The effect of geotechnical uncertainty on the preliminary design of large diameter monopiles

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ABSTRACT: The offshore wind industry has evolved over the past decade with significant technological advances. These have been driven by the need for offshore wind energy to provide cost-competitive renewable energy. The economics of offshore wind necessitates lean and efficient designs and nowhere is this more prevalent than in the turbine foundations. This paper examines the impact of geotechnical spatial variability on the design of large diameter monopiles supporting offshore wind turbines (OWTs). Monte Carlo simulation is used to examine the reliability and probability of failure of a monopile designed using a standard deterministic approach.

1. INTRODUCTION

Driven by growing concerns about climate change and the need to decarbonise our energy production, the global offshore wind market has exhibited a rapid expansion over the past decade with an average 30% increase per year since 2010 (International Energy Agency, 2019). During this period, a number of significant technological advances have reduced the levelized cost of offshore wind energy. The foundations supporting offshore wind turbine structures, offer significant scope for optimisation due to the high degree of uncertainty in the engineering design process. This is partly driven by the variability and complex behaviour of soils. Moreover, foundations can represent a large portion of the overall development cost (in the range 25-34% according to Bhattacharya 2019). The high costs of the foundations are a result of not only the high volumes of steel required but also the costs of transportation and installation which require the hire of specialist equipment and vessels. Monopile foundations, which are large diameter (ranging from 4m to 10m) steel tubes driven into the ground, represent around 80% of all offshore wind turbine foundations installed to date and are likely to continue to be the most common foundation solution for offshore wind for the next decade (Smith, 2018). As a result, there has been a substantial research effort recently aimed at optimising the design of monopiles supporting offshore wind turbines.

Geotechnical Engineers typically model the monopile soil-structure interaction with 1D Winkler beams and decoupled non-linear soil reaction springs (often referred to as p-y analysis). For preliminary geotechnical design, the loads applied on a monopile foundation can be simplified into equivalent loads and moments at the seabed level as shown in Figure 1, where H is the horizontal force, V is the vertical force, M is the overturning moment around a horizontal axis, T is the torsion moment around a vertical axis (M and H typically govern).

2. MONOPILE DESIGN CRITERIA

Monopile design requires the verification of the Ultimate (ULS), Serviceability (SLS) and Fatigue (FLS) limit states. For each limit state, there are one or more checks which need to be undertaken in order to verify the design.

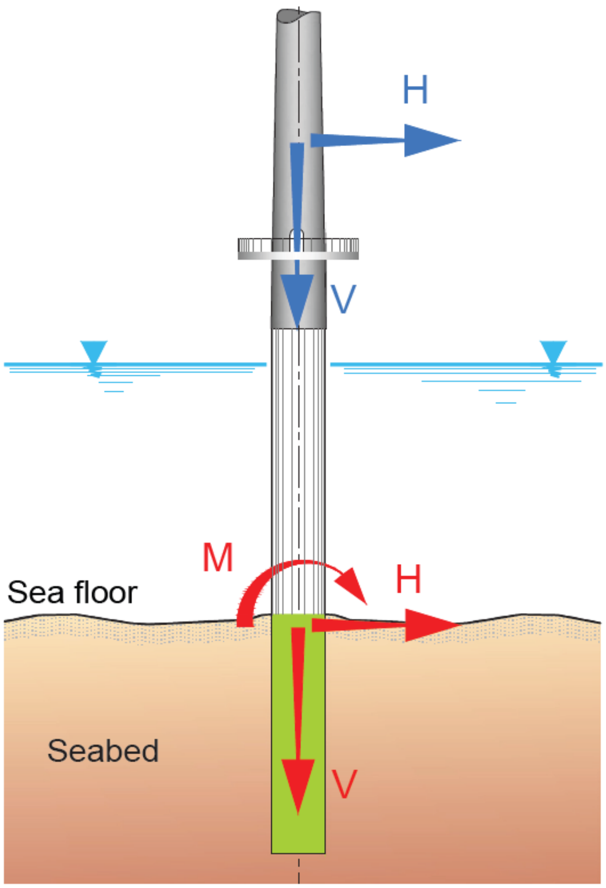

Figure 1: Schematic representation of a structure on a monopile (from CFMS 2020)

2.1. Fatigue Limit State (FLS)

The natural frequencies of monopile supported wind turbines are typically close to the frequencies of loading (swell, wind, blade rotation) and are therefore typically restricted to relatively narrow ranges. The analysis of natural frequencies and fatigue (FLS) is often the determining factor for the selection of the monopile diameter (CFMS 2020).

