Cyclic loading of caisson supported offshore wind structures in sand

Chargement cyclique des éoliennes offshore soutenues par des caissons à succion en sable

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ABSTRACT: With the number of offshore wind turbines in Europe growing rapidly, offshore wind farm developers are looking for support structures which are relatively light, easy to produce and install and are suited for water depths in excess of 30m. Suction caissons could offer a solution for these requirements. Since cyclic environmental loads form an important part of the loading conditions, the cyclic degradation of the caisson capacity needs to be evaluated in detail. During storm events, pore pressure build-up inside and around the caisson can lead to degradation of capacity and stiffness. To date, there are no generally accepted material models which combine generation and dissipation of pore pressure with the mechanical response of the sand. Existing methods for analyzing pore pressure build-up are reviewed. Subsequently, a numerical model is proposed which captures the phenomena of pore pressure generation and dissipation around the caisson. Pore pressure increases under storm load cycles are calculated from cyclic laboratory tests and are added to existing pore pressures in the numerical model. The influence of cyclic loading history and drainage effects on the caisson performance is assessed using the 3D FE model. Implications for suction caisson design in sand are outlined.

RÉSUMÉ : Vu la croissance rapide du nombre d'éoliennes offshore en Europe, les développeurs des parcs éoliens offshore sont intéressés par des structures combinant légèreté, facilité de fabrication et qui sont adaptées à des profondeurs d'eau supérieures à 30m. Les caissons à succion répondent à ces critères. Comme les charges environnementales cycliques constituent une partie importante du chargement total, la dégradation cyclique de la capacité portante du caisson doit être évaluée en détail. Lors de tempêtes, l'accumulation de pressions d'eau interstitielle à l'intérieur et autour du caisson peut induire une dégradation de la capacité et de la raideur. A ce jour, il n'existe pas de modèle de matériau unanimement accepté qui combine génération et dissipation de pression interstitielle et comportement mécanique du sable. Les méthodes existantes d'analyse de génération de pression interstitielle sont examinées dans un premier temps. Ensuite, un modèle numérique intégrant les principaux mécanismes de génération et dissipation de ces surpressions autour du caisson est introduit. L'augmentation de pressions interstitielles résultants des charges cycliques dues aux tempêtes est estimée de manière indirecte sur base des résultats d'essais cycliques en laboratoire; ces surpressions sont ensuite ajoutées aux pressions interstitielles existantes dans le modèle numérique. L'influence de l'historique de chargement cyclique et des conditions de drainage est évaluée à l'aide du modèle éléments finis 3D. Enfin, les implications de ces résultats pour la conception de caissons à succion sont exposées.

KEYWORDS: suction caisson, cyclic loading, liquefaction analysis, offshore wind turbine, marine geotechnics

1 INTRODUCTION

1.1 *Suction caisson as foundations for offshore wind turbines*

The European Wind Energy Association expects that the installed offshore wind capacity within the EU will increase from 4GW to 40 GW by 2020 (EWEA 2011) requiring the installation of approximately 6000 6MW turbines located ever further offshore in consequently deeper waters. Due the demanding working conditions at sea and the limited availability of offshore installation vessels, the foundation system typically accounts for up to 25-30 % of the total cost of an offshore wind farm. This makes the choice and design of the foundation an important factor in the overall cost effectiveness of offshore wind farms.

Offshore wind farm developers are thus looking for support structures which are relatively light, easy to produce and install and are suited for water depths in excess of 30m. Suction caissons could offer a solution for these requirements.

A suction caisson is a steel structure consisting of a circular top plate with peripheral vertical skirts (Figure 1). In operation it is similar to a skirted gravity foundation, but the skirt length is significant compared to the diameter.

Installation of the caisson is achieved in two phases. After initial penetration under the self-weight of the caisson, water is pumped out. The induced pressure difference pushes the caisson into the soil, while the induced seepage forces and reduced effective stress near the skirt tips facilitate penetration.

Advantages of the caisson include a potentially lower cost than equivalent piled foundations (Senders 2008) and relatively easy installation and removal, not restricted by water-depth.

