Shallow foundations for offshore wind towers

Fondations superficielles pour des installations éoliennes maritimes

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ABSTRACT: Direct foundations are present in about 25% of the installed offshore wind power towers. The peculiarities of this type of structure are well known: high dynamic sensitivity, complex couplings between environmental actions, machine operation and structural response, complex installation and maintenance, difficult site investigation. There is a clear need for optimized foundation design tools that would enable cost reduction and a more detailed assessment of the risk of every installation. One such tool is likely to be the systematic use of failure envelopes for capacity checks. The paper explores the benefits of such an approach with various realistic design examples.

RÉSUMÉ : Les fondations superficielles interviennent dans la réalisation de 25% des structures éoliennes maritimes. Les particularités de ce type de structures sont bien connues: haute sensibilité dynamique, couplages complexes entre les actions environnementales, le fonctionnement de la machine et la réponse structurelle, installation et maintenance difficiles, investigation géotechniques onéreuses. Un besoin évident d'optimisation des outils de conception est nécessaire pour permettre la réduction des coûts et une évaluation plus détaillée du risque de chaque installation. Le recours systématique à des enveloppes de rupture pour les justifications de la capacité portante des fondations peut bien être un tel outil. Ce papier explore les avantages d'une telle approche avec divers exemples de conception réalistes.

KEYWORDS: direct foundation, capacity, offshore, energy, wind farms

1 INTRODUCTION

Offshore wind is an increasingly large contributor to the energy production mix of several European countries, particularly those bordering the North and Baltic seas. An exponential increase in installations is currently anticipated in this region. It is reasonable to expect that other regions of the world will follow suit.

Offshore wind turbines (OWT) are generally larger than those installed on land, with 3 to 5 MW of nominal capacity being now the norm, but with turbines of up to 10 Mw coming soon to the market. Rotor diameters of more than 100 m and nacelle locations 80 m above mean sea level are common. The result is a relatively lightweight and slender structure, supporting a rotating machine finely tuned to maximize power production while minimizing structural loading.

While initial OWT installations took place near shore (< 10 km) at locations with relatively shallow water depths (< 20 m), current developments are clearly located offshore (10 -100 km from the nearer coast) with water depths of 20-50 m being typical. Several floating support concepts are now being developed; however, commercial installations are still always supported by some kind of fixed structure. For these, the foundation of choice would depend in any case on the particular site conditions, construction equipment availability and, to a certain extent, local traditions.

To this date pile foundations have been largely dominant, mostly as single large (4-6 m diameter) monopile installations, and lately also as smaller (1-2 m) piles for jackets and tripods. However, examining the industry databases (e.g. Burton et al 2011) it appears that at the end of 2011 about 25% of the installed power was supported by direct foundations or gravity base substructures (GBS). Most of these GBS installations took place in relatively shallow waters, but there are some examples already at larger distances from the coastline and in deeper waters. Perhaps the most significant is the Thornton Bank I project, 27 km offshore Zeebrugge in Belgium, where 6 OWT of 5 Mw were installed in water depths of 20-30 m. The foundation design for this installation was described by Peire et al (2009) and its outline is reproduced here in Figure 1. These are large (44 m height; 23.5 m base diameter) concrete shells, floated into place and later ballasted with a mixture of sand and olivine with the base at 4 m below the original seafloor level. The geotechnical profile at the site comprises medium and high density sands and stiff tertiary clays.

2 DESIGN ISSUES FOR DIRECT OWT FOUNDATIONS

There are several specific standards dealing with OWT. Perhaps the highest ranked is IEC 61400-3 (2009) which, from the point of view of structural design, establishes design cases and site ambient load specification procedures, introduces a safety format and gives broad indications about structural design procedures. However, detailed specification of structural and foundation design procedures is deliberately referred to other documents, like the ISO 1990X offshore standard series or DNV-OS-J101 (2010).

As might be expected, the indications given by such standards are, on the one hand, firmly based in conventional design practice when being specific, and somewhat elusive with problems that lack a clear conventional solution. An example of the later is the consideration of fatigue or foundation failure under cyclic loading. An example of the former is the consideration of foundation bearing capacity which, for shallow foundations, follows a conventional superposition and correction procedure not very different from those outlined by Brinch-Hansen (1970) or Vesic (1975).

When designing foundations for OWT, there will be of course issues of geotechnical capacity under extreme loads. However the design drivers might be sometimes related to other considerations, such as dynamic characteristics of the whole structure (Van der Temple and Molenaar, 2002) or displacement limits imposed by operating constraints (e.g. foundation tilting limits of 0.25° - 0.5° are sometimes quoted). However, even if we narrow our focus to bearing capacity considerations there are reasonable grounds to question the suitability of the conventional design approach.