2.2. Serviceability Limit State (SLS)

The main SLS requirement for laterally loaded piles is that the pile deformations do not exceed some specified tolerance. For the case of monopiles, this typically involves ensuring that the permanent accumulated mudline rotation at seabed, due to the history of loads applied throughout its lifetime, does not exceed a tolerance set by the turbine manufactures. Often the total allowable rotation is taken as 0.5 degrees, with 0.25 degrees allowed for pile installation and 0.25 degrees for permanent accumulated rotation.

In this paper a simplified approach for estimating the permanent accumulated rotation, θ_{perm}, was assessed via the formula proposed by Hettler (1981):

$$\theta_{perm} = \theta_{static}(1 + t \ln N) - \theta_{elastic} \qquad (1)$$

Where θ_{static} is the rotation at seabed under the applied characteristic extreme load, $\theta_{elastic}$ is the elastic deformation found by unloading to zero along a line parallel to the initial stiffness tangent (see Figure 2), t is a parameter representing the rate of accumulation and N is the number of equivalent load cycles at the characteristic extreme load which represents the damage from a lifetime of cyclic loading. In practice, the t parameter should be determined from cyclic laboratory testing and N determined by analysing individual load packets applied to the turbine and using accumulation laws (as described in Lapastoure et al. 2022). For simplicity and based on experience, values of t=0.22 and N=15 were used in the analysis to represent cyclic accumulation.

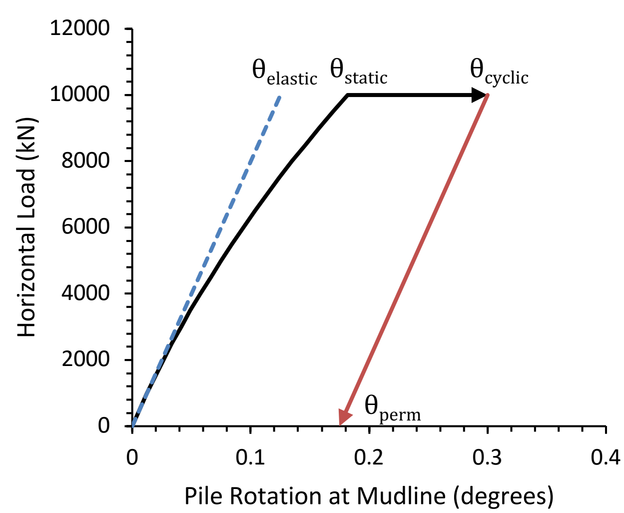

Figure 2: Method for determining SLS permanent rotation

2.3. Ultimate Limit State (ULS)

Due to the small rotations allowed by the wind turbine manufacturers, horizontal and vertical loading can be separately verified against axial and lateral capacity. Monopile axial capacity tends to be several times higher than the vertical design loads, and therefore axial ULS checks do not govern the design. Horizontal and moment loading typically govern the ULS geotechnical checks. Current design standards for offshore monopiles (DNV 2021a,b) require that for combined lateral loading and moment loading in ULS sufficient pile capacity shall be ensured with two requirements (DNV 2021a):

ULS1. *"The theoretical design total lateral pile resistance, which is found by vectorial integration of the design lateral resistance over the length of the pile, shall not be less than the design lateral load applied at the pile head."*

ULS2. *"The lateral displacement at the pile head shall not exceed some specified limit which is defined in the corresponding design basis for example. The lateral displacement shall be calculated for the design lateral load and moment in conjunction with characteristic values of the soil resistance and soil stiffness*."