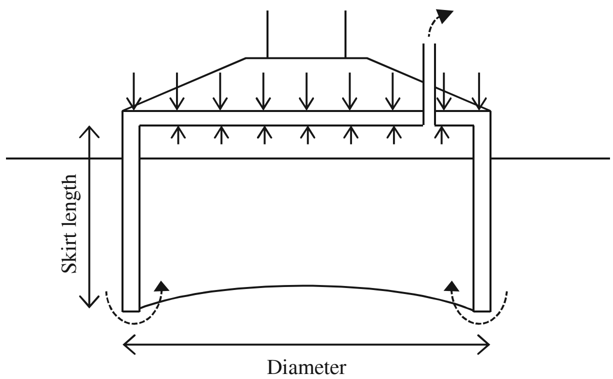

Figure 1: Cross-section sketch of a suction caisson and installation principle

1.2 *Loads on offshore wind turbines*

The foundation must resist loads caused by the weight of the structure, the operation of the turbine, currents, wind and wave action. Incoming waves exert a cyclic horizontal force (and moment) on the foundation, which in the case of offshore wind turbines may be a significant proportion of the weight of the structure. A vertical weight of 6MN and a horizontal wave loading of up to 3MN are realistic values for a 3.5MW turbine (Houlsby et al. 2005).

The offshore design standard DNV-OS-J101 (DNV 2011) specifies that the structure must be able to resist a 50-year design storm (a storm with a probability of occurrence of 1/50 during one year), where not only the peak loads, but the entire history of cyclic loading affects the stability of the structure. For the cyclic loading assessment, the irregular wave loading is usually converted into an idealized, equivalent design storm.

1.3 *Structural configuration*

Caissons could support offshore wind turbines in two ways, based on mode of load transfer to the soil (Figure 2). A monopod foundation consists of a single caisson and is suited for shallow waters. In deeper water, the increased moments acting on the caisson would require a very large caisson. In that case a tripod (three caissons) or quadripod (four caissons) structure could be economical, as moment loads are converted into a vertical push and pull action on the individual caissons.

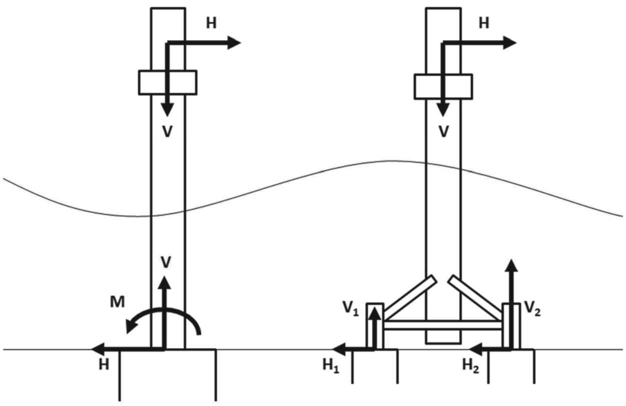

Figure 2: The monopod and multipod concept and reaction forces on the caissons

1.4 *Scope of work*

The aim of this paper is to examine the effect of cyclic loading during a design storm on both the monopod and multipod and to produce a model which is suitable for engineering practice. The presented model is still under development, and is considered a starting point for more sophisticated approaches.

2 CYCLIC DEGRADATION OF SOILS AND FOUNDATIONS

2.1 *Pore pressure build up in sand under cyclic loading*

Cyclic shearing of sand degrades the soil structure and causes a tendency to densify. This is the case even for very dense sands that are dilative under monotonic loading conditions (Seed and Idriss 1980, Andersen and Berre 1999).

Under undrained conditions, volume changes are prevented by the low compressibility of water, so normal stresses carried by the soil will be transferred to the pore water, thus increasing the pore water pressure in the sample as illustrated in Figure 3. The decrease in effective stress furthermore causes a progressive increase in average shear strain. Failure occurs when the generated pore pressure reaches a critical value u_{max}.

