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Generalized P-y curves for monopile design using the PISA methodology

Courbes P-y généralisées pour la conception de monopieux avec la méthodologie PISA

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ABSTRACT: The PISA design model is rapidly becoming the standard methodology for the design of offshore monopile foundations supporting wind turbine structures. The PISA model allows designers to either derive from finite element models (FEM) or use rule-based equations, to obtain normalized parameters. Despite the clear benefits, due to the relatively high computational cost and specialist knowledge required to calibrate advanced constitutive models from specialized soil testing, FEM-based PISA calibrations are only adopted during detailed design stages. On the other hand, the rule-based approach has documented limitations namely in new provinces, for soils with distinct geological histories than those tested as well as in layered soil profiles. Since binding monopile geometry decisions are, however, made at earlier project phases, this presents a dilemma for practising engineers. This paper presents an alternative approach of the P-y curve components of the PISA method. It can used at both early stages of design, to derive the normalized parameters in layered profiles, and at detailed design stage to inform the best-fit procedure of the normalized curves. The input soil profiles may contain both coarse and fine-grained formations, either exhibiting drained or undrained behaviour. The new formulation takes into account physical principles of the soil resistance components, to define the PISA normalized parameters.

RÉSUMÉ: Le modèle de conception PISA est rapidement en train de devenir la méthodologie standard pour la conception des fondations de monopieux offshore supportant des structures d'éoliennes. Le modèle PISA permet aux ingénieurs de dériver à partir de modèles d'éléments finis (FEM) ou d'utiliser des règles pour obtenir des paramètres normalisés. Malgré les avantages clairs, en raison du coût de calcul relativement élevé et des compétences spécialisées nécessaires pour calibrer les modèles constitutifs avancés à partir d'essais de sol spécialisés, les calibrations PISA basées sur les éléments finis sont uniquement adoptées au cours des étapes de conception détaillée. D'autre part, l'approche basée sur des règles a des limites connues, notamment dans les nouvelles géographies, pour les sols ayant une histoire géologique différente de celle des sols testés, ainsi que dans les profils de sol stratifiés. Comme les décisions relatives à la géométrie des monopieux sont prises dès les premières phases du projet, cela pose un dilemme aux ingénieurs en exercice. Cet article présente une approche alternative des composantes de la courbe P-y de la méthode PISA. Elle peut être utilisée à la fois aux premiers stades de la conception, pour dériver les paramètres normalisés dans les profils en couches, et au stade de la conception détaillée pour informer la procédure de meilleur ajustement des courbes normalisées. Les profils de sol utilisés peuvent contenir des formations sableuse ou argileuse, présentant un comportement drainé ou non drainé. La nouvelle formulation prend en compte les principes physiques des composantes de la résistance du sol, pour définir les paramètres normalisés PISA.

Keywords: Offshore engineering; finite element modelling (FEM); monopiles; PISA methodology.

1 INTRODUCTION

The modelling of soil-structure interaction in pile foundations has advanced substantially in the field of offshore geotechnical engineering. The developments to tackle the technical challenges have been driven by a need to improve the existing P-y method approaches and also to integrate additional lateral soil resistance components which became relevant in larger foundation sizes. As monopile diameters increase to ranges beyond the basis of the standard method, designers optimize foundation geometries by adopting the benefits of a larger structure in all resistance components.

For detailed design of offshore monopile foundations, FEM-based methods have become the industry practice, Burd et al. (2020). Due to the complexity, time constraints and high computational costs, however, pile geometry design iterations are not typically performed within FEM software. These rather rely on 1D Beam-in-Elastic Foundation (BEF) simplifications, resorting to soil-structure interaction models consisting of non-linear interaction curves. Moreover, at earlier stages in design, solely the design of certain positions in the offshore wind farm will be FEM-based, while the remainder rely on extrapolated normalized parameters to define the soil-structure interaction models. For both these cases, normalization methods as well as rule-based parameters to determine the soil interaction curves are required. This paper presents a methodology to derive P-y curves, based on the PISA design model, proposed by Byrne et al. (2017), and supported on soil mechanics principles, to better address the design challenge of monopile foundations.

