axial load, in the order of 3% to 10%, was calculated to be transferred via pile end bearing (Qb).

The pile shaft component Qs was then verified to comply with the working stress design factor of safety for the SC conditions (FOS_{SC}) using equation (4):

$$FOS_{SC} = Qs/Qt \ \gamma_M \ \gamma_L \ \gamma_{CYC(SC)} \ \gamma_{GMR} \ \gamma_{GCL} \quad (4)$$

where, Qt (MN) is the total pile head load equal to the the load taken by end bearing Qb (MN) plus the shaft load resistance Qs (MN), $\gamma_{CYC(SC)}$ (-) is the cyclic degradation factor on final load conditions.

Table 3-1 summarises the adopted parameters that were applied for defining the factors of safety FOS_{DB} and FOS_{SC} derived from equations (3) and (4).

The pile sizing accross all the WTG locations were formerly designed to comply with a FOS_{DB} greater than 2.11. In addition, the pile penetrations were required to comply with a FOS_{SC} ranging from 1.7 to 2.32 subject to the model factors γ_{GMR} and γ_{GCL} and the pile end bearing capacity Qb.

The WTG locations subject to higher confidence in the ground model and presenting consistent rock mass stiffness with respect to the WTGs of the same geocluster were required a lower bound FOS_{SC} greater than 1.7. In contrast, the WTG locations subject to lower confidence in the ground model and rock mass conditions within the upper bound with respect to its geo-cluster dataset were required to comply with higher FOS_{SC} values of up to 2.32.

In addition, the locations with pile embedment depths within higher strength rock were limited to minimum lengths as described in section 3.2.1, therefore, some of these cases resulted with FOS_{SC} greater than 2.32.

Description	Design Basis (DB)	Sensitivity Analysis (SC)
Shaft resistance ratio, Qs/Qt	1	0.90 to 0.97
Load factor, γ_L	1.35	1.35
Cyclic load degradation, γ_{CYC}	1.25	1.12
Material factor, γ_M	1.25	1.25
Geo-cluster rock mass, γ_{GMR}	1.0	1.0 to 1.15
Ground model, γ_{GCL}	1.0	1.0 to 1.1
FOS	**2.11**	**1.7 to 2.32**

Table 3-1, summary of parameters that define FOS.

4 ASSESSMENT RESULTS

Figure 4-1 shows the FOS_{SC} results gathered from 39 WTG locations presenting rockhead near the seabed. The abscissa shows the range of FOS_{SC} ranging from 1.7 to 3.1. The ordinate shows average pile length over diameter (L/D) and associated average design shaft friction qs (kPa). In addition, it shows the logarithmic trendlines to illustrate the correlation between the FOS versus average L/D and qs.

The WTG locations subject to a higher degree of weathering and hence, assigned with lower bound geo mechanical parameters but with high confidence levels in the interpretation of the ground model, were typically assigned with longer pile embedment depths to satify lower FOS_{SC} greater than 1.7. In contrast, the WTG locations subject to fresh rock mass at the near seabed, hence with upper bound gemechanical parameters, but subject to greater uncertainties in the ground model were assigned with shorter pile embedment depths subject to complying with higher FOS_{SC} closer to 2.32. In addition, there were WTG locations that were assigned with FOS_{SC} up to 3.1 as these were set a minimum depth, in line with criteria described in section 3.2.1.

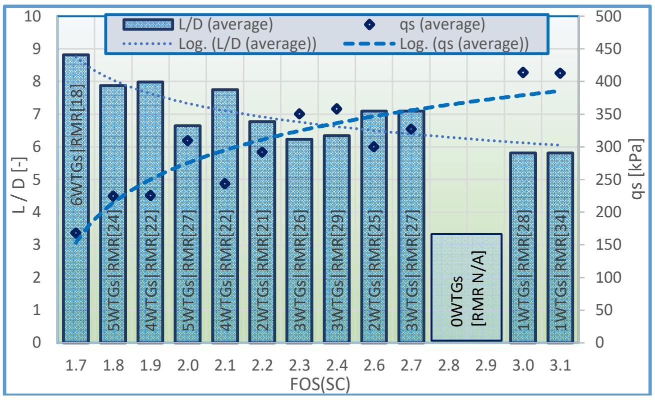

Figure 4-1, FOS_{SC} versus L/D and qs

The range of FOS_{SC} values presented in Figure 4-1 were considered appropriate to apportion equivalent safety margins with respect to potential detrimental impacts of variable ground conditions and localised installation effects causing socket overbreak. For instance, the average qs ranged from about 180kPa to 660kPa across the WTG sites. A change in rockhead level or the effect of socket overbreak affecting a nominal pile length gradient ΔL of 1m could signify reductions at gradients ΔFOS_{SC} in the order of 0.45 and 0.12 for the shortest and longest piles designs accordingly. I.e. the shorter piles would be subject to more significant reduction in the FOS_{SC} as a result of detrimental effects due to variable ground conditions or socket overbreak during the installation phase.

5 PILE INSTALLATION FEEDBACK

Figure 5-1 shows an example of the ROV seabed survey data, taken at location SB51A, that was used together with the drilling records, to verify that the as-built conditions were compliant with the design requirements.