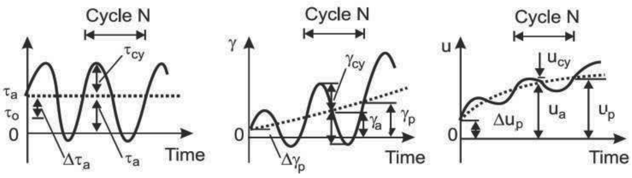

Figure 3: Behaviour of sand under cyclic loading (after Andersen and Berre 1999)

The intensity of cyclic loading is expressed in terms of the cyclic shear stress ratio, the ratio of cyclic deviatoric stress amplitude over mean effective stress. This formulation is convenient for the interpretation of triaxial test results and for implementation in the finite element procedure.

$$CSR = q_{cy}/p'_o \quad with\, q = \sigma_1 - \sigma_3 \tag{1}$$

Based on several cyclic tests at different CSR, cyclic shear strength curves can be established, expressing the number of cycles required to induce failure N_l as a function of the CSR and Dr.

The cyclic shear strength depends on the relative density and the initial shear stress in the sample. The set of curves used in this study was presented by Lee and Focht (1975) in their investigation of the liquefaction potential at the Ekofisk site, North Sea. The curves for this typical dense North Sea sand are redrawn in Figure 4.

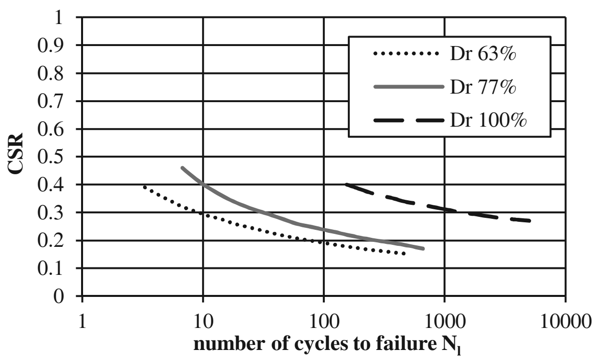

Figure 4: Cyclic shear strength curves for dense North Sea sand at the Ekofisk site (after Lee and Focht 1975)

The build-up of pore pressure in samples can be described by the empirically determined pore pressure generation function given in Eq. 2 and plotted in Figure 5. The empirical constant α depends on the soil properties and is on average equal to 0.7 (Rahman et al. 1977). As it is cyclically loaded, the soil sample evolves from the initial, undisturbed state at $N = 0$ to a state of liquefaction at $N = N_l$ and $u = u_{max}$.

$$\frac{u}{u_{max}} = \frac{2}{\pi} arcsin\left[\left(\frac{N}{N_l}\right)^{1/2\alpha}\right] \tag{2}$$

2.2 *Drainage conditions*

In laboratory tests soil samples are brought to failure under undrained conditions. However, in situ loading conditions may be fully or partially drained, depending on the combination of soil permeability, frequency of the loading and drainage conditions.

For offshore turbines founded on sand, the high permeability and relatively slow wave loading results in the dissipation or redistribution of a significant part of the generated pore pressure during the cyclic loading itself. This effect becomes more important as the soil permeability increases and the loading frequency diminishes. Not taking into account the simultaneous dissipation leads to overestimation of the generated pore pressure and potentially to overconservative design.

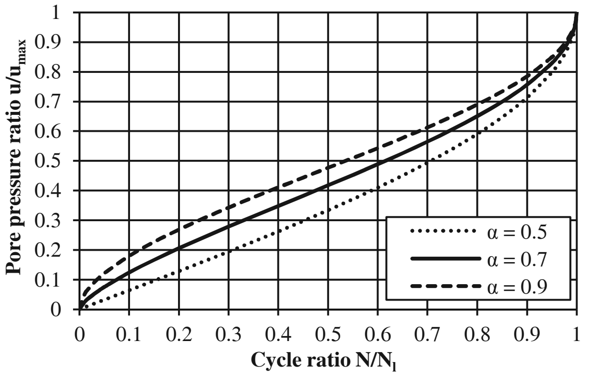

Figure 5: Pore pressure generation function.

2.3 *Liquefaction of foundations*

The definition of failure of a foundation due to liquefaction requires special attention. Not all parts of the soil under a foundation will fail at the same time or will fail at all. Some intensely loaded zones may liquefy completely or partially, while other zones may still be intact.

Taiebat (1999) discussed the problem and proposed the following definitions. *Total failure* of a foundation-soil system under cyclic loading is defined as the condition where the soil mass deforms continuously under the ambient and cyclic loads applied to the foundation, resulting in bearing capacity failure. *Partial failure* involves large permanent displacements during cyclic loading. Some elements of the soil liquefy and lose their strength, but overall, the soil mass remains stable.