2 CURRENT METHODS

The traditional design methodologies provided in API RP 2GEO (2014) and DNVGL-ST-0126 (2018), and which can be used for pile foundations subject to lateral loading, were not developed for the large diameter structures currently adopted in the offshore wind industry. The work undertaken by the PISA group aimed to close this gap, as presented in Byrne et al. (2017), and has become the standard design methodology for monopile design. It provides a framework to both include additional lateral components of resistance but mainly a normalization approach to define soil interaction curves in normalized space with a simple mathematical formulation of a conic function. Normalization factors of the P-y curve components, as shown in Table 1, are proposed for both effective and total stress-based soil responses, typically named representative for sand (coarse-grained) and clay (fine-grained) soil materials, respectively.

Table 1. Normalization of P-y curves, Byrne et al. (2017).

Normalized variable	Clay	Sand
Lateral displacement, $\bar{\boldsymbol{v}}$	$\frac{v}{D}\frac{G_0}{S_u}$	$\frac{v}{D}\frac{G_0}{\sigma'_{vi}}$
Distributed load, $\bar{\boldsymbol{p}}$	$\frac{P}{S_u\,D}$	$\frac{P}{\sigma'_{\mathrm{vi}}\,D}$

Based on these curve normalization functions and in-situ large diameter pile load testing, Burd et al. (2019) and Byrne et al. (2019) propose a rule-based approach where normalized PISA parameters are defined for both the sand and clay frameworks, respectively. The rule-based approach, however, has been shown to perform poorly namely in new geographies, for soils with distinct geological histories from those tested as well as in layered soil profiles with relevant stiffness contrast, as shown in Burd et al. (2020), among other limitations.

3 MODEL DESCRIPTION

The proposed P-y model approach comprises an improved normalization function for the 'sand framework', a methodology to model the undrained behaviour of sand as well as a soil mechanics'-based analytical approach within the PISA framework.

3.1 Effective stress normalization

The PISA sand framework normalization functions of the p-value solely normalize to the effective stress and pile diameter. This limits its applicability to derive global PISA parameters per unit for sites where the same unit may have distinct strength parameters. More importantly, the normalization does not relate the normalized parameters to other physical quantities. The latter is due to the missing strength parameters and in situ confinement stress definition for it to be compatible with critical state soil mechanics. The proposed modification serves to complement the effective stress normalization, based on Leonards (1962), to normalize by the shear stress as follows:

$$\bar{v} = \frac{v}{D}\,\frac{G_0\,(1-sin\phi')}{\sigma'_{vi}\,sin\phi'} \quad (1)$$

$$\bar{p} = \frac{P}{D}\,\frac{(1-sin\phi')}{\sigma'_{vi}\,sin\phi'} \quad (2)$$

where ϕ' is the friction angle. The rule-based formulation in the PISA sand framework also limits its applicability for other sands than those tested in the PISA project. The main parameter selected is ill conceived since relative density (D_r) is not a good indicator of soil behaviour nor of its critical state. Different soils behave differently at the same relative density. Based on the new normalization formulation, a total stress rule-based approach, which is by definition conservative, is defined in Section 3.2. The conservativism of such an approach can be seen by comparing the results of two laterally loaded monopile PLAXIS 3D FE models, in two sand profiles, one adopting effective stress parameters and a second with maximum shear stress, both adopting the Hardening Soil Small-Strain model, according to equations (1) and (2). An additional result for an undrained modelling, adopting the NGI-ADP model, is also shown in Figure 1. The model has a pile diameter of 9m and a pile embedment of 28m. Soil parameters were derived following correlations given in the PLAXIS MoDeTo Manual (2018). The slight difference in secant stiffness is due to limitations on the constitutive model parameter definition when selecting a very low friction angle in PLAXIS. The ultimate capacity is underestimated as expected for both the total and undrained modelling, since from the constitutive model perspective, this results from the underestimation of the frictional contribution of the resistance during loading as the positive contribution of the increase in horizontal stress is not considered.

