Long term effects of cyclic loading on suction caisson foundations in sand

Ir. Cristina Lupea
TU Delft

Ing. Rene Thijssen
Volker InfraDesign / SPT Offshore

Prof. Frits van Tol
TU Delft / Deltares

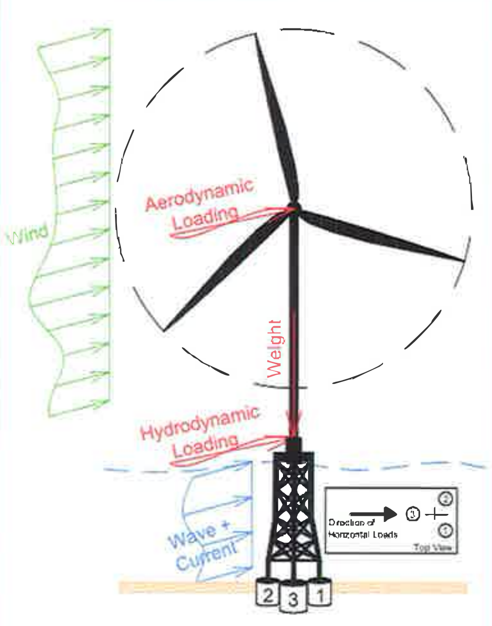


Figure 1 - *Sketch of the considered structure, 6MW wind turbine with a three-leg jacket substructure*

Criteria \ Type of Literature	Norms, Standards and/or Guidelines			Other Literature, Research	
	API/ISO	DNV	NORSOK	Strain accumulation models	Pore pressure build-up models
Model Type	Empirical	Empirical	focus - steel structure	Hybrid/Constitutive	Hybrid
Foundation Type	Long pipe piles	GBS	Piles	Any	Any - case study on GBS
Soil Type	Sand	Sand	Not mentioned	Sand	Sand
Cyclic loading	-/+	+	-/+	++	++
Displacements/Strain Accumulation	only due to static loading	only due to static loading	-	++	+
Strength and/or Capacity	+	+/-	-	+	+
Stiffness / Relative density	-	-	-	+	++
Stresses / Pore pressures	-	+	-	++	++
Limitations	cyclic load "=" quasi-static	effective stress methods - all contributions to pore pressures must be included	Focus - steel components and not soil. Selected criteria ref. steel.(ISO 19901-4 for geotechnical design)	Iterative process - error generation (explicit + control steps). Large number of parameters, soil data for calibration.	Iterative process -error generation (explicit + control steps). Large number of parameters with unclear physical meaning, calibration based on laboratory tests.

Figure 2 - *Comparison of modelling capabilities of the available approaches*

INTRODUCTION

Recently published statistics by the European Wind Energy Association, EWEA (2012), have shown that the investments in the offshore wind energy sector have increased from under €500 million in 2001 to €4500 million in 2012. According to DTI (2001), 30% of these investments is related to foundation costs. In order to reduce the overall costs two solutions were identified: increasing the turbine capacity and/or moving further from the shore where higher wind power could be tapped in. Both these solutions lead to an increase in the foundation size and its costs. According to Senders (2008), significant project costs during installation are caused by stand-by periods as consequence of bad weather conditions. Thus, decreasing the number of offshore operations required by simplifying the foundation installation may reduce costs significantly.

Suction caisson foundations provide a simplified installation procedure. According to Senders (2008) and Byrne & Houlsby (2003) suction caissons within a multi-footing configuration are a viable economical and environmentally friendly solution. Unfortunately, in the current guidelines and standards limited guidance is given related to the assessment of their long term performance throughout the 25 year design life time (characterised by over 10^8 loading cycles).

Lupea (2013) gives an overview of the available approaches and suggests an alternative method to assess the long term behaviour of suction caissons under cyclic loading conditions. The current article presents only a summary of these approaches. The focus is to present the results of the modelling carried out, as these may simplify the design process, provided that validation through laboratory and/or in-situ testing is done.

PROBLEM DESCRIPTION

The effects of long term vertical cyclic loading on a suction caisson embedded in sand are to be determined in the context of:

- Loss of stability – the reduction of subsoil bearing capacity due to pore pressure build-up;
- Loss of serviceability – accumulation of differential settlements (i.e. introduction of rotation).

It is to be noted that the governing requirements are related to the overall tilting of the wind turbine structure, thus imposing strict boundaries related to differential settlement of the suction caissons underneath the three leg jacket structure (see figure 1).

AVAILABLE APPROACHES

The two most commonly used standards within

Summary

Offshore wind turbine projects have been characterised by an increase in costs, sizes and distances from shore, EWEA (2012). This created a need of investigating the adequacy of alternative and more financially attractive foundation types such as suction caissons. Within a multi-footing configuration, such as a three-leg jacket structure, these foundations prove to be an advantageous solution for increasing water depths. It is due to their configuration that a simplified design is possible, leading to only vertical cyclic loading conditions. This article provides a concise summary of the M.Sc. thesis of C. Lupea and presents an approach that may allow for the simplification of the long term performance assessment under vertical cyclic loading of suction caissons embedded in sand. It provides a conservative theoretical base for the identification of potentially damaging loads, which could cause significant pore pressure build-up and strain accumulation. One of the key conclusions of this research is that the foundation response is a function of both the applied mean load and its cyclic amplitude for both tensile and compressive loading. Nonetheless, experimental work must be carried out to validate these results.