The second requirement (ULS2) is needed as laterally loaded piles are unable to mobilise the full soil resistance along its length, as there will be points along the pile where the deflection is reversed or close to zero. ULS2 typically involves undertaking a p-y analysis using design (factored) loads but characteristic (unfactored) soil parameters and ensuring the seabed rotation does not exceed some specified value (often taken as 1.5 degrees). In some cases, designers may choose to undertake additional p-y analysis instead of calculating ULS1. In German waters, the ULS1 verification is undertaken in accordance in EA-Pfahle (German Geotechnical Society, 2013) and DIN 4085, where the design value of the utilized integrals of the embedment force (soil reaction force) $B_{h,d}$ must be smaller than the design value of the spatial soil resistance evolving in front of the pile $E_{ph,d}$ down to the point of rotation (point of zero deflection, z_{rot}) according to:

$$B_{h,k} \times \gamma_G < E^r_{ph,k}/\gamma_{Ep} \quad (2)$$

where:

$$E^r_{ph,k} = D^{Er}_{pg} \int_0^{z_{rot}} K_{pgh}.\gamma.z.dz + c.z_{rot}.D^{Er}_{pc} \quad (3)$$

$$K_{pgh} = \sqrt{\frac{\left(\left(\frac{1+sin\varphi'_k}{1-sin\varphi'_k}\right)\left(1-0.53\delta_{p,k}\right)^{0.26} + 5.96.\varphi'_k\right)^2}{1+\left(tan\delta_{p,k}\right)2}} \quad (4)$$

Where φ'_k and $\delta_{p,k}$ are the characteristic values of the soil internal friction angle and the soil-steel interface friction angles (in radians) respectively. D^{Er}_{pg} and D^{Er}_{pc} are defined in EA-Pfahle (German Geotechnical Society, 2013). In this check, the partial factors $y_G = 1.5$ and $y_{Ep} = 1.4$ are applied to the reaction force and the soil resistance, respectively.

3. ANALYSIS INPUTS

In order to assess the reliability and probability of failure accounting for Geotechnical uncertainty, a simple monopile design case was developed. The monopile pile length was designed assuming standard approach using deterministic input parameters. Once the pile length was determined deterministically, the pile geometry was then analysed using Monte Carlo simulations to assess the reliability and probability of failure in both SLS and ULS. In order to be applicable to modern wind turbines, seabed design loads were estimated based on the NREL 15MW reference turbine, in 30m water depth, including hydrodynamic loading. A pile diameter of 8m was assumed and a single wall thickness ratio, D/t, of 120 was used (t=67mm) throughout the analyses. The analyses were undertaken using a 1D Winkler beam model implemented in Matlab. The soil reaction curves were implemented using the PISA rule-based method for sand (Burd et al. 2017, Burd et al. 2020) which requires soil unit weight, γ, relative density, D_r, and small-strain shear modulus, G_0, as inputs. A saturated soil unit weight of 20 kN/m^3 was used for all analysis cases.

3.1. Deterministic inputs

The estimated loads used in the deterministic analysis are provided in Table 1. For the deterministic soil input parameters, a dense sand with a constant characteristic relative density $D_{r,k}$ = 75% was used. G_0, was estimated based on relative density using the formula proposed by Brinkgreve (2010) as shown in eq. (5):

$$G_0 = (60 + 68 \times D_r)\left(\frac{\sigma'_{v0}}{0.1}\right)^{0.5} \text{ in MPa} \quad (5)$$

The value of the soil internal friction angle, used in the ULS1 (DIN) check, was determined using the correlation with relative density proposed by Bolton 1986:

$$\varphi' = \varphi'_{cv} + 3\big(D_r\big(10 - ln(p')\big) - 1\big) \quad (6)$$

$$\delta_p = 0.67 \times \varphi' \quad \text{in cohesionless soils} \quad (7)$$

where p' is the mean effective stress at failure (assumed equal to the in-situ vertical effective stress in this analysis for simplicity).

Table 1: Estimated monopile seabed loads for 15MW turbine

Characteristic (unfactored)	H	10	MN
	M	600	MNm
Design (factored)	H	13.5	MN
	M	810	MNm

3.2. Monte Carlo simulation inputs

3.2.1. Soil Inputs

In this paper geotechnical spatial variability is considered through spatially varying Cone Penetration Test (CPT) cone resistance profiles developed using the random field approach. The CPT profiles were assumed to be normally distributed requiring, three parameters to define the random field, namely, the mean, standard deviation, and scale of fluctuation (θ).