The review of the installation records, considering the parameters described in section 2.3, reported a total of four out of the 39 WTG locations that presented adverse rock mass conditions and hence, they required further design substantiation work to comfirm that they were compliant with the design requirements. These conditions mainly comprised:

- Localized rock fall due to greater degree of fracturing of the rock mass within depths up to about 6m below the seabed. Subsequently, this effect induced unexpected drilling or dredging of loose rock falling into the rock socket.
- High torque reaction recorded in the temporary casing together with too slow drilling rates. This effect was interpreted to be caused by localised rock collapse increasing the shaft friction and rock falling into the socket, with potential risks of localised socket overbreak.

Figure 5-2 shows the MSE sample frequency recorded at location SB51A that was subject to higher torque reaction during the drilling phase. Figure 5-1 also shows the MSE sample frequency from ten WTG locations of the same geo-cluster. SB51A had assigned factors γ_{GCL} and γ_{GMR} equal to 1.1 and 1.0 respectively. However, the FOC_{SC} was equal to 2.45 based on the design limit set on minimum embedment depth of 18m in line with the criteria described in section 3.2.1. Figure 5-2 shows MSE values of SB51A within the lower bound of the MSE sample distribution derived from the geo-cluster data. Subsequently, the drilling data from SB51A was categorised with a higher residual risk to present RMR values lower than the ones adopted in the design basis together with greater suceptibility to socket overbreak.

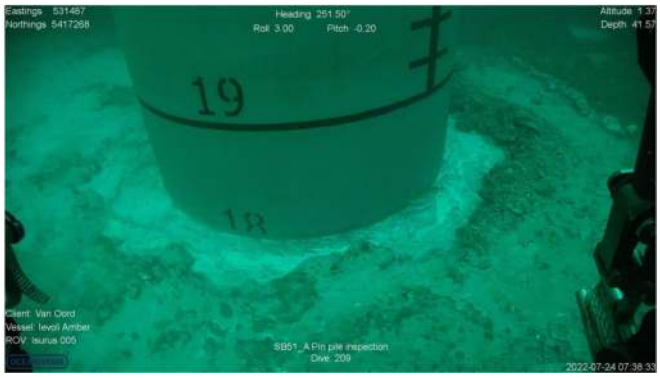

Figure 5-1, WTG_51A – as built seabed conditions

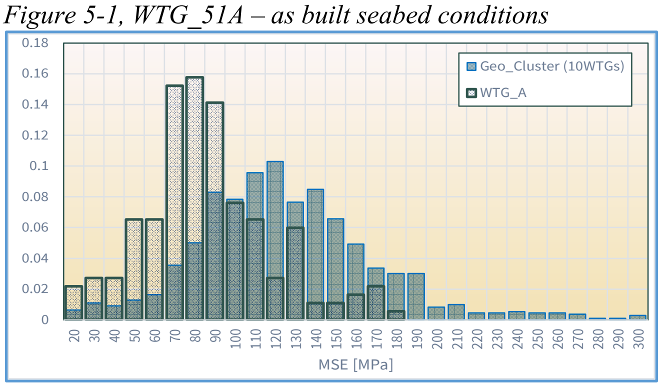

Figure 5-2, MSE sample frequency distribution [%] – WTG_51A and associated geo-cluster

Following substantiation design work by Kent (2024) comprising back analysis (BA) of RMR and Em_{WTG} to assess the impacts into the pile Qs, the resulting factor of safety (FOS_{BA}) was equal to 2.0.

Hence, it was categorised to be acceptable to meet the design requirements. In this case, the extra margin was demonstrated by the benefit of having set a minimum pile embedment length of 18m.

6 CONCLUSIONS

The pile design was originated from a detailed site data integration study with focus on characterising the varying rock mass conditions from different geological units at varying degrees of weathering.

The uncertainties from the site ground model were scored via a rating system, the GMR, to quantitatively determine the implications of these within the WTG jacket footprint. In addition, the WTG locations presenting similar geological conditions were grouped into geo-clusters for cross correlating geo-mechanical parameters and adopting consistent design basis.

The pile sizing was initially carried out in line with the DNVGL standards adopting geo mechanical parameters based on engineering judgement on a qualitative view on the site ground model confidence level and using earlier available information on the load conditions and installation methodology. In addition, the final design phase included a sensitivity study to account for a model factor to assess the implications of the ground variability together with revised assumptions on the cyclic load effects and installation method to facilitate pile end bearing.

The adopted installation methodology proved successful to install all the WTG foundations across the site. However, from a detailed review of the as built data there were reported localised effects, such as, increased friction resistance at the temporary casing with risks on socket overbreak. The drilling records were processed using the parameter MSE to review the ground conditions and carry out back analysis to verify compliance with the design requirements.

The implementation of model factors to account for the ground model and using a geo-cluster overview on the adopted design parameters, together with pile lengths to reach minimum required depths, were fundamental for implementing strategic extra design margins and be able to mitigate issues during the installation phase.

AUTHOR CONTRIBUTION STATEMENT

A. D. Bertossa: Methodology, Formal analysis, Writing – original draft. **R. McLean, L. Costa**: Resources, Supervision, Writing – review & editing.

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