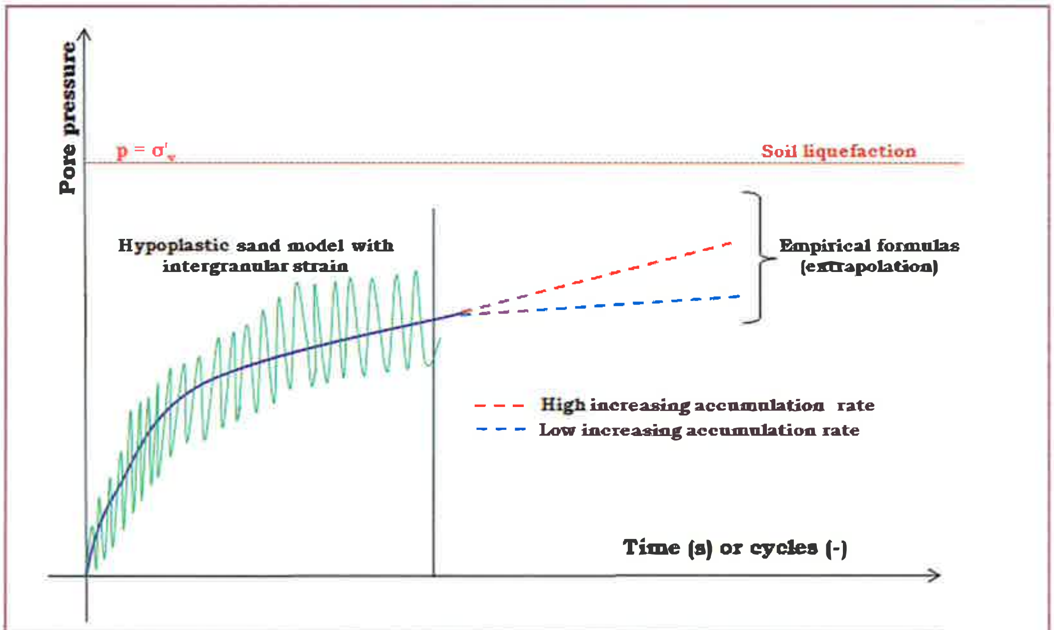


Figure 3 - *Sketch of proposed model for pore pressure build-up (similar procedure for strains)*

the offshore industry, the American Petroleum Institute (API) and Det Norske Veritas (DNV), provide guidelines for the analysis of slender pipe piles (length (L) over diameter (D) ratio > 10) and gravity based foundations (L/D<1). For suction caissons having an embedment ratio (L/D) between 1 and 10, depending on the soil type and installation pressures, no specific guidance is given. Furthermore, these standards provide limited guidance in the assessment of cyclic loading on foundations through empirical formulas (the API suggests the use of a reduction factor A = 0.9 for the lateral resistance of slender piles, that is independent of the cyclic loading characteristics; the DNV proposes an effective stress analysis and a reduction of the shear stress based on pore pressure build-up for gravity based foundations).

The available research suggests the use of constitutive models with a better capacity to assess the change in soil properties throughout each individual cycle. These models are either based on strain or pore pressure accumulation. A bridge over the large number of cycles, characteristic to offshore loading conditions, is generally provided through the use of empirical formulas and control cycles (Safinus et al. 2011). These types of models shall be considered as hybrids.

From literature search, several criteria were identified as being significant in the assessment of this problem. The comparison of the available solutions to the problem at hand is given in figure 2. It becomes clear from this figure that the research carried out covers much better the questions faced now in the offshore wind industry.

PROPOSED MODEL

From literature it is concluded that for the problem at hand the use of a hybrid model using the hypoplastic sand model (Von Wolffersdorff, 1996) with intergranular strain (Niemunis & Herle, 1997) is to be preferred. The reasons for this choice reside with the good prediction capabilities of the model and its previous usage as the base for the High Cyclic Strain Accumulation model (Wichtman et al. (2011)). Once a trend can be observed in pore pressure build-up and strain accumulation, extrapolation formulas can be used – see figure 3.

Loading conditions

An important role in the proposed model is played by the considered loading conditions (see figure 1). In the case that was analysed for the thesis, representative for North Sea conditions, loading conditions were represented by the resultant forces at seabed level for the simulation of the 25 year design life time of a 6MW wind turbine. The raw data on reaction forces at jacket leg- caisson interface were statistically processed in order to determine both extreme values as well as the most probable operational loads.

The extreme values were used in the predesign of the suction caissons for a three-leg jacket structure. A safety factor of 2.5 was used in order to check the foundation's capacity to undertake tension. The obtained caisson dimensions are 15 m diameter and 16 m embedment length, using formulas provided by API and DNV for undrained conditions. A simplified analysis showed that the required pressures for installation were within the limits imposed by the water depth of 40m. As this was not the main topic of research no further analysis was carried out (i.e. buckling analysis).

The most probable operational load cases were determined based on the highest number of occurrences and the highest mean and/or amplitude of the load, whichever is governing related to pore pressure build-up and strain accumulation. Due to the high safety factors used in the predesign, the dominant operational loading cases for the three caissons proved to be below 10% of the caisson's capacity. For clarity, the following values will be referring to unfactored loads with respect to the capacity of the 15 m diameter suction caisson, with a 16 m embedment length.