To develop the stochastic CPT profiles, a mean CPT cone resistance profile, $\overline{q_t}$ (corrected for pore pressure effects), was derived based on a constant relative density by rearranging the correlation proposed by Kulhawy and Mayne (1990):

$$D_r = \sqrt{\frac{Q_{tn}}{305 \cdot OCR^{0.15}}} \quad (8)$$

$$Q_{tn} = \left(\frac{q_t - \sigma_{v0}}{p_{ref}}\right) \cdot \left(\frac{p_{ref}}{\sigma'_{v0}}\right)^n \quad (9)$$

where p_{ref} is a reference stress (atmospheric pressure) taken as ≈100 kPa, σ_{v0} is the in-situ vertical total stress, σ'_{v0} is the in-situ vertical effective stress and n is a stress exponent which varies between 0 and 1 depending on soil type and stress level. Robertson and Cabal (2014) proposed the following equations for n:

$$n = 0.381(I_c) + 0.05\left(\frac{\sigma'_{v0}}{p_{ref}}\right) - 0.15 \le 1.0 \quad (10)$$

A constant I_c value of 1.8 has been assumed in this paper for clean sand (Robertson and Cabal 2014). In the deterministic analysis, a characteristic relative density $D_{r,k} = 75\%$ was assumed. The characteristic value is assumed to be based on a mean with confidence of 95% as is typical for problems which involve large soil volumes where local strength variations from point to point can be assumed to average out. The mean value of the distribution of relative density, $\overline{D_r}$, is calculated from as:

$$\overline{D_r} = D_{r,k} + 1.68\frac{\sigma}{\sqrt{N_{samples}}} \quad (11)$$

Where σ is the standard deviation and $N_{samples}$ is the number of sample points assumed = 50 (i.e. large number of samples). A coefficient of variation on the relative density of 30% was assumed resulting in a mean value of $\overline{D_r} \approx 80\%$ which was the value used in generate the mean

CPT cone resistance profile, $\overline{q_t}$. A scale of fluctuation value of $\theta = 1m$ was assumed in this analysis. A correlation matrix can be developed using the Markov correlation structure:

$$\rho(\tau_j) = \exp\left(\frac{-2|\tau_j|}{\theta}\right) \tag{12}$$

where j = 0,1,..., n–1 with n being the number of data points, $\tau_j = j\Delta\tau$ is the lag distance between the two points and $\boldsymbol{\rho}$ is the correlation matrix. The correlation matrix is positive definite and so can be decomposed into upper ($\mathbf{L^T}$) and lower ($\mathbf{L}$) triangular forms using Cholesky decomposition:

$$\boldsymbol{\rho} = \boldsymbol{L}\,\boldsymbol{L^T} \tag{13}$$

The spatially correlated normal random field, G, can then be obtained through multiplying the lower triangular matrix with a matrix of independent normal random numbers with zero mean and unit standard deviation, U:

$$\boldsymbol{G} = \boldsymbol{L}\,\boldsymbol{U} \tag{14}$$

By scaling the normal random field G to the correct dimension using the mean and standard deviation the stochastic CPT profiles are generated as:

$$\boldsymbol{q_t} = \overline{q_t} + \sigma\boldsymbol{G} \tag{15}$$

A coefficient of variation on the corrected cone resistance of 30% was assumed (i.e. $\sigma = 0.3\overline{q_t}$).

3.2.2. *Load Inputs*

For the deterministic analysis, a characteristic extreme horizontal load of $H_k = 10MN$ was assumed along with a characteristic overturning moment at seabed $M_k = 600MNm$. For the Monte Carlo simulation a stochastic load distribution was developed based on a Gumbel distribution (for extreme loads), requiring inputs for the mean and standard deviation of the loads. For this paper a standard deviation of 15% of the characteristic load (i.e. 1.5 MN) was deemed representative for North Sea conditions. EN1990 suggests that a characteristic value for an environmental load should have a 2% probability of exceedance. The mean of the Gumbel distribution was then calculated as:

$$\overline{H} = \frac{H_k}{1.35} - \frac{0.577\sigma\sqrt{6}}{\pi} - \frac{\sigma\sqrt{6}}{\pi} ln\{-ln\Phi(-0.7\beta)\} \tag{16}$$

where β is the target reliability index (taken as 2.9 for SLS) and Φ is the cumulative standard normal distribution function. The value of $\overline{H}$ used in this calculation is 5739 kN. The distribution of loads used for the Monte-Carlo simulation is shown in Figure 3. A fixed ration of seabed moment to horizontal of 60 was assumed for this analysis.