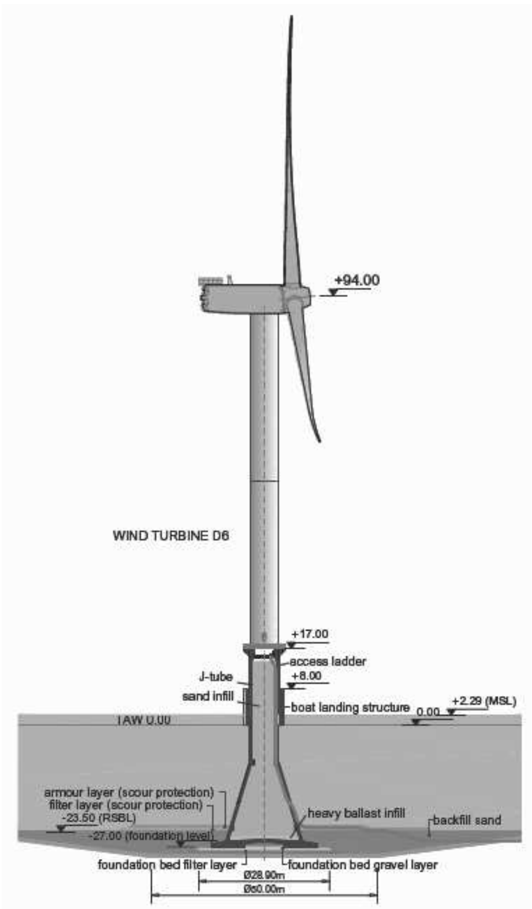

Figure 1. Thornton Bank GBS (Peire et al, 2009).

Indeed there are several aspects of the traditional approach to bearing capacity that are poorly suited to deal with OWT. Firstly, using separate corrections for shape, depth, load inclination, load eccentricity is cumbersome and prone to calibration error if the effects that are being corrected for are not truly independent. This is perhaps the reason behind the large scatter between inclination factor formulations (Siefert & Bay-Gress; 2000); that uncertainty is particularly undesirable for structures, like OWT, that are mostly designed to sustain horizontal loads.

Secondly, the traditional approach to bearing capacity quickly leads to conundrums when the security format (as is the case for most modern codes, like DNV-OS-J101) is based on separate partial factors for loads and resistances. As discussed in detail by Lesny (2007) the same action might have a detrimental or favourable effect depending on which other actions are being simultaneously considered. Also it is fairly evident that a traditional bearing capacity check is far from eliminating the most likely path towards failure.

Finally, it is very difficult to generalize the traditional approach to cases when two major horizontal loads (wind, wave) are acting in separate planes. All these problems are best dealt with if the traditional approach to capacity checks is replaced by a failure-envelope based one.

3 FAILURE ENVELOPES

3.1 *Concept*

Failure envelopes were introduced (Butterfield & Ticof, 1979) as an alternative to classical bearing capacity analyses. They were based on the concept of interaction diagram, which was applied to the system of loads acting on the foundation. Most developments to date –but not all-, refer to the case in which that system can be reduced to loads acting within a plane (V, H, *M*) –where *M* represents the moment acting within the plane, M normalised by a characteristic foundation dimension, M/B.

Failure envelopes are implicit in the traditional approach to bearing capacity. However, it was clearly appreciated from the beginning that an explicit failure envelope was useful to link previously separate checks on different foundation failure modes (e.g. sliding and bearing capacity) into a coherent view. Failure envelopes offered advantages also from the experimental viewpoint, because they provide a clearer framework for experimentation, even suggesting new, more efficient, procedures (like “swipe” tests).

Failure envelopes are also attractive because they can fit well with generalized force-displacement foundation models (“macroelements”; Nova and Montrasio, 1991) that are used to compute foundation displacements and represent an economical solution to non-linear soil-structure interaction studies. Finally, failure envelopes are interesting because they enable a more coherent approach to foundation safety.

3.2 *Safety considerations*

Already Georgiadis (1985) clearly identified as one major advantage of failure envelopes that they allow a very natural consideration of the influence of different loading paths. To do that, it is important to distinguish between the reference design load state and incremental loading paths (Figure 2).