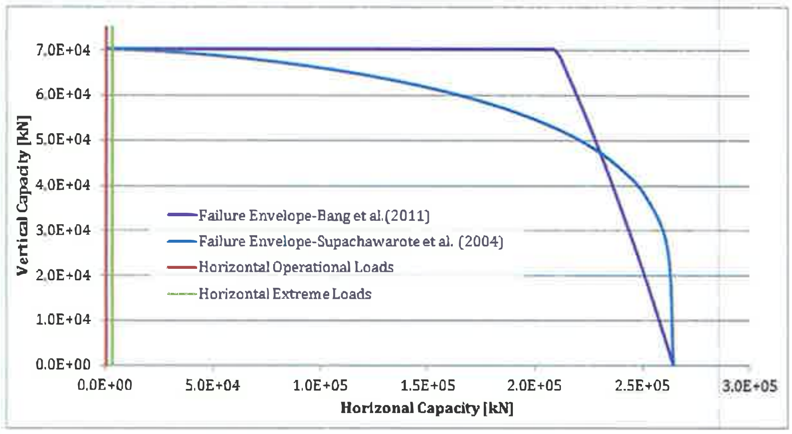


Figure 4 - *Failure envelope of the predesigned caisson*

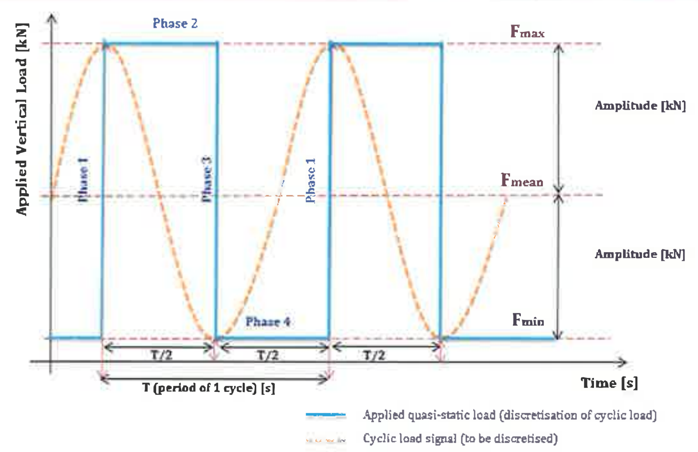


Figure 5 - *Sketch of discretised loading in four phases*

The influence of horizontal loading was investigated by using the formulas suggested by Bang et al. (2011) and Supachawarote et al. (2004). The latter method is applicable for clays. For this method, the undrained shear strength of sand was determined using DNV (1992) recommendations.

It was concluded that horizontal loading did not yield a significant change in the vertical capacity – see figure 4. The problem at hand was therefore simplified to vertical cyclic loading only. The loading conditions were not applied cyclically, but quasi-statically using four phases as shown in figure 5. In the odd numbered phases the load was instantly applied undrained, while in the even numbered phases time was allowed for consolidation for half the period of one cycle. The period was conservatively chosen equal to the wave period, approximately 7 s, even though from a study on the loading conditions follows that the governing cyclic loads are likely to be closer to the typical wind period of 30s. These values are also outside the order of the operating frequency of wind turbine generators.

Table 1 - Hypoplastic sand model with intergranular strain concept parameters

Symbol	Description
ϕ_c	Critical state friction angle [°]
h_s	Granular hardness (controls the shape of the limiting void ratio curves – slope)
n	Exponent of compression law (controls the shape of the limiting void ratio curves – curvature)
e_{d0}	Void ratio at maximum density for p=0 (controls the peak strength)
e_{c0}	Void ratio at critical state for p=0 (controls the peak and residual strength)
e_{i0}	Void ratio at minimum density for p=0 (controls the initial stiffness)
α	Pyknotropy factor controlling the peak friction angle
β	Pyknotropy factor controlling the shear stiffness
m_R	Stiffness multiplier for initial or reversed loading (stiffness increase for a 90° reversal)
m_T	Stiffness multiplier for neutral loading (stiffness increase for a 180° reversal)
R_{max}	Small strain stiffness limit (radius of elastic range, may be taken as a material constant)
β_r	Parameter adjusting stiffness reduction curve slope (used to calibrate against cyclic test data)
χ	Parameter adjusting stiffness reduction curve slope (taken as material constant)

The hypoplastic sand model with intergranular strain

The hypoplastic sand model with intergranular strain is a non-linear incremental constitutive model. It associates the strain rate to the stress rate, expressing all mechanical behavioural characteristics in one single tensorial equation. The particularity of the model is related to the fact that it does not decompose the strain rate into elastic or plastic parts, as one was accustomed from elasto-plastic theory. The total strain is in fact the sum of two components: an intergranular strain tensor, related to the deformation of interface layers at intergranular contacts and a component related to the rearrangement of the soil skeleton. The first component is observed in reverse and neutral loading conditions, being characteristic to hypoelastic behaviour. In continuous loading conditions both components contribute and the behaviour may be characterised as hypoplastic.

The modified equation by Gudehus (1996) includes the influence of the stress level (barotropy) and of density (pyknotropy). The currently considered standard hypoplastic sand model by Von Wolffersdorff has a Matsuoka-Nakai critical

Table 2 - Overview of soil characteristics

ϕ_c	$\rho_{i,bulk}$	$\rho_{i,dry}$	k_y	e_0	h_s	n	e_{d0}	e_{c0}	e_{i0}	α	β	m_R	m_T	R_{max}	β_r	χ
[°]	[kg/m³]	[kg/m³]	[m/s]	[-]	[GPa]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]
28	2072	1722	2.5E-5	0.54	2.2	0.22	0.4	0.55	0.85	0.13	1.5	1.1	1	1.E-04	0.016-0.75	1