4. ANALYSIS AND DISCUSSION

4.1. Deterministic analysis

The required pile length was calculated from the deterministic analysis. The utilisation for both the SLS and ULS2 checks are plotted in Figure 4. It can be seen from this check that based on the SLS criteria a pile length of at least 34m is required. At this length the ULS1 DIN utilisation is 27% and the ULS2 utilisation ($\theta_{ULS}/1.5°$) is 29%, indicating clearly that SLS governs the design.

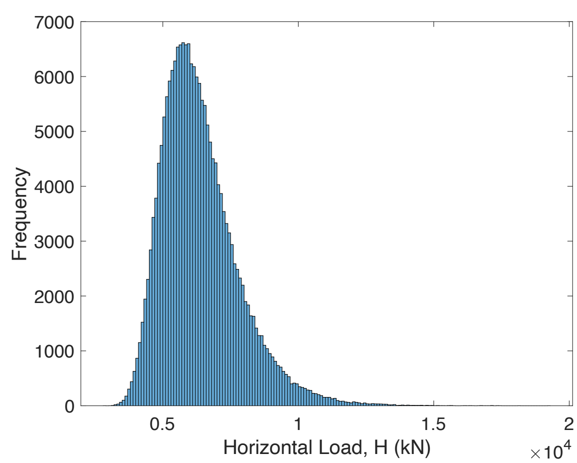

Figure 3: Distribution of stochastic extreme loads

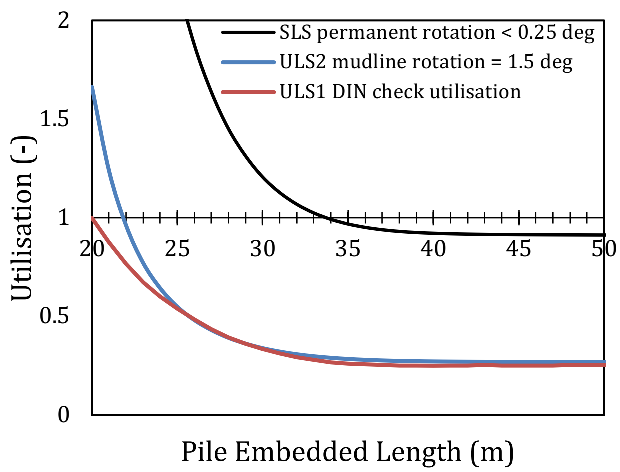

Figure 4: Pile length vs utilisation in SLS and ULS using deterministic approach.

4.2. Monte-Carlo Simulation

The reliability index and probability of failure in this paper is estimated using Monte-Carlo simulation. For each realization, a load and CPT profile is randomly picked from a sequence. The IEC 61400-1 (IEC, 2009) recommends a ULS target reliability level for wind turbines with a maximum annual probability of failure, $p_{f,ULS} = 5 \times 10^{-4}$ (corresponding to a reliability index of β_{ULS} =4.7). The IEC61400-1 does not specify a target reliability level for SLS but Eurocode EN1990 suggests an SLS annual target reliability index of $\beta_{SLS} = 2.9$. Using Monte-Carlo simulation more than 100/p_f realisations may be required to accurately estimate the probability of failure. For this reason, 200,000 realisations were used for this analysis. If $\boldsymbol{X}$ is the set of input stochastic variable, then the limit state functions $g(\boldsymbol{X})$ for SLS and ULS can then be written as:

$$g_{SLS}(\boldsymbol{X}) = 0.25° - \theta_{perm}(\boldsymbol{X}) \quad (17)$$

$$g_{ULS1}(\boldsymbol{X}) = 1 - \frac{B_{h,k}(\boldsymbol{X}) \times \gamma_G}{E^{r}_{ph,k}(\boldsymbol{X})/\gamma_{Ep}} \quad (18)$$

$$g_{ULS2}(\boldsymbol{X}) = 1 - \frac{\theta_{static,ULS}(\boldsymbol{X})}{1.5°} \quad (19)$$

