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MONOTONIC RESPONSE OF CAISSONS FOR OFFSHORE WIND TURBINES

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Abstract

The design of mono-caissons for offshore wind turbines requires the definition of the yield envelope of the soil-foundation system. Literature review has pointed out an inconsistency with the definition of the yielding condition for caissons in sand. Since current guidelines requirements are mainly expressed in terms of allowable foundation tilt, this work suggests how three-dimensional finite element modelling can be applied to the analysis of monotonic soil-caisson interaction to calibrate a deflection-based approach that allows to calculate the system moment bearing capacity at any allowable angular rotation.

1. Introduction

The foundation system supporting an offshore wind turbine (OWT) depends on site water depth, that is, on environmental load amplitudes. In shallow water depths (about 30 m), suction caissons are widely regarded as an attractive solution, being less expensive than monopiles of equivalent capacity (Houlsby, 2016). Despite the advantages that the suction installation process provides, the monopile remains the most common foundation choice for offshore wind turbines at present, because of issues about the design of suction caissons for offshore wind turbines, as the loading conditions transmitted by OWTs to the foundation, characterised by very high lateral loads compared to the vertical load. In a monopod configuration, the single foundation must resist directly the vertical, the horizontal and the overturning moment loads. Therefore, the lateral loads control the foundation design.

According to the existing guidelines, offshore foundation design requires the following verifications:

- ULS (Ultimate Limit State): the maximum loads on the foundations, deriving from exceptional events, must be adequately lower than the bearing capacity of the foundation system;

- SLS (Serviceability Limit State): displacements produced by ordinary loading must not compromise the equipment functionality;

- FLS (Fatigue Limit State): displacements due to long-term repeated loads acting on the foundation during the whole OWT operational life must not be greater than the allowable ones.

A straightforward method to estimate the factor of safety against the ultimate limit state of the foundation system is to report yield load combinations in interaction diagrams. Experimental studies provide evidence that the three-dimensional yield surface for combined vertical, moment and horizontal loading of a caisson foundation in sand has a characteristic 'rugby-ball' shape, described by Eq. 1.

$$\left(\frac{M/D}{m_0 V_0}\right)^2 + \left(\frac{H}{h_0 V_0}\right)^2 - 2a\frac{H\,M/D}{m_0 h_0 V_0^2} - \beta_{12}^2\left(\frac{V}{V_0} + t_0\right)^{2\beta_1}\left(1 - \frac{V}{V_0}\right)^{2\beta_2} = 0 \qquad (1)$$

where: m_0V_0 represents the moment load to cause yield for the current (uniaxial) yield load V_0; h_0V_0 represents the horizontal load to cause yield for the current (uniaxial) yield load V_0; a is the eccentricity of yield surface ellipse; β_1 and β_2 are shaping parameters; t_0 is the tension factor; D is the foundation diameter.

Several authors carried out tests with different experimental set-up (in terms of soil relative density, caisson geometry, vertical load, etc.) that allows to explore how it affects the yield surface parameters. Generally, the loads are normalised with respect to the maximum load, V_0, that has been applied to the foundation. To provide insight into the available experimental approaches to predict yielding, they are compared in this section for the case of a caisson foundation characterised by D = 20 m and skirt length, d = 0.5D, subjected to a normalised vertical load, $V_{ad} = V/\gamma' D^3 = 0.4$, embedded into dense sand (D_R = 0.8). For the sake of simplicity, the loads were normalised with reference to the vertical bearing capacity ($V_0 = V_{peak}$). To obtain the vertical load-bearing capacity of the caisson foundation, classical methods for calculating the ultimate bearing capacity of shallow foundations may be used (Liu et al., 2014). It is suggested reducing the friction angle of the soil assuming $\varphi_1 = \frac{2}{3}\varphi$ to consider that the failure mode of the caisson foundation is punching failure. Fig. 1(a) reports the yield surfaces derived with the set of parameters provided by Byrne & Houlsby (1999), Villalobos et al. (2009), Ibsen et al. (2014) (low vertical preload).