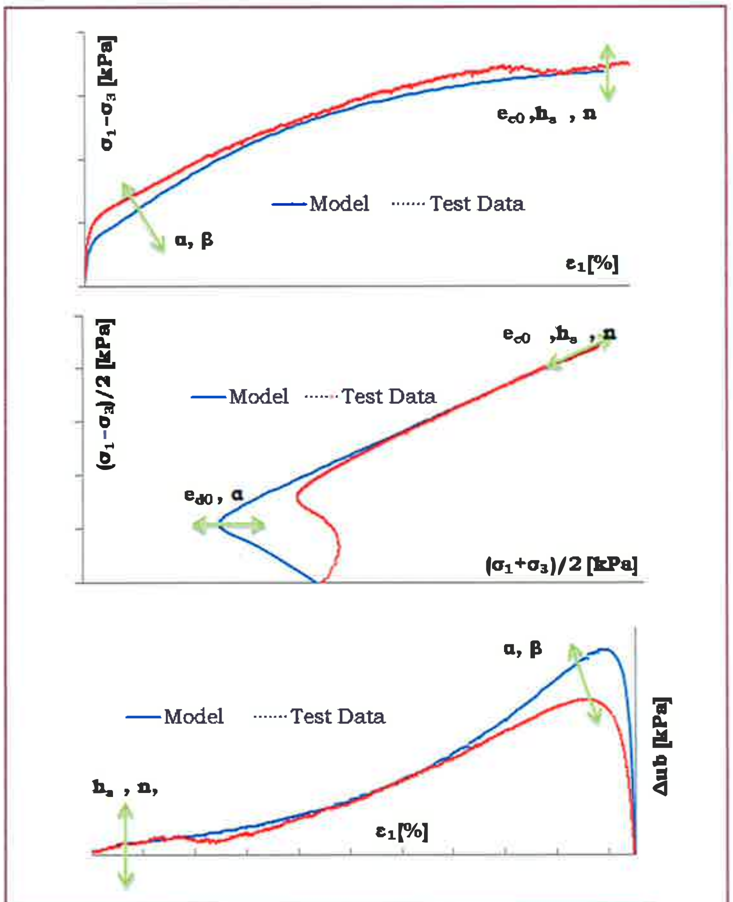


Figure 6 - *Calibration of hypoplastic sand model parameters*

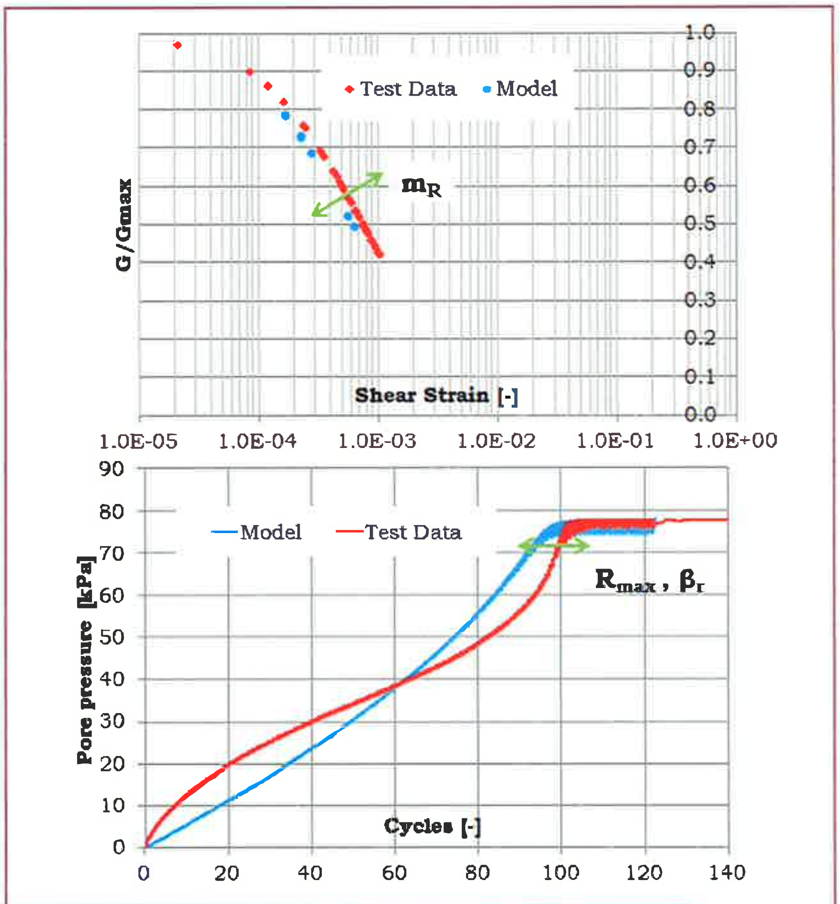


Figure 7 - *Calibration of intergranular strain concept parameters*

stress state condition incorporated. Any material point within the soil mass is described using the stress state and void ratio. Hypoplasticity allows for all the stress history to be incorporated in the current stress, while the presence of the void ratio in the formula makes the model more sensitive to past deformations. According to Kolymbas (2000) hypoplasticity has an algebraic formulation using a simple tensorial relation, whilst the hyperplastic model is a geometrical approach, using pictorial concepts such as a yield and plastic potential surfaces. More information regarding the hypoplastic sand model can also be found in Bardet (1990) and Ling and Yang (2006).

The hypoplastic sand model with intergranular strain, as implemented in Plaxis by Mašín (2012), has a total of 13 parameters to be calibrated, see Table 1. The first eight parameters belong to the hypoplastic sand model and are to be calibrated using isotropically consolidated undrained triaxial test data. The last five parameters, related to the intergranular (small) strain, are to be calibrated using cyclic undrained test data (cyclic undrained direct simple shear tests or cyclic undrained triaxial tests) with varying cyclic shear stress ratios (CSSR).

The calibration procedure requires a good understanding of laboratory test results, soil behaviour and of the influence of the model parameters. The soil testing facilities in Plaxis were used for the calibration of the soil with respect to the existing cyclic laboratory test data– see figure 6 and figure 7.

The values resulting from the in-situ, laboratory testing and calibration procedure are summarised in table 2.

The β_r parameter depends on the applied CSSR (see figure 8). Therefore, it is used as a bridge between the laboratory and in-situ conditions. It ensures that the response of the soil to the applied loading conditions in-situ is correctly extrapolated from the loading conditions in the laboratory.

Extrapolation formulas

The extrapolation formulas are a best fit based on the accumulation trend that is observed in the results generated by running the analyses, as shown in figure 12 and figure 13.

