PLUG HEAVE PHENOMENA DURING PILE INSTALLATION

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ABSTRACT

Recently plug heave (soil rising within open ended tubular piles) phenomena caused, sometimes unexpected, problems in offshore developments. While some had relevant field experience concerning these effects and were prepared to deal with them, others were not forewarned and experienced the consequences at their cost. Surprisingly it turned out that, apart from suction installed piles, the plug heave phenomenon is not mentioned in recommended practices or industry standards, neither is it a topic in research literature.
This paper is a first step to remedy this. Plug heave in long slender piles, where plug heave would not be expected up front by many, is discussed. The difference between static and dynamic installation conditions is shown to be an essential factor contributing to the development of plug heave in these piles. Finally, the use of pile installation templates is demonstrated to have a potential effect on (initial) plug heave. While accurate prediction of plug heave is shown to remain a challenge, providing insight in the mechanisms is easier. This insight is believed to be essential to create awareness and ensure that contractors and designers are not caught by surprise.

Keywords: plug heave, plugging, soil rise, pile installation

INTRODUCTION

The plug, as referred to in this paper, is the soil that, after installation, remains inside an open tubular pile. With slender tubular piles it was often observed that after installation the ground level inside the pile was lower than the ground level on the outside. Often the exact ground level inside an open pile is not a critical issue, but there are situations where the plug position is of interest and can indeed be a critical item. Soil rise or plug heave is a recognized phenomenon and potential risk in suction installed piles where it may result in a rising seabed inside the pile touching the top plate, blocking further penetration to target installation depth. As such the prediction of plug heave is a default assessment in suction pile design and is captured in design guidelines (e.g. DNVGL). Contrary to this, no guidance is provided for other pile types. A so far unpublished risk occurs in situations where jacket piles are pre-installed with limited 'stick-up' (distance between the top of the pile and the original seabed), using a seabed piling template and follower between pile and hammer. There plug heave can lead to followers getting stuck in the pile and requiring significant efforts to remove them (Fig. 1). Subsequent dredging work, required to make room for insertion of jacket legs, has been underestimated both in terms of the volume as well as the composition of the excess soil material, resulting in wrong equipment choices and excess offshore dredging time.

Fig.1. Residue from soil plug in pile (stick-up: 2.4 m) after excessive efforts to remove follower

The lack of available guidance and the fact that various parties have ended up in unwanted situations, requiring substantial mitigation costs in order to being able to continue operations, are reasons to address the phenomenon of plug heave or soil rise in this paper.

Before proceeding it is important to note that the term plug often causes the misconception that the soil inside the pile is completely stuck in its position and cannot move at all. A "plugged" pile would then behave just like a close-ended pile. One should realize that during and after installation the soil inside the pile will deform when the stress level at the pile tip changes and that (partial) slip between pile and soil plug can occur even when one might consider a pile "plugged". The development of a soil "plug" is beneficial where the vertical bearing capacity of an open tubular pile is concerned. A commonly used indicator of soil plug growth during installation is the incremental filling ratio (IFR), defined as (Brucy et al. 1992):

$$IFR = \Delta L / \Delta D \qquad [1]$$

Where ΔL is the increment of the soil plug length corresponding to an increment of the pile penetration depth ΔD [m].

The IFR varies during the installation process since the amount of soil that enters the pile varies during the installation, the length of the soil column in the pile changes due to volume strains of the soil in the plug and changes with variations of wall thickness. During pile installation there may be phases or soil layers in which the pile is close to plugging (where the IFR approaches zero) as well as phases where the IFR is larger than one and the top of the plug rises while the pile penetrates. Another often used indicator (of the end result) is the plug length ratio PLR, which is the integrated value of the IFR over the complete installation process. Figure 2 shows different modes of incremental filling. This figure also introduces the parameter η, which describes how much of the soil volume that is displaced by the steel volume of the pile ends up inside the pile. If conditions near the pile tip are symmetrical inside versus outside the value of η would be 0.5. The figure illustrates that the range of $\eta = 0$ (the displaced soil ends up outside the pile) to $\eta = 1$ (the displaced soil ends up inside the pile) does not cover the whole range of possible pile filling modes.