Due to the complexity of the problem, numerical analysis is often the preferred method to asses to what extent the foundation capacity is degraded.

3 EXISTING NUMERICAL METHODS

There are at least two approaches to numerical modelling of offshore foundation liquefaction. In the first approach an appropriate constitutive model is used to capture cyclic stress-strain behaviour of the soil. Many such models exist and they can successfully reproduce soil behaviour in laboratory conditions (e.g. bounding surface plasticity, multi-surface plasticity). However, the number of required parameters and calculation time are two obstacles that up to now have limited application of these models to analysis of boundary value problems in engineering practice.

The second approach is simpler and consists of improving a conventional (possibly slightly modified) constitutive model by incorporating the effects of cyclic loading separately, based on a set of laboratory tests. A rigorous review of the work by researchers who followed this approach to analyze offshore foundations subjected to wave loading is given by Taiebat (1999).

4 IMPLEMENTED METHOD

The proposed method follows the second approach and is based on the work by Rahman et al. (1977), Taiebat (1999) and to a lesser extent Lee & Focht (1975) and Verruijt & Song (1991).

The calculation procedure is as follows: undrained pore pressure increases are calculated analytically, at regular time intervals in the FE analysis. At each node, the pore pressure at the end of the previous interval (which includes effects of all previous loading) is converted into an equivalent number of cycles using Eq. 2. The increase in pore pressure during next interval (containing a number of load cycles) can then be calculated from Eq. 2, assuming the CSR is constant during this interval.

After the pore pressure and effective stress in the FE analysis are updated accordingly, the dissipation analysis continues over the length of the considered time interval. This is done in a coupled Biot-type consolidation analysis in the FE package Abaqus.

The total design storm consists of a number of load parcels, during which the cyclic load (and thus the CSR) is assumed to have a constant average and amplitude. The load parcels are subdivided in a number of steps and the process of updating the pore pressure and subsequent consolidation is repeated for every subdivision, tracing the average pore pressure response (excluding oscillations within each load cycle) over the entire load history of the design storm.

5 APPLICATION TO SUCTION CAISSONS

In two case studies the influence of cyclic loading history and drainage effects on the caisson performance is assessed using the proposed model. Realistic forces acting on the foundation are estimated from the loads outlined in section 1.2 and a simplified load histogram is adopted. Corresponding realistic caisson dimensions are found by applying the bearing capacity equation (DNV 1992) for the tripod caisson and the formula proposed by Byrne and Houlsby (2003) for the monopod caisson. In both cases the sand is represented by an isotropic elastic material model with Mohr-Coulomb plasticity.

5.1 *Leeward caisson of a tripod*

5.1.1 *Model*

Initially the horizontal load, divided over three caissons, is neglected. The resulting axisymmetric problem only considers vertical cyclic loading on the individual caisson due to weight of the structure and overturning moments as this is the most important load component. The histogram consists of 3 load parcels of 2000 seconds each, applying 200 load cycles at 60% of the maximum load in the first and last parcel and 200 cycles at maximum loading in the middle parcel.

5.1.2 *Results*

An example of calculated pore pressure response within and around a 8x8m caisson is shown in Figure 6. First of all it is clear that the abrupt increases (generation) and gradual decreases (dissipation) are an approximation for the real behaviour.

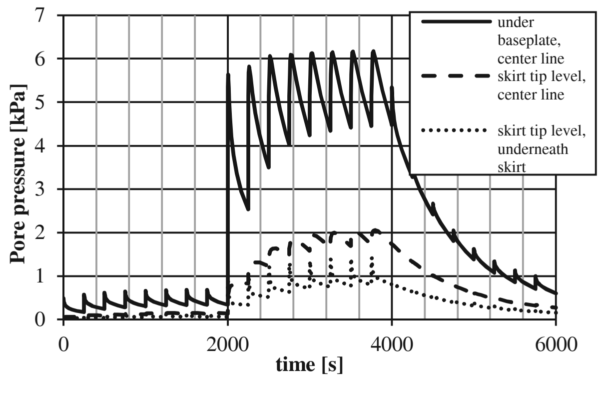

Figure 6: Example of excess pore pressure history, tripod caisson

The analysis predicts liquefaction of the soil near the skirt tips and build-up of pore pressure inside the caisson during the second load parcel. Stress redistribution towards the baseplate will cause an additional increase in pore pressure inside the caisson. The abrupt increase at t = 2000s is due to the nonlinear dependency of generated pore pressure on the CSR, which increases at the start of the second load parcel. Much of the pore pressure is dissipated by the end of the last load parcel, even though cyclic loading continues (at 60% of the second parcel). As the pore pressure dissipates, settlements due to the cyclic loading are expected.