Problem Discretisation

Even though the problem is axisymmetric at boundary value level, the problem was analysed using Plaxis 3D, due to the Expert Mode functionality which allows the user to input all the information using command lines. This feature allows for a more rapid and semi-automatic introduction of the loading conditions.

Geometrically, the problem was discretised in two different ways. First a rectangular soil volume extending 45m around the caisson, with all drained boundaries was considered. Secondly only a quarter of the problem was analysed, with the vertical adjacent boundaries considered undrained – see figure 10. The latest was chosen in order to ensure more symmetry in the automatically generated mesh, as it was observed that the non-symmetry of the mesh was influencing the results. Furthermore, the boundaries of the soil volume were chosen at 6·R (R is the radius of the suction caisson) in order to avoid that they influence the simulation results. For the analysis of this problem interfaces were used between the caisson and the sand volume within and around it.

In order to be able to observe the response of the soil mass to the loading conditions, nodes and stress points were selected both within and

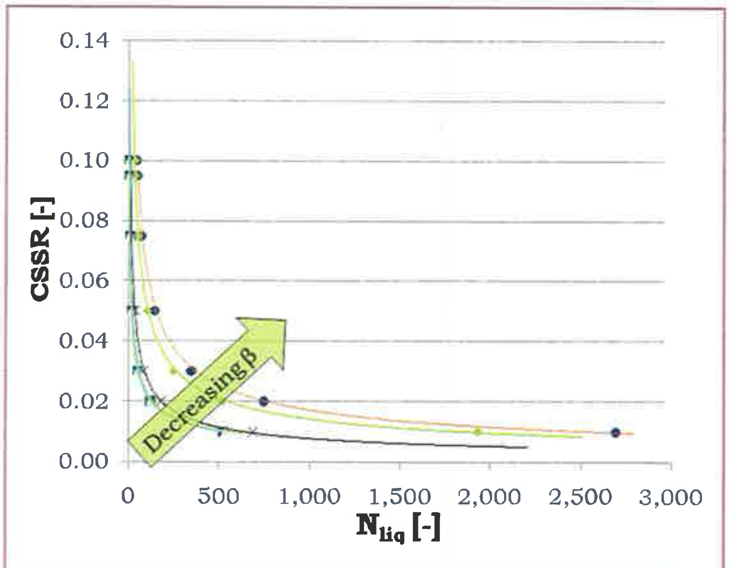


Figure 8 - *Influence of β_r on CSSR and number of cycles required to reach liquefaction (N_{liq})*

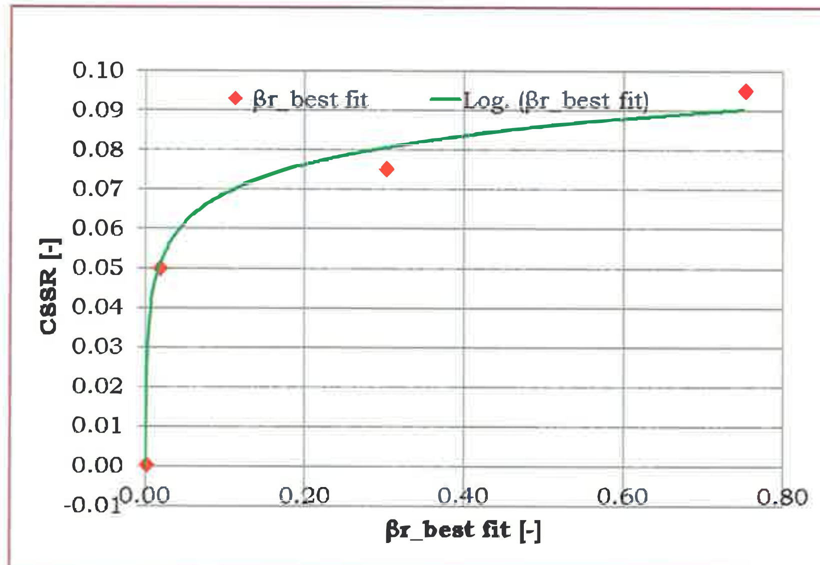


Figure 9 - *Relationship between β_r and CSSR - curve through the values of β_r giving the best fit with laboratory data*

outside the caisson – see figure 11.

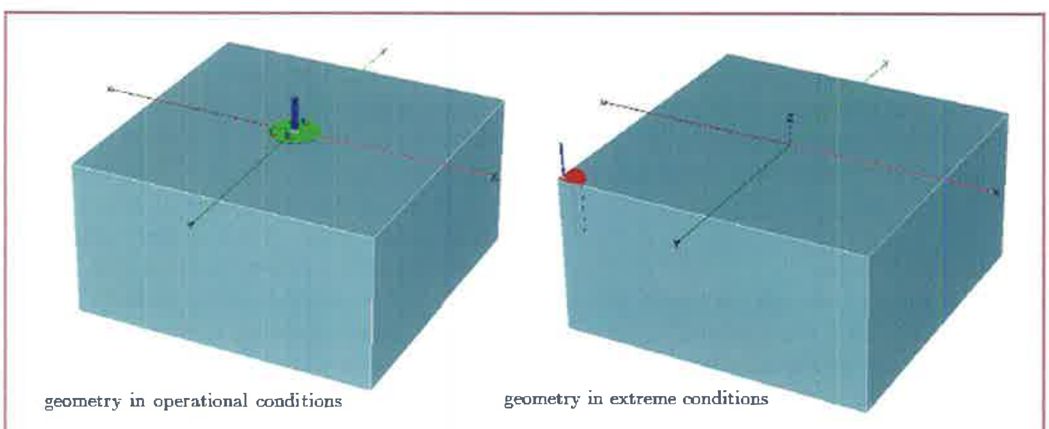
RESULTS

As the applied operational loads were actually under 10% of the predesigned caisson's capacity, the results showed no pore pressure build-up during the course of the 520 cycles analysed (see figure 12); having a symmetric response around the zero mean value at different points within the soil mass). Pore pressure build-up is the driving factor in reducing the bearing capacity of sand and consequently endangering stability as it was defined in the Problem Description. In figure 12 it can be seen that no significant changes occur and therefore the stability of the structure is ensured.