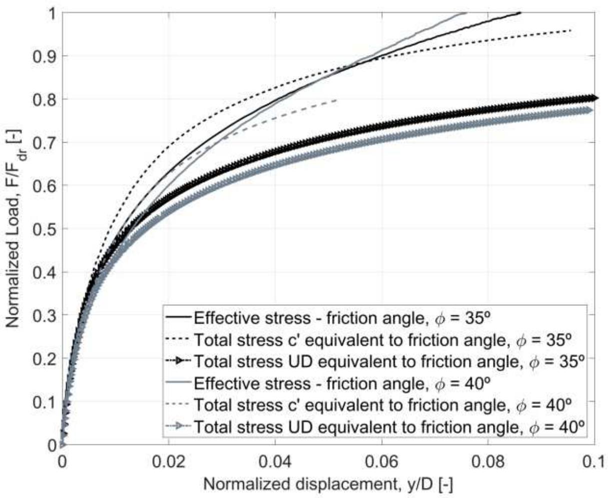

Figure 1. Comparison of 3D FEM effective, total and undrained stress models, where F_{dr} is the effective stress resistance.

3.2 Total stress / Shear stress approach

The sand framework formulation, and its respective rule-based formulation, has been found to be the least representative to be used in design. This is due to several reasons. One of the foremost is the fact that the extreme loading condition of offshore foundations is due to rapid individual loading events, such as wind and waves, for which the soil behaviour may be either undrained or partially drained, rather than fully drained, as is the basis of the PISA formulation. An assessment of the drainage type, using the dimensionless velocity ratio proposed by Finnie and Randolph (1994), indicates that the soil behaviour during the wave impact is not fully drained. Furthermore, the generation of excess pore-pressure will be greater in the vicinity of the pile, which most impacts the soil resistance components.

A pragmatic approach to extend the PISA formulation to an undrained loading of sands is to resort to the total stress approach, i.e., the PISA clay framework, where the undrained shear strength of sands is determined from triaxial extension as:

$$S_{u-sand} = \sigma'_{vi} \cdot \frac{\sin\phi' \left(1 - A_f + K_0 A_f\right)}{1 - \sin\phi' + 2 A_f \sin\phi'} \qquad (3)$$

where A_f is the Skempton pore-water pressure parameter A at failure. Janbu (1985) defines for triaxial compression tests on saturated soils the dilatancy parameter D as:

$$D = \frac{1}{3} - A_f \qquad (4)$$

where $\mathrm{D} = 0$ represents elastic behaviour, $\mathrm{D} < 0$ contractive behaviour and $\mathrm{D} > 0$ dilatant behaviour. The determination of $\mathrm{A_f}$ in sands depends on several factors namely relative density, confinement pressure and loading stress path. This parameter should ideally be determined from constant normal stiffness shear tests or directly from available undrained testing of soil samples. Care should be taken when calibrating from undrained tests since these simulate perfectly rigid boundary conditions. Hence, the increase in effective confinement stress, which in turn increases the shear stress resistance, observed in the laboratory, will not be present in situ. A possible approach to estimate the A_f parameter is through the state parameter ψ, as defined in Jefferies and Been (2006). This parameter can be either directly or indirectly determined from the cone penetration test (CPT). The standard CPT correlations, when inverted, provide a relationship to the commonly derived relative density, shown in Figure 2, for different vertical stress intervals.