The discretization of cyclic loading in load parcels and subsequently in subdivisions affects the accuracy of the analysis, but the results seem to converge as the number of subdivisions is increased. Where short drainage paths or high CSR values are involved, sufficiently short steps are required. The rate of pore pressure dissipation is affected by the length of the skirts. Longer skirts result in slower dissipation and higher potential for pore pressure accumulation inside the caisson.

5.2 *Monopod*

5.2.1 *Model*

The monopod caisson (20x10m) is subjected to three degree of freedom loading, including a horizontal and moment load. A 3D FE model of half the caisson is sufficient, taking advantage of the plane of symmetry formed by the vertical and the direction of aligned wind and wave loading. A six hour design storm, consisting of 2160 waves in five load parcels, was adopted.

5.2.2 *Results*

The five load parcels are distinguishable in the pore pressure response plotted in Figure 7 and peak pore pressure occurs right after the peak of the storm. The permanent horizontal load due to wind and/or current causes an asymmetric cyclic shearing in the example, so the observed peak pore pressure (4 kPa) does not occur on the center line. The consequences, such as potential differential settlements and tilting of the turbine, should be examined in a more advanced analysis.

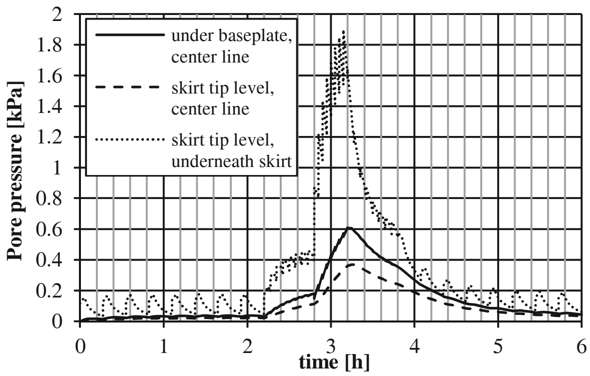

Figure 7: Example of excess pore pressure history, monopod caisson

6 CONCLUSIONS AND FURTHER DEVELOPMENTS

A pore-pressure generation and dissipation model has been developed to study the effect of cyclic loading on suction caissons in sand. Example analyses have shown that the proposed model can be successfully applied to the study of suction caissons, both in 2D and in 3D. However, the model needs further improvement to allow prediction of the complete liquefaction behaviour, including settlements, of a caisson.

The model can be used to predict which areas are prone to pore pressure build-up, estimate the rate of pore pressure build-up and to some extent how fast this pore pressure is dissipated.

Analysis of the type presented here may be useful to assess the geotechnical and structural risks related to cyclic loading of caissons in sand such as:

- reduction in caisson bearing capacity due to generated pore pressures;
- caisson foundation stiffness reductions;
- pore pressure induced total and differential settlements for offshore wind turbine structures;
- analysis of the effect of scour on pore pressure gradients.

The model can be improved to reflect more realistic soil behaviour. As some zones underneath the suction caisson liquefy, the load is transferred to other parts of the foundation. This leads to secondary pore pressure increases which are not yet considered in the presented model.

If sufficient soil data are available, the cyclic shear strength curves could include dependency on the relative density and initial shear stresses in the soil.

Finally, a large part of the vertical load on suction caissons is taken by friction between the caisson skirts and the soil. A systematic study of the influence on the liquefaction potential would be interesting.

7 ACKNOWLEDGEMENTS

The work described in this paper was performed as a part of the author’s master thesis (Versteele 2012), supervised by professor Charlier (Université de Liège), whose guidance is gratefully acknowledged. Development of the model and calculations were performed at, and with support of Cathie Associates SA/NV.

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