When evaluating the strain accumulation in time (see figure 13) a trend may be observed within two of the points located in the soil mass in the caisson. The accumulation trend for these points represents a linear distribution of $1.1 \cdot 10^{-8}$/cycle. If this trend is extrapolated linearly for one million cycles, a tilt of 0.5° of the full superstructure could be reached.

A linear extrapolation is made over one million cycles only, due to the fact that a significant error may be propagated. Measured laboratory or in-situ data may provide a more accurate extrapolation method.
Having such a small strain accumulation rate, tilting of the structure due to differential settlements is within the boundaries used in the offshore wind industry and, therefore, serviceability is ensured.

An additional analysis was carried out to investigate the foundation's behaviour to more extreme conditions (storm events), based on loads at increased percentages of the capacity. Unfortunately the model response in some of these cases showed significant instability (see figure 15). In these analyses it could be observed that the medium dense sand initially densified within the caisson, but as the cyclic loading continued, localised loosening due to increased pore pressure build-up under the caisson top plate and centre took place. These effects are not expected to be significant in denser sand, according to Andersen (2009).



Figuur 10 - *Problem geometry (in operational loading conditions full caisson, in extreme loading conditions only a quarter)*

The reasons for which instability and overestimation of pore pressure build-up occurs within the software may be related to 3 factors (Plaxis, 2013):

1. Even though the problem is symmetric at boundary value level, the generated mesh is not symmetric and this causes non-symmetric failure mechanisms;
2. The high water depth of 40 m may have also caused unbalanced forces within the software;
3. The over-estimation of pore pressures was already expected from the soil calibration procedure.

Even though for the extreme cases no conclusions could be drawn regarding stability and serviceability, this model allowed for the creation of a chart that allows for a safe design zone (see figure 16), within which long term performance is ensured, by having a volumetric strain accumulation rate smaller or equal to $1.1 \cdot 10^{-8}$ /cycle. This boundary is drawn for a medium dense sand (I_D = 50%), considering a loading period of approximately 7 seconds for the vertical cyclic load and a linear accumulation trend. Moreover, the ballast weight that may be applied on top of the caisson (which provides a positive effect on the axial tensional capacity) was not considered.

It is important to notice from this chart, also found by Jardine et al. (2012), that the foundation response depends on both the mean load and the amplitude of the applied load. Moreover, it is

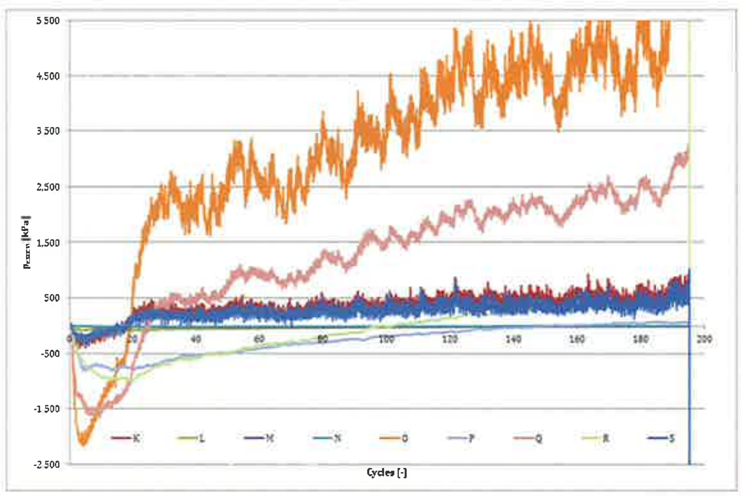


Figure 15 - *Sample of pore pressure build-up instability as a function of the number of cycles for more extreme cases*

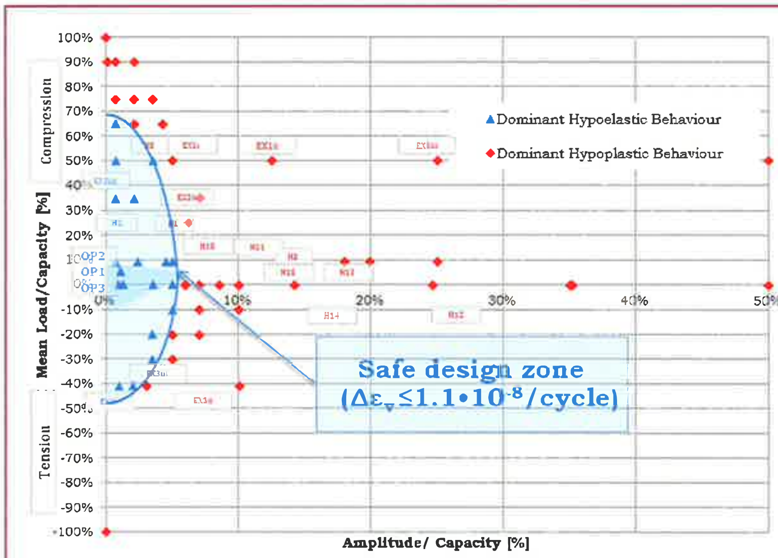


Figure 16 - *Influence of mean load and amplitude on the foundation response for medium dense sand under undrained conditions, with a vertical loading period of approx. 7s (the safe design zone is characterised by dominantly hypoelastic behaviour, while outside of it by dominantly hypoplastic behaviour)*

accuracy of the obtained results. Nonetheless, the boundary as given in figure 16 provides a guideline for a safe foundation design, though it might not always prove economic. This boundary is also an indication of which cyclic loads need to be further investigated in order to assess possible long term damages. The same figure also confirms that the foundation can resist tensile loads, but that capacity decreases significantly as tensile cyclic loading components appear. Taking this tensional capacity into account in the design is an important optimization since in the design of suction caissons up to now all tensional loads are normally avoided by adding ballast on the caissons. Therefore, within certain loading ranges this additional weight might not be required.