The probability of failure can be calculated from N_f / N where N_f is the number of realisations which fail and N is the total number of realisations. For the SLS limit state, 2987 of the 200,000 realisations failed leaving a probability of failure of 0.0149. For both ULS1 and ULS2 limit states, 0 of the 200,000 realisations failed the limit state. Once the probability of failure is calculated, the reliability index β can be estimated by taking the negative inverse standard normal distribution of the probability of failure:

$$\beta = -\Phi^{-1}(p_f) \quad (20)$$

Based on the SLS probability of failure, the reliability index achieved was β_{SLS} =2.173 which was lower than the target reliability index of 2.9 for SLS. An alternative estimate of the reliability index, β, for each limit state check can be made by fitting a normal distribution to the limit state function $g(\boldsymbol{X})$. β can then be estimated from fitted mean, μ_g, and standard deviations, σ_g, as follows:

$$\beta = \frac{\mu_g}{\sigma_g} \quad (21)$$

Fitted values and estimated reliability indexes are shown in Table 2. The estimated reliability index in ULS is seen to be greater than 19. The reliability index estimated using the normal distribution fitting approach is seen to be $\beta_{SLS} = 2.98$ which is in excess of the target reliability in SLS, contrary to the measured probability of failure. Figures 5, 6 and 7 show the normal distribution fitting to the data, which highlights the limit state functions are not normally distributed and therefore the poor fitting may explain the deviation in β_{SLS} values.

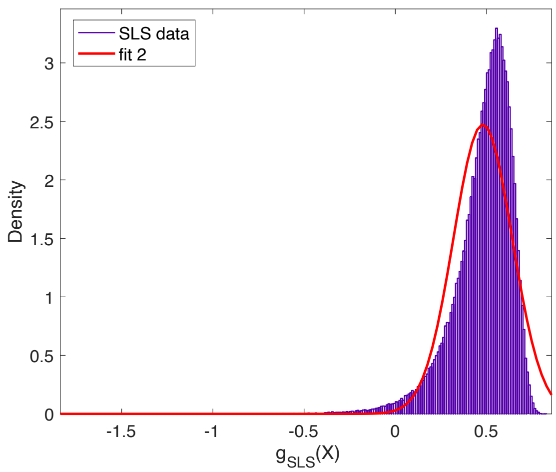

Figure 5: Normal Distribution fit to $g_{SLS}(\boldsymbol{X})$

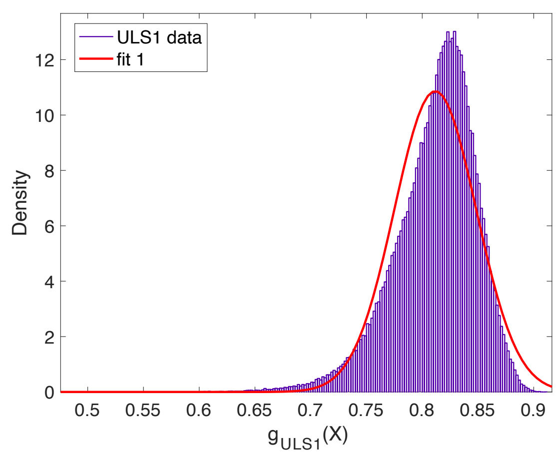

Figure 6: Normal Distribution fit to $g_{ULS1}(\boldsymbol{X})$

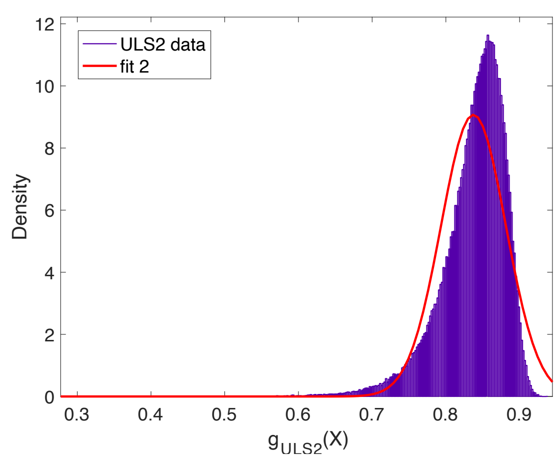

Figure 7: Normal Distribution fit to $g_{ULS2}(\boldsymbol{X})$

Table 2: Normal Distribution fitting parameter to g(X) and estimated reliability index from distribution