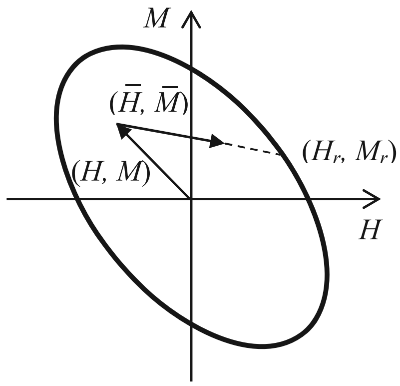

Figure 2. Schematic load envelope illustrating a reference design load and one incremental load path

Any load system (V, H, *M*) shall remain within the failure envelope. It is however convenient to establish a non-dimensional safety measure. To do so a simple approach is, for any incremental loading direction, to obtain the crossing point with the failure envelope (V_r, H_r, M_r) and then define a generalized safety factor, SF, as

$$SF_{(\bar{V},\bar{H},\bar{M})}\left(V,H,M\right)=1+\left|\lambda_r\right| \tag{1}$$

$$(V_r = V+\lambda_r\bar{V} \;\;,\;\; H_r = H+\lambda_r\bar{H} \;\;,\;\; M_r = M+\lambda_r\bar{M}) \tag{2}$$

It is thus made explicit the fact that safety is not only dependent on the initial design situation but also on the incremental loading path. This definition includes, as a particular case, the traditional safety factors against bearing capacity (the incremental load direction and the reference design load are collinear) or sliding (incremental loading direction collinear with the Horizontal component of the reference design load). Another particular case included is that of "plastic overturn", a prescribed check for breakwater design in Spanish regulations (Puertos del Estado, 2005) in which the lever arm of the horizontal loading is maintained (i.e. the incremental load is aligned with the the Horizontal and Moment components of the reference load).

3.3 *Example formulations*

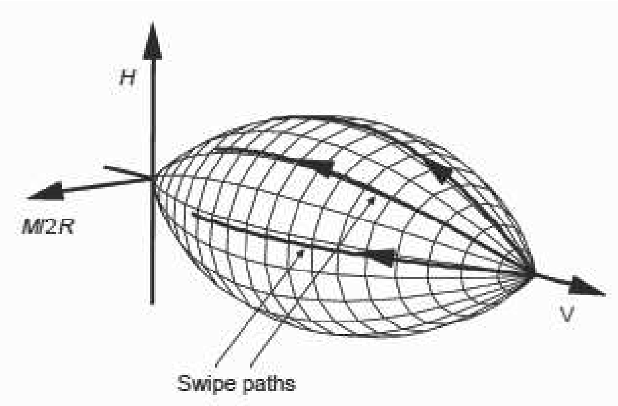

Figure 3 Failure envelope by Gottardi et al (1999)

There are many failure envelopes in the literature. For foundations failing without drainage at the soil-foundation interface Gourvenec & Randolph (2011) offer an excellent review. For the example below a sand profile is assumed and drained conditions are reasonable. In these circumstances a convenient expression for a failure envelope is that proposed by Gottardi et al. (1999) (Figure 3)

$$F(V,H,M) = \left(\frac{h}{h_0}\right)^2 + \left(\frac{m}{m_0}\right)^2 - 2a\left(\frac{hm}{h_0 m_0}\right) - (4v(1-v))^2 = 0 \tag{3}$$

Where (a, h_0, m_0) are shape factors, empirically determined as (-0.22, 0.12, 0.09) for quartzitic sand, and we use a non-dimensional notation in which $v = V/V_0$, $h = H/V_0$, $m = M/(DV_0)$ and D is the foundation diameter. The normalizing factor V_0 is the maximum load (i.e. centered vertical) that the foundation can sustain. Here that maximum load is computed assuming no embedment and introducing the bearing capacity factor N_γ from Bolton & Lau (1993) into

$$V_0 = \frac{1}{2}\gamma D N_\gamma \left(\frac{\pi D^2}{4}\right) \tag{4}$$

It is worth noting that (a) it is relatively straightforward to generalize expression (3) to more complex loading situations – e.g. Lesny 2010- although the experimental base for adjusting the parameters in those circumstances is somewhat scarce, (b) that the shape of (3) above has proven rather resilient and very similar expressions have been found to fit well other foundation test results in materials like carbonate sand or even clay (Martin & Houlsby, 2001), as long as the contact surface remains drained. Of course the choice of V_0 would change according to the material and foundation shape.

4 EXAMPLE APPLICATION

To illustrate the argument we propose an example, synthetic but realistic. The case is developed using the characteristics of the gravity base substructure built at Thornton Bank (Peire et al. 2009) and the design loading specified for a Baltic windfarm development site, Kriegers Flak (Bulow et al, 2009). This reference gives some basic characteristics for the OWT superstructure (Table 1).

Table 1 Super-structure characteristics

Rated power	5 MW
Rotor diameter	126 m
Nacelle height above msl	90 m
Nacelle-rotor weight	4.1 MN
Tower weight	3 MN

The same reference also includes resultants from ambient loads for a range of depths and load hypothesis (e.g. extreme, fatigue). Using these data, Table 2 has been computed for a 30 m depth case and extreme load scenario. It appears that, in this particular case, 80% of the total horizontal thrust is due to sea action, but this load is the source of less than 20% of the overturning moment at foundation level. This might partly reflect the fact that at that particular site sea current is relatively strong, lowering the action line of sea forces.