The given results are the outcome of the research and numerical analyses conducted as part of the author's M.Sc. thesis. A full overview of all the considered assumptions, limitations and obtained results can be found in Lupea (2013). Nonetheless, the provided results are based purely on numerical analyses, thus field and/or laboratory testing must still be carried out for validation.

ACKNOWLEDGEMENTS

As mentioned in the abstract, this article is based on the work carried out for the M.Sc. thesis of C. Lupea as part of her graduation from the Geo-engineering programme at the Technical University of Delft. Great appreciation is given to dr. ir. R.B.J. Brinkgreve, ir. W.J. Karreman, ir. W.G. Versteijlen and ir. H.J. Luger, whose guidance and support have made all this possible. The authors also thank all the others that have provided input during the course of this research.

REFERENCES

- K.H. Andersen (2009). Bearing capacity under cyclic loading - offshore, along the coast, and on land. the 21st Bjerrum lecture presented in Oslo, 23 November 2007. Canadian Geotechnical Journal, 46(5):513–535.
- J. P. Bardet, (1990). "Hypoplastic Model for Sands." Journal of Engineering Mechanics, 116(9), 1973-1994
- B.W. Byrne and G.T. Houlsby (2003). Foundations for offshore wind turbines. Philosophical Transactions of the Royal Society A. 361:2909–2930.
- S. Bang, K. Jones, K. Kim, Y. Kim and Y. Cho (2011). Inclined loading capacity of suction piles in sand. Ocean Engineering, 38(7):915 – 924.
- European Wind Energy Association (EWEA, 2013). The European offshore wind industry - key trends and statistics 2012.
- Department of Trade and Industry (DTI, 2001). Monitoring & Evaluation of Blyth Offshore Wind Farm.
- G. Gudehus (1996). A comprehensive constitutive equation for granular materials. Journal of the Japanese Geotechnical Society: Soils and Foundations 36(1): 1–12.
- R. Jardine, A. Puech and K.H. Andersen, "Cyclic Loading of Offshore Piles: Potential Effects and Practical Design", in proceedings of 7th int. conf. Offshore Site Investigation and Geotechnics, London, Sept. 2012.
- D. Kolymbas (2000). Constitutive Modelling of Granular Materials. Engineering Online Library. Springer-Verlag. ISBN 9783540669197.
- H. Ling and S. Yang, (2006). "Unified Sand Model Based on the Critical State and Generalized Plasticity." Journal of Engineering Mechanics, 132(12), 1380-1391.
- C. Lupea (2013). Long term effects of cyclic loading on suction caisson foundations. M.Sc. Thesis, Technical University of Delft
- D. Mašín (2012). PLAXIS Implementation of Hypoplasticity. Charles University of Prague.
- A. Niemunis and I. Herle (1997). Hypoplastic model for cohesionless soils with elastic strain range. Mechanics of Cohesive-Frictional Materials 2:279–299.
- Plaxis (2013) private e-mail communication.
- S. Safinus, G. Sedlacek and U. Hartwig (2011). Cyclic Response of Granular Subsoil Under a Gravity Base Foundation for Offshore Wind Turbines. In Proceedings of the 30th International Conference on Ocean, Offshore and Arctic Engineering OMAE, pp. 875–882.
- M. Senders (2008). Suction Caissons in Sand as Tripod Foundations for Offshore Wind Turbines. Ph.D. Thesis, University of Western Australia—School of Civil and Resource Engineering.
- C. Supachawarote, M. Randolph and S. Gourvenec (2004). Inclined Pull-out Capacity of Suction Caissons. In Proceedings of the 14th International Offshore and Polar Engineering Conference, France, pp. 500–506.
- P. von Wolffersdorff (1996). A hypoplastic relation for granular materials with a predefined limit state surface. Mechanics of Cohesive-Frictional Materials 1(3):251–271.