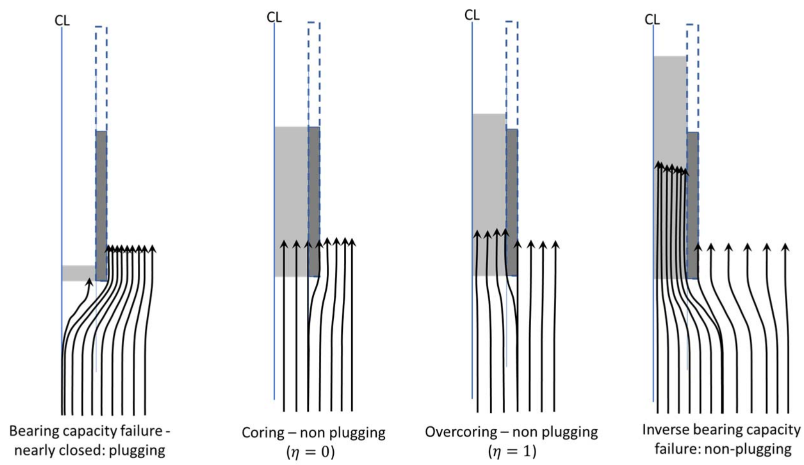

Fig. 2. Modes of incremental filling of the pile: Ranging from IFR<<1 (pile base bearing failure), to IFR=1 (coring), IFR>1 (over-coring) and IFR>>1 (inverse pile base bearing failure)

In literature, only a very limited number of plug heave events are reported (Paikowsky et al. 1989; Shioi et al. 1992; Nicolini et al. 2015), mostly hidden among many more plug settlement events. Only Shioi (1992) provided enough background information of the extent of plug heave that occurred including information about pile geometry and soil stratigraphy. Mechanisms resulting in observed plug heave were not discussed. This may be due to the plug heave not being relevant in the few reported cases.

FACTORS DETERMINING THE IFR AND ULTIMATE PLUG POSITION FOR STATIC INSTALLATION

When a pile is pushed into the ground, the soil that enters the pile generates a shear stress that is proportional to the radial effective stress times an interface friction factor in purely non-cohesive soils and proportional with the interface adhesion in non-frictional, cohesive soils. This shear stress increases the vertical total stress towards the base of the soil plug. Under fully or partially drained conditions in frictional soils this higher vertical stress leads to an exponential increase of the effective radial stress towards the pile tip and thus a much higher friction than could be mobilized with the initial horizontal stress level.

This effect significantly increases the resistance of the soil plug against being pushed upwards into the pile and is very beneficial for the static compressive bearing capacity of an open tubular pile.The effect is less pronounced in cohesive soils or under undrained conditions in frictional soils. Since the shear stresses that are transferred from the inside pile wall to the soil can only spread over the inside pile area, while the shear stresses transferred from the outside pile wall can spread radially away from the pile, the vertical stress at pile tip level will be higher inside than outside the pile.

To illustrate the range of IFR values that can be encountered for static penetration into a homogeneous undrained material (clay) FE calculations were performed whereby the vertical stress inside and outside was varied by application of a surcharge at a distance of one diameter (D_p) above the pile tip. The analyzed pile had an outer diameter of D_p = 3.048 m and a wall thickness of t = 80 mm and was therefore relatively thick-walled (D_p/t = 38). This implies that (not counting volume changes in the soil plug itself) for $\eta = 1$ the IFR would be equal to $(D_p^2/(D_p - 2t)^2)$ = 1.114. The stress difference ΔP was divided by the undrained shear strength S_u to make the results non-dimensional regarding the stresses. The results are shown in Fig. 3 with the red and green lines denoting the $\eta = 0$ and $\eta = 1$ coring conditions.