SLS	mean	Standard deviation	Reliability index
SLS	0.4799	0.1613	2.98
ULS1	0.8117	0.0367	22.10
ULS2	0.8374	0.0439	19.04

5. CONCLUSIONS

This paper examines the impact of geotechnical spatial variability on the design of large diameter monopiles supporting offshore wind turbines (OWTs). Monte Carlo simulation is used to examine the reliability and probability of failure of a monopile designed using a standard deterministic approach. 200,000 Monte-Carlo simulation realisations were performed. The method for determining the reliability index was seen to have a big impact on the calculated reliability index in SLS. The reliability index in ULS was far in excess of target requirements. Future work will focus on introducing model uncertainty and will use more advanced methods for cyclic rotation accumulation including full load histories.

6. ACKNOWLEDGEMENTS

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7. REFERENCES

Bolton, M. D. (1986). "The strength and dilatancy of sands." *Géotechnique*, 36(1), 65–78.

Burd, H.J., Byrne, B.W., McAdam, R.A., Houlsby, G.T., Martin, C.M., Beuckelaers, W.J.A.P., Zdravkovi´c, L., Taborda, D.M.G., Potts, D.M., Jardine, R.J., Gavin, K., Doherty, P., Igoe, D., Skov Gretlund, J., Pacheco Andrade, M., Muir Wood, A. (2017). Design aspects for monopile foundations. In proceedings of TC209 Workshop - Foundation Design of Offshore Wind Structures, At Seoul, Korea, 35–44.

Burd, H.J., Taborda, D.M.G., Zdravković, L., Byrne, B.W., Houlsby, G.T., Gavin, K., Igoe, D., Jardine, R.J., Martin, C.M., McAdam, R.A., Potts, D.M., (2020), PISA Design Model for Monopiles for Offshore Wind Turbines: Application to a Dense Marine Sand, Geotechnique, 70(11), pp. 1048–1066, 10.1680/jgeot.18.p.277.

Brinkgreve, R.B.J., Engin, E., Engin, H.K. (2010). Validation of empirical formulas to derive model parameters for sands. 7th NUMGE, Trondheim, Norway.

CFMS (2020). Recommendations for planning and designing foundations of offshore wind turbines. Version 2.1

DGGT (2013). Recommendations on pilling (EA-Pfähle).

DNV (2021a). DNV-ST-0126: Support structure for wind turbines

DNV (2021b). DNV-RP-C212: Offshore soil mechanics and Geotechnical Engineering

Hettler, A. (1981). Verschiebungen starrer und elastischer Griindungskorper in Sand bei monotoner und zyklischer Belastung. Veroffentlichungen des lnstituts ftir Bodenmechanik und Felsmechanik, Universitat Karlsruhe, 1981, Heft 90

Kulhawy, F. H., and Mayne, P. W. (1990). Manual on Estimating Soil Properties for Foundation Design. EPRI-EL-6800.

IEC: IEC 61400-3 - Wind turbines Part 3: Design requirements for offshore wind turbines, International Electrotechnical Commission, 2009

Lapastoure, L., Diambra, A., Jalilvand, S., Igoe, D. (2022), Review of cyclic models for the design of monopiles supporting offshore wind turbines, Proceedings of the Geotechnical Society of Ireland Conference 2022,

Prendergast, L., & Igoe, D. (2022). Examination of the reduction in natural frequency of laterally loaded piles due to strain-dependence of soil shear modulus. Ocean Engineering. Volume 258,111614, ISSN0029-8018, https://doi.org/10.1016/j.oceaneng.2022.111614.

Robertson, P. K., and Cabal, K. (2014). Guide to Cone Penetration Testing for Geotechnical Engineering - 6th Edition.

Zorzi, G., Mankar, A., Velarde, J., Sørensen, J., Arnold, P., and Kirsch, F. (2019), Reliability analysis of offshore wind turbine foundations under lateral cyclic loading, Wind Energy Science Discussions, https://doi.org/10.5194/wes-2019-57