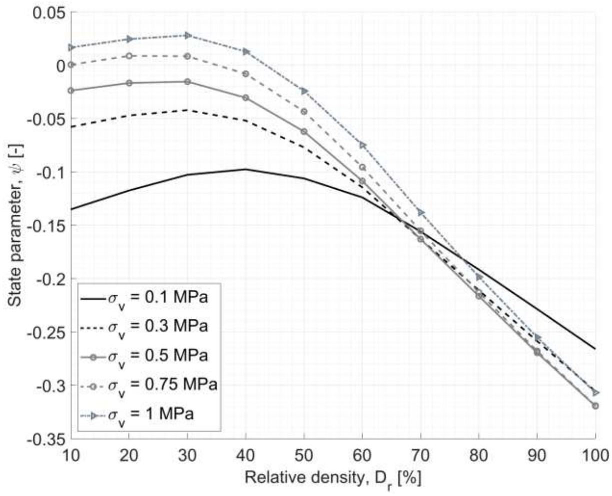

Figure 2. Inverted CPT correlations between D_r and ψ.

This approach implies that the behaviour of sands and parameter usage are in the same context as the undrained shear of clays, which is consistent with critical state soil mechanics' principles. Adopting the formulation given in Wroth (1984), the normalized ratio of both the undrained shear strength of loose sands and clays to the reference level of $\psi = 0.05$ and over-consolidation ratio (OCR) of 1.0, respectively, can be determined, as shown in Figure 3 for the stress path of isotropic triaxial compression. The starting point of both lines is so defined since, as discussed in Jefferies and Been (2006), while the state parameter and over-consolidation are different entities, the undrained shear strengths of loose sands (ψ ≈ 0.05) are comparable to normally consolidated clays (OCR=1).

In Figure 3, the red line represents the OCR effect on clay undrained shear strength, given in Wroth (1984), while the black line represents the peak friction angle relation to the state parameter in sands, as given in Jefferies and Been (2006). The different rates of variation are due to the distinct relationship between OCR and ψ to volumetric behaviour of fine- and coarse-grained materials.

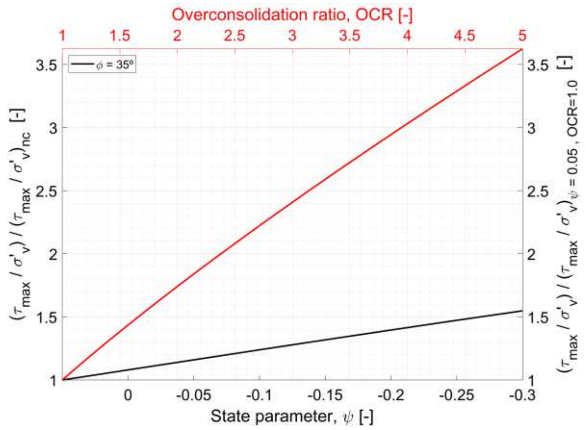

Figure 3. Variation in normalized undrained strength ratio.

This section summarized an analytical approach to determining the undrained shear strength for both fine- and coarse-grained materials for different initial stress conditions and loading histories.

4 RULE-BASED FORMULATION

The proposed formulations in the previous sections are defined based on maximum shear stress, therefore, can be directly incorporated within the PISA clay framework. The normalization functions proposed for the 'clay framework', based on a total stress approach, are simpler and broadly based on analytical solutions. The approach defined results in the normalized P-y PISA parameters ($\bar{p}_u, \bar{v}_{pu}, n_p$ and k_p) having a physical basis from soil mechanics. As shown in the following equations, for example, $\bar{p}_u$ can be seen to be the bearing capacity factor N_p, while k_p is the multiplying factor to G_0, the small-strain shear modulus, to define the initial stiffness of the P-y curve.

$$P_u = \bar{p}_u \, S_u \, D \, , \qquad \bar{p}_u = N_p \tag{5}$$

$$\mathrm{k_i} = \mathrm{k_p} \, \mathrm{G_0}, \qquad \mathrm{k_i} = \frac{\mathrm{P_i}}{\mathrm{v_i}} \tag{6}$$

For both these parameters several analytical solutions exist, such as Jeanjean et al. (2017), Ashford and Juirnarongrit (2003) and Suryasentana and Lehane (2016), respectively, providing a robust rule-based approach for these values. The curvature parameter n_p can be derived from backbone curves such as Darendeli (2001), assuming a shear strain displacement relationship from several simplified approaches summarized in Zhang and Andersen (2017). Finally, the $\bar{v}_{pu}$ parameter can be estimated from rigidity index values at failure, typically varying between 5 to 15% of the ratio of lateral displacement to pile diameter, as per Jeanjean and Zakeri (2023).