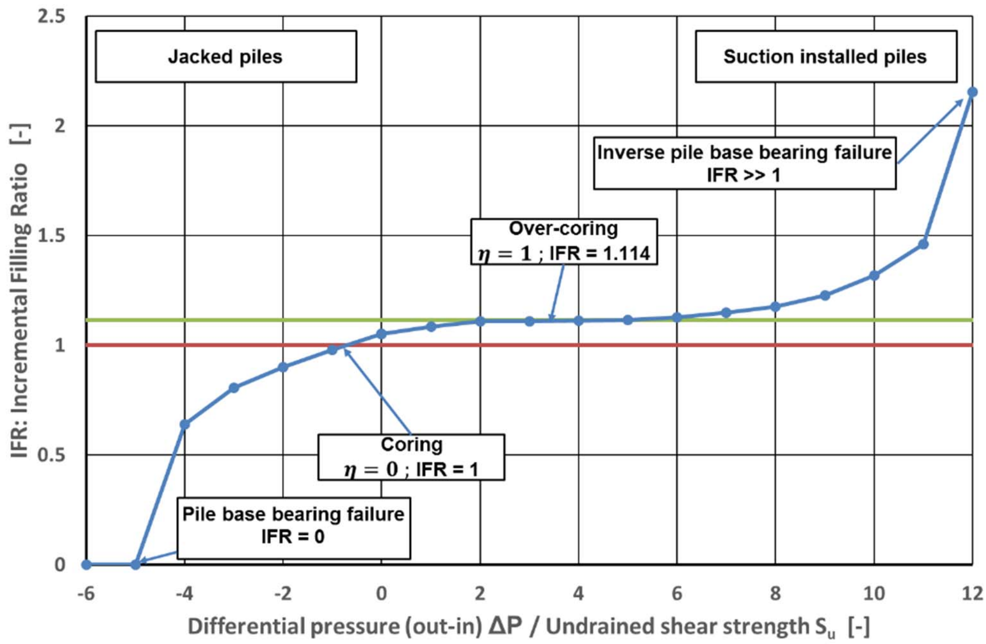

Fig. 3. FE-results for pushed-in undrained pile installation in clay: installation conditions ranging from jacked plugging ($\Delta P/S_u < -5$) to excessive heaving suction installation ($\Delta P/S_u > 12$).

When an open pile is installed, the edge of the pile penetrates the soil and the material below pile tip will end up either on the outside or the inside of the pile. The amount of material that ends up inside the pile and any changes in volume that this material experiences determine the volume of the soil plug.

Soil-steel replacement (soil displacement)
A way to estimate plug heave from soil-steel replacement is to assume that a portion η of the replaced volume ends up inside the pile (a first estimate would typically be $\eta = 0.5$). The plug volume V_{plug} is then found by summation over all (n) layers that are penetrated by the pile:

$$V_{plug} = \sum_{i=1}^{n} \left\{ \left(A_p + \eta_i A_s \right) h_i \right\} \qquad [2]$$

Where A_p = inside pile area at tip level [m^2], A_s = pile wall (steel) cross sectional area at tip level [m^2], η_i = part of the soil volume in layer i, displaced by the pile wall, that enters into the pile [-] and h_i = pile embedment in layer i [m]. By distributing the volume V_{plug} from the tip upwards over the available volume inside the pile the resulting level of the top of the soil plug inside the pile can be estimated.

This approach does not consider the fact that stresses on the inside and outside of the pile can be very different, which leads to other η ratios. The static analysis of the stress levels and differential stresses around the pile tip during installation (although valid for jacked and suction installed piles) does not suffice for driven piles. There dynamic effects are dominant, shear stress reversals occur during and after the blow, partially drained and undrained soil behavior occurs and residual stresses and unloading-reloading effects should be considered. For this, dynamic numerical analyses allowing for large strains and incorporating sophisticated constitutive models are required.

Strains and volume changes in the plug
Apart from the deviatoric strains that the soil which is squeezed into the pile experiences, also volumetric strains occur. Deviatoric strains occur due to shearing in close vicinity of the pile wall, due to contraction upon entering the pile and later lateral expansion or compression when passing through parts of the pile with varying diameter, e.g. when progressing past a thickened section of the pile wall (e.g. a driving shoe). Deviatoric strains and cyclic loads lead to volume changes provided enough time is allowed to let pore pressures dissipate.