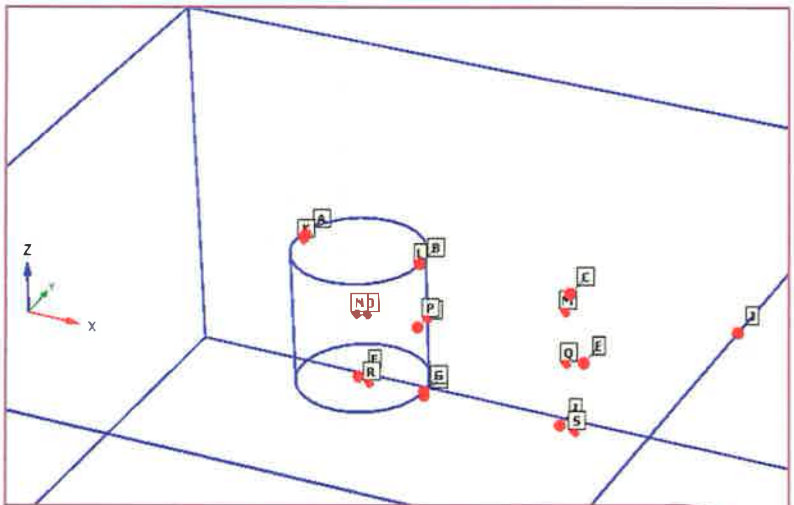


Figure 11 - *Selection of nodes and stress points for measurements*

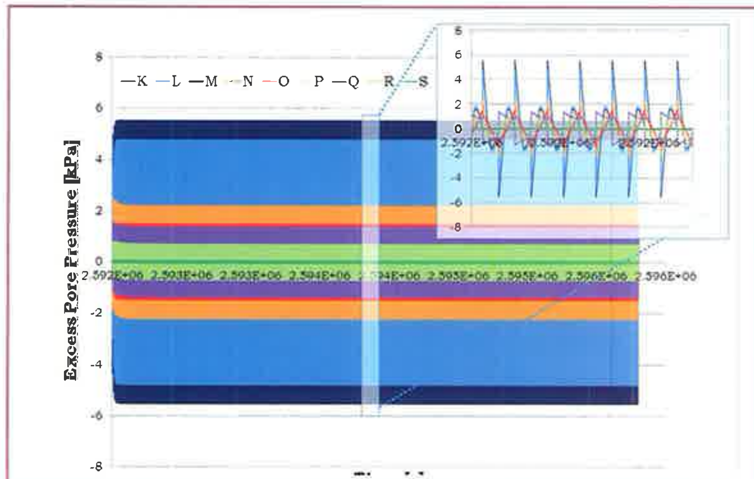


Figure 12 - *Sample of excess pore pressure build-up in time at various points within the soil mass (520 cycles)*

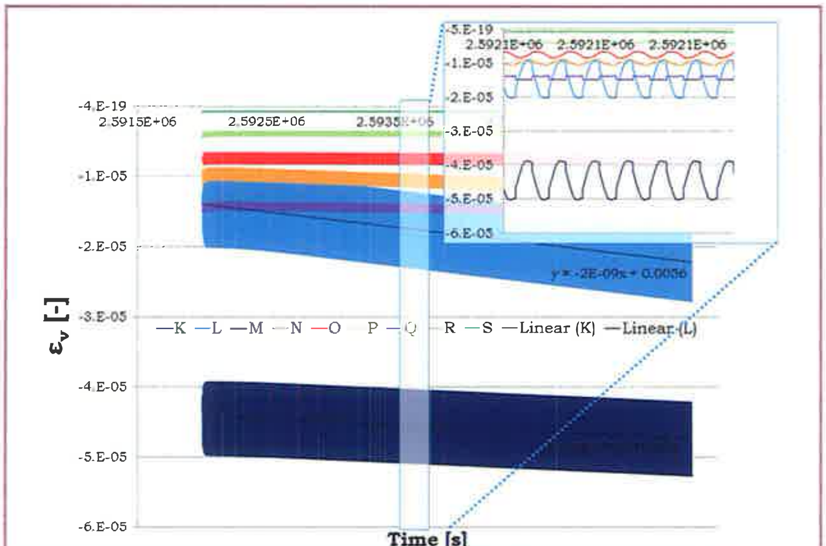
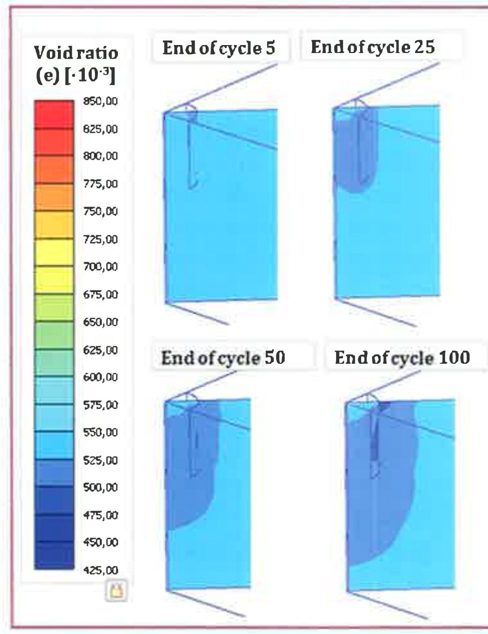


Figure 13 - *Sample of volumetric strain accumulation in time at various points within the soil mass (520 cycles)*

Figure 14 - *Densification effects within the medium dense sand under extreme loading conditions (change in void ratio)*



observed that as loading within the tensile region occurs, due to increased amplitudes and low mean values, the undrained capacity decreases. Yet, this chart shows that there is still a possibility for the foundation to undertake a limited amount of tensile loading, thus reducing the total amount of ballast weight required. A design completely within the safe zone might not be economic, but it can help the engineer identify the cases that will cause neither significant pore pressure build-up nor strain accumulation.

The obtained results represent a first step into having a better understanding of the behaviour of suction caissons embedded in sand under vertical cyclic loading. Additionally, laboratory model testing is still required for validation. Furthermore, the sensitivity of the foundation response to change in relative density of the sand, compressibility, effects of preshearing should also be investigated.

CONCLUSIONS

The problem analysed during the course of this research refers to the long term performance of suction caissons under cyclic loading. This was investigated in order to clarify the adequacy of suction caissons as foundations for large (6 MW) offshore wind turbines.

An important aspect that has been discovered and proved with this investigation is that for a three leg jacket structure, suction caissons have a significant advantage in horizontal resistance: due to the specific geometry of the problem and the choice of large diameter suction caissons, the horizontal resistance is much larger than the applied horizontal loads. Thus, the problem was reduced to the behaviour of the suction caisson under vertical cyclic loading only.

The model proposed to analyse the behaviour under vertical cyclic loading is based on the hypoplastic sand model with intergranular strain concept integrated. It has proven to be a good tool for the prediction of the foundation's behaviour for several hundreds of cycles for specific loading ranges. The soil condition under investigation is represented by sand, as one of the most commonly encountered soil types within the North Sea. The medium dense sand (I_D = 50%) used for the calibration of the soil mass, represents a conservative case, as the encountered sand within the offshore soil investigations in the North Sea is generally dense to very dense. Cyclic loading effects are not expected to be significant in denser sand (Andersen, 2009).

For the obtained results the overestimation of pore pressure, from the soil calibration phase, proves to be problematic in defining the