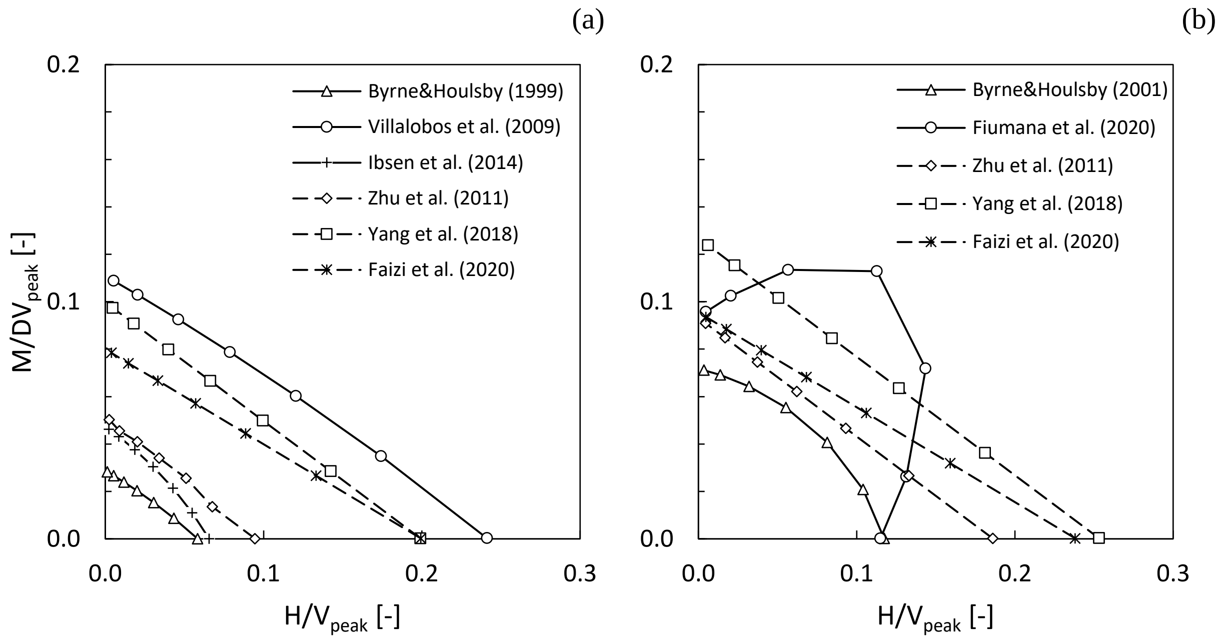

Fig. 1: yield surface for a caisson with d/D = 0.5, subjected to V_{ad} = 0.4, embedded into (a) dense (b) loose sand, experimentally and analitically derived.

The size of the yield surface based on Villalobos et al. parameters appears to be overestimated in comparison to the other experimentally derived envelopes. It seems possible that these results are due to the introduction of t_0 to capture the potential tensile capacity of the foundation (Fiumana et al., 2020). On the other hand, Byrne&Houlsby provide a quite conservative estimation of the yield loads. This might be related to an inconsistency in the h_0 value experimentally derived for d/D > 0.33. Ibsen et al.

results are between the lower bound and Villalobos et al. surface. As Villalobos et al., Ibsen et al. include the tension factor in their approach.
To provide contribution to advance the understanding of the available analytical approaches to predict yielding, the yield surfaces predicted by the expressions proposed by Zhu et al. (2011), Yang et al. (2018) and Faizi et al. (2020) are included for comparison in Fig. 1(a). It could be deduced that the simplified method proposed by Zhu et al. gives yielding estimation in accordance with Ibsen et al. (2014), while the other considered methods overestimate the yield surface, in comparison with the latter analytically derived.
To predict yielding of a caisson ($d/D = 0.5$) embedded into loose sand ($D_R = 0.3$), the set of parameters provided by Byrne&Houlsby (2001) and Fiumana et al. (2020) could be considered. They are compared in Fig. 1(b), along with the analytical approaches. The set of parameters proposed by Fiumana et al. give a different shape of the yield surface that may be due to the higher stress levels investigated, reflecting the prototype scale, in contrast with previous natural gravity tests. Although the latter surface should represent a conservative approximation of the capacity for a caisson with aspect ratio $d/D = 0.5$, as pointed out by the authors, Byrne&Houlsby yield surface underestimates the capacity respect to Fiumana et al. (2020). This may be due to the different properties and response of the soil tested by Byrne&Houlsby, that is an uncemented carbonate sand.
The most striking result to emerge from the comparison is the variety of solutions instead of the expected unique one. An explanation for that may lie in the different: test conditions, force equilibrium model (for the analytical solutions), criterion to determine the yielding load of the soil-caisson system. Regarding the latter aspect, current design guidelines do not provide any recommendation regarding the definition of yielding; in addition, typical load-displacement curves do not present a distinct point of yield. For these reasons, there is not generally agreement upon a method for selecting the yield load of the soil-caisson system. However, as a matter of fact, the design of caissons as monopod foundations for OWTs is not controlled by the Ultimate Limit State but it is dictated by the strict Serviceability Limit State requirements to safeguard the operation of the wind turbine during its lifetime. The main requirement regards the maximum turbine tilt at the mudline, limited to 0.5° by current guidelines as the DNV code (DNV-GL, 2016). Therefore, a deflection-based approach that allows to calculate bearing capacity at any allowable angular rotation results to be the most useful tool for the engineering practice. Hence, a relationship between the moment load and the caisson tilt that allows to predict the moment-rotation curve of the soil-foundation system has been proposed (Letizia, 2022), based on the best-fitting of the numerical results, and presented in the following.