5 FINITE ELEMENT ANALYSIS OF P-Y RESPONSE

The comparison between the proposed rule-based method, for both effective and total stress approaches is performed by comparison of the $\bar{p}_u$ and k_p PISA parameters, calibrated to match the P-y curves from the effective stress and undrained load-displacement curves, calculated in FEM presented in Figure 1. The PISA calibration procedure ensures a near perfect match per soil curve extracted which in turn result can replicate the overall pile behaviour. The results obtained are plotted in Figure 4 and Figure 5. Alongside the calibrated PISA parameters, the analytical PISA parameters as per equations (5) and (6) are also plotted for comparison. The rule-based approach is shown to be conservative, both in stiffness and strength, when compared to FEM results. For detailed design stages, multiplication factors, as shown in the right of Figure 4 and Figure 5, can be applied to the formulation to obtain a better match to the FEM model and thereby define global PISA parameters.

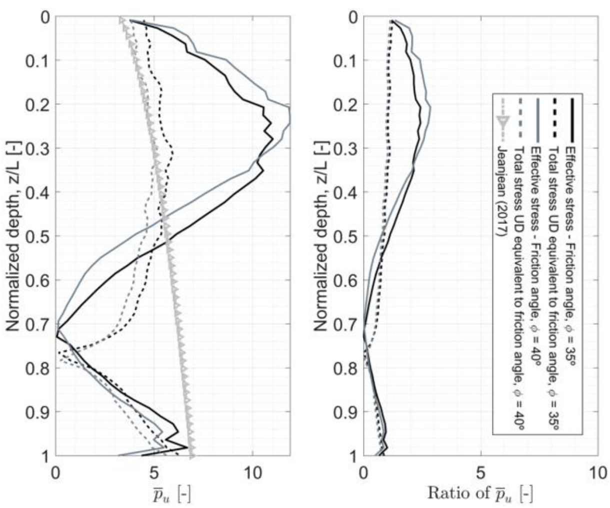

Figure 4. Left - FEM calibrated, analytical $\bar{p}_u$ PISA parameter as given in Eq. (5); Right - ratio between numerical and analytical values.

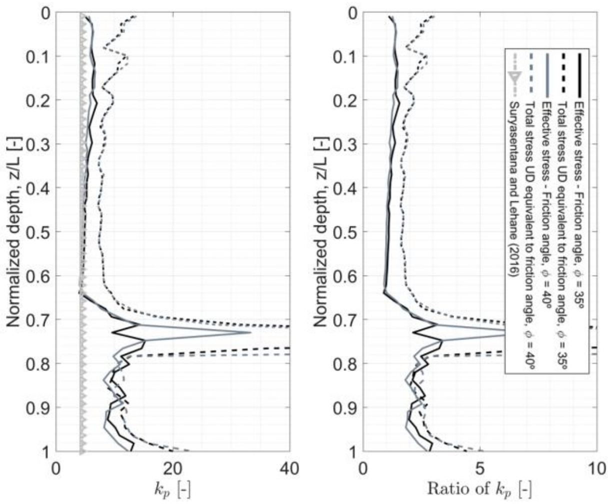

Figure 5. Left - FEM calibrated, analytical k_p PISA parameter as given in Eq. (6); Right - ratio between numerical and analytical values.

6 CONCLUSIONS

The proposed formulation to determine P-y curves for both coarse and fine-grained formations, exhibiting either drained or undrained behaviour has been presented. The method comprises an alternative normalization for coarse-grained materials in the PISA framework and a methodology to determine the failure shear stress of sands under undrained behaviour and a soil mechanics'-based rule-based approach. A comparison to FEM results indicates the rule-based approach proposed to be adequately conservative for early-stage designs.

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