These ambient loads should be combined with the OWT selfweight. Using the Thornton Bank design like a template for substructure shape, the relevant characteristics of that part of the OWT are those listed in Table 3. As usual with gravity base OWT, the dead weight of the substructure is significantly larger than that of the superstructure. Combining all environmental actions and structure selfweight the resultant load combination acting at the foundation level is (H, V, M) = (10.1; 44.5; 284.3) in MN and MNm. This will be the reference design load state in this example.

Tabla 2 Ambient loading parameters

Parameter / load	Unit	Value
Total thrust, H	MN	10.1
Total overturning moment, M	MNm	284.3
Wind thrust, Hw	MN	2.03
Wind arm lever, bw	m	120
Sea thrust, Hs	MN	8.07
Sea arm lever, bs	m	5

Tabla 3 Thornton Bank type substructure characteristics

Parameter / load	Unit	Value
Base diameter	m	23,5
Concrete weight	MN	30
Fill weight	MN	38
Buoyant volume	m3	2965

From that reference state we probe the failure surface alongside three different incremental loading paths. One will correspond to a simultaneous and proportional increase of all ambient actions (the "plastic overturn" case). The other two hypothesis would correspond to increases of just one of the ambient horizontal actions, (sea, wind) while the other remains constant. These hypotheses would, for instance, naturally follow from any circumstance in which the estimates of wind and wave carry different uncertainties. Figure 4 illustrates graphically the meaning of these load directions in an idealised section of the failure envelope at constant V.

For this check we use the failure envelope of Gottardi et al described above. The soil profile below the foundation is characterised by a friction angle of 33° and submerged weight of 10 kN/m^3. These values might correspond well to the characteristic values of a medium-dense sand profile, frequently encountered in North Sea locations. It is assumed that the foundation base is perfectly rough.

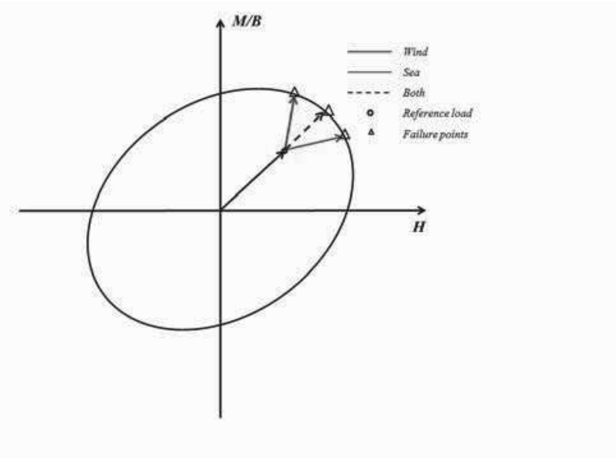

Figure 4 Incremental load paths in the example

Table 4 Example: results

Hypothesis	Hr (MN)	Hr / Hi	ΔH (%)
Sea	14.1	1.4	50
Both	11.6	1.15	15
Wind	10.5	1.04	21

Some relevant results from the computation are presented in table 4. For each incremental loading path a failure point is identified in the envelope, with values (H_r, M_r). In the table the value H_r corresponding to each loading path is reported in the first column. In the second column this value is normalized by the reference state horizontal load. This corresponds to the generalized safety factor defined above, which, only for the hypothesis in which both loads are simultaneously increasing, coincides with the "plastic overturn" safety factor of ROM 0.5-05. As a reference the value required for that safety factor in breakwaters is commonly above 1.3 (Puertos del Estado; 2005).

For the other two load hypothesis in which only one environmental action is increased no similar reference exists to judge on the computed safety factor. For these cases it is perhaps more meaningful the number in the third column of Table 4, where the difference between failure and reference thrust is expressed as a percent of the reference ambient load that is increasing. In the case computed, a 21% error in the reference estimate of wind thrust would result in foundation failure, whereas it would be necessary a 50% underestimate of the hydrodynamic thrust to fail the foundation.

The previous computations have always been made under the hypothesis of increased thrust and constant lever arm. This can be interpreted as action magnitude uncertainty. Alternative hypothesis dealing with lever arm uncertainty can be equally set up with relative ease. Note, finally, that most geotechnical uncertainty can be lumped in the V_0 estimate to achieve a relatively straightforward approach to reliability evaluation.

5 CONCLUSION

Failure envelopes offer a powerful framework to analyze shallow foundation capacity problems. They seem particularly suitable for offshore wind towers, where refined design in the face of large load uncertainties is likely to be a frequent situation.

6 ACKNOWLEDGEMENTS

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