2. Description of the finite element model

2.1 Geometry and meshes

Numerical analyses were conducted using the finite-element program Abaqus/Standard. Considering the symmetry of the geometric model and of the loading condition, a semicylindrical model was developed. 8-node linear brick elements with reduced integration were adopted. The caisson foundation was modelled as a rigid body. The cross-section of the caisson was assumed to be circular, characterised by: diameter, $D = 20$ m; skirt length, $d = 10 \div 20$ m; skirt thickness, $t_s = 0.6\%D = 0.12$ m; lid thickness, $t_L = 0.5$ m. A mesh sensitivity analysis was performed to estimate the diameter and the height of the domain. It resulted that a diameter equal to 9D and a height equal to 4D could be considered sufficient to avoid boundary effects. In this model, the nodes belonging to the base were restrained against vertical and horizontal displacements; displacements in the horizontal plane were restrained on the lateral surface; the symmetric plane was constrained in the normal direction; symmetry mechanical boundary condition was applied to the reference node of the caisson.

2.2 Soil constitutive model

To the aim studying significant patterns of foundation behaviour in sand under monotonic loads, as well as cyclic loads, hypoplastic model with intergranular strain (von Wolffersdorff, 1996; Niemunis & Herle, 1997) was selected.

2.3 Material parameter calibration

For the current study, the soil was assumed to be a loose to dense Toyoura sand deposit. The basic hypoplastic model for sands involves eight parameters: φ_c, h_s, n, e_{d0}, e_{c0}, e_{i0}, α, β. The first six basic model parameters for Toyoura sand will be based on the calibration of Herle & Gudehus (1999). The values of the remaining two parameters, α and β, and of the five parameters involved by the hypoplastic model enhancement of intergranular strain (m_R, m_T, R, β_r and χ) were calibrated by Ng et al. (2016). All calibrated model parameters are summarized in Tab. 1.

Tab. 1: Parameters for Toyoura sand of the hypoplastic model with intergranular strain concept (Herle & Gudehus, 1999; Ng et al., 2016)

Parameter	Description	Value
φ_c	Critical state friction angle	31°
h_s	Hardness of granulates	2.6 GPa
n	Exponent in the power law for proportional compression	0.27
e_{d0}	Minimum void ratio at zero pressure	0.61
e_{c0}	Critical void ratio at zero pressure	0.98
e_{i0}	Maximum void ratio at zero pressure	1.1
α	Exponent	0.14
β	Exponent	1.1
m_R	Parameter controlling initial shear modulus upon 180° strain path reversal	11
m_T	Parameter controlling initial shear modulus upon 90° strain path reversal	6
R	Size of elastic range	$2{\cdot}10^{-5}$
β_r	Parameter controlling degradation rate of stiffness with strain	0.1
χ	Parameter controlling degradation rate of stiffness with strain	1

2.4 Interface properties

The interaction behaviour normal to the contact surfaces was modelled as “hard contact”. The soil-caisson separation was permitted when the contact pressure between them becomes zero. To describe the tangential interaction of contacting surfaces, the Coulomb friction model was adopted. The interface friction angle between the soil and the caisson, δ_μ, was assumed equal to 20°, typical for steel-soil interface.

3. FEM simulations

3.1 Definition of steps and load conditions

The finite element analyses consisted in three calculation steps: after the geostatic step, the vertical load representing the own weight of the tower in addition to the own weight of the foundation was applied. It was assumed V_{ad} = 0.2 and 0.6. In the last step of the analyses, horizontal load, H, and overturning moment, M, were applied together and monotonically increased, in a prescribed ratio, until the caisson experienced a rotation equal to 4°. The ratio M/H corresponds to the eccentricity, h, of the horizontal load respect to the mudline. In the current investigation, h/D was varied parametrically between 0.2 and

7 (h = 4 m ÷ 140 m) so that the analyses results were sufficient to map out a yield envelope.

3.2 Results

To find a general mathematical expression of the interaction diagrams, as a function of the caisson rotation, ϑ_{RP}, H and M were normalised by the following relations:

$$H_{ad,mod} = \frac{H}{\gamma' k_p D^{2.1} d^{0.9} \left(1 + \frac{V^{0.1}}{\gamma'^{0.1} D^{0.2} d^{0.1}}\right)} \tag{2}$$

$$M_{ad,mod} = \frac{M}{\gamma' k_p D^{2.1} d^{1.9} \left(1 + \frac{V^{0.6}}{\gamma'^{0.6} D^{1.2} d^{0.6}}\right)} \tag{3}$$

where k_p is the passive pressure coefficient, estimated using the peak friction angle, $\varphi_{p,ref}$, evaluated by Eq. (4) (Bolton, 1986) at a reference depth $z_{ref} = D + 0.5d$

$$\varphi_{p,ref} = \varphi_c + 3I_{R,ref} \tag{4}$$

$$I_{R,ref} = D_{R,ref}\left[5.4 - \ln\left(\frac{p'_{ref}}{p_{atm}}\right)\right] - 1 \tag{5}$$

$$p'_{ref} = \frac{\gamma' z_{ref}(1 + 2k_0)}{3} \tag{6}$$

where: I_R is the relative dilatancy index, p_{atm} is the atmospheric pressure (100 kPa).
Interaction diagrams derived in terms of the above modified dimensionless variables are reported in Fig. 2. The fitting curves are expressed by Eq. (7):

$$M_{ad,mod} = -\,0.94 \cdot H_{ad,mod} + 0.11 \cdot \vartheta_{RP}^{0.36} \tag{7}$$

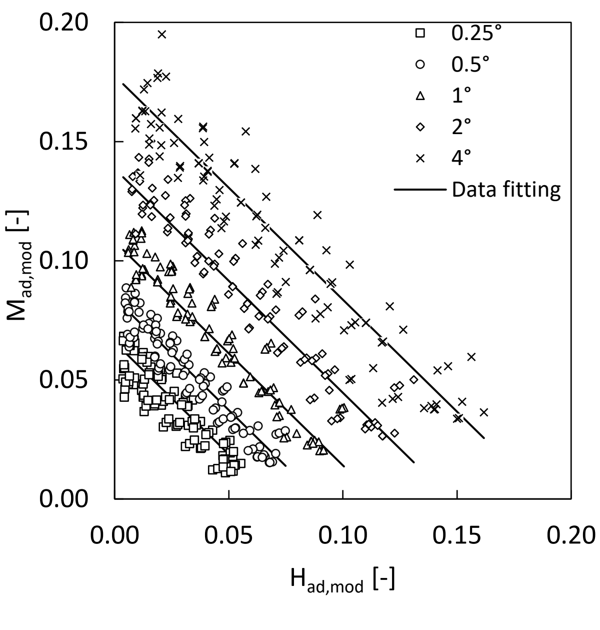

Fig. 2: modified dimensionless interaction diagrams for caissons with d/D = 0.5 ÷ 1.0 under V_{ad} = 0.2 ÷ 0.6 in loose to dense sand for caisson rotation up to 4°.

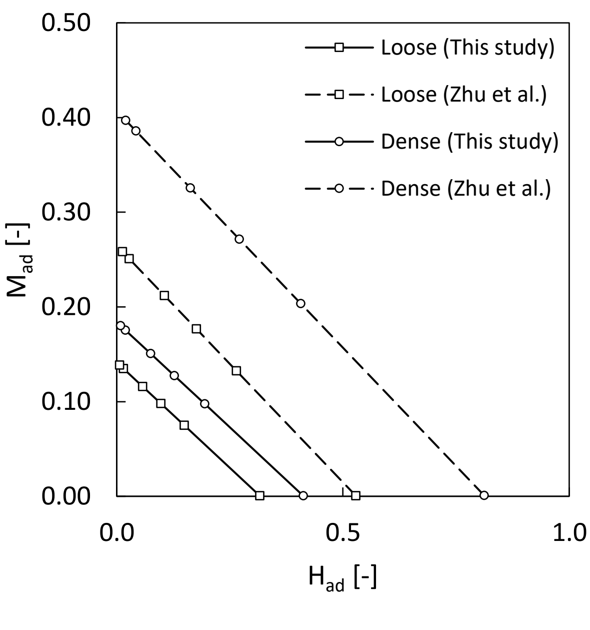

Fig. 3: comparison between interaction diagrams derived from FE results (Eq. 7) and from the simplified method proposed by Zhu et al. (2014).

3.3 Comparison to literature interaction diagrams

A comparison between the analytical approach for deflection-based moment capacity, based on the force equilibrium analysis of the caisson, provided by Zhu et al. (2014), and the approach represented by Eq. (7) is provided in this section.

A caisson with a diameter of 20 m and a skirt length of 10 m has been considered, embedded into a loose, medium dense and dense sand, subjected to a vertical load V_{ad} = 0.4 and a horizontal load at varying level arm. The selected reference rotation is equal to 0.5°.

The major limitation of the analytical calculation considered in this section is that the possibility of a complete detachment, as observed in the described FE simulations and reported in literature, is not considered. In addition, the contribution to moment capacity of the vertical subgrade reaction under the lid results to mainly affect the moment capacity of a caisson in sand. This finding contrasts with previous observations (Yang et al., 2018) that indicate that the main contribution to the moment resistance is provided by horizontal and skin friction stresses acting on the outer skirt at the passive zone. Such shortcoming could lead to a significant overestimation of the moment capacity, as illustrated in Fig. 3.

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