When driving in an alternating soil profile consisting of both loosely packed sand layers (likely to exhibit compaction) as well as densely packed sand layers, positive and negative volumetric strains may partly cancel each other out. This could lead to a preliminary conclusion that for soil profiles existing for a large part of densely packed sand layers, soil heave due to volumetric strains resulting from loosening effects may play an important role in the prediction of plug heave, whereas for soil profiles consisting of a balanced mix of loosely packed and densely packed sand layers, no significant plug heave due to changes in volume is to be expected. Also, permeable layers that are laterally confined (cut-off) by the pile wall penetrating through adjacent impermeable layers, could also result in insufficient pore pressure dissipation within the time represented by impact loads from the piling hammer. As such, these layers may not exhibit the full extent of volume change that could occur in drained conditions. This is supported by the fact that especially loosely packed sand layers with clay/silt lenses experience significant pore pressure buildup and hardly to no compaction during pile installation.

Effects from dilation
For dense and/or strongly overconsolidated (OC) soils, shearing along the pile wall but also shear deformation that is imposed when the soil is “squeezed” into the pile, is associated with dilatant behavior. With dilatancy, an increase in volumetric strain can be expected when the material is not constrained and dissipation of negative pore pressures (suction) takes place. For dense, non-cohesive materials with a dilatancy angle between 5 and 15 degrees, approximately 10 to 25% of the axial strain could be converted to volumetric strain. In situations in which only a small zone within the direct vicinity of the pile annulus is exposed to

shearing/dilation (in coring mode), one finds that for piles often used in offshore jacket structures (D_p/t typically 40 – 80) dilation will contribute to a small extent to plug heave. For OC soils with higher K0 values however, the zone affected by dilation may extend over a greater distance from the pile wall, therefore contributing to a larger extent to plug heave.

Consolidation
Finnie and Randolph (1994) provided insight in whether a soil acts in a drained, partially drained or undrained manner during penetration of a penetrometer cone. They found that its behavior can be described by a non-dimensional velocity parameter:

$$V = \frac{vD_p}{C_{vh}} \qquad [3]$$

Where V = non-dimensional velocity [-], v = penetration velocity [m/s], D_p = diameter [m], C_{vh} = vertical or horizontal consolidation coefficient [m^2/s]. It was found that transitions in material behavior lie around $V < 0.1$ (drained) and $V > 10$ (undrained). Intermediate values would lead to a state in which partial consolidation takes place.

It can be determined that when taking into account a typical range in consolidation coefficients for sands, a typical diameter of open ended jacket piles, for a calculated velocity of the pile toe penetrating the soil (the set per blow divided by the duration of the blow), the soil behavior may vary from undrained (low consolidation coefficient combined with large set per blow; $V > 10$) to partially and even fully drained (high consolidation coefficient combined with small set per blow; $V < 1$). This would mean that for loosely packed sands with low permeability and a large set per blow (easy driving) no direct dissipation and thus volumetric strain would occur whereas for densely packed sands with high permeability and a small set per blow (hard driving) volumetric strains could develop during penetration.

Furthermore, a typical blowrate for hydraulic hammers (35 to 50 blows per minute) results in a time between the strokes in the order of 1 to 1.5 seconds, which would allow for additional dissipation. This could mean that soil which behaves (practically) undrained during the actual blow (order of magnitude 10 ms; $v \approx 0.5 - 2.5$ m/s), undergoes a volume change due to dissipation in the period between the actual hammer blows.

In case dissipation of negative pore pressures in dilative soils cannot take place during pile installation itself, for example when permeable layers are cut-off by the pile penetrating through impermeable layers, this dissipation could still develop over mid to long term. As such, even when no plug heave was observed during or immediately after driving, gradual plug heave could manifest itself over time.

EFFECTS OF DYNAMIC INSTALLATION AND LOADING

Contrary to the static installation case of jacked or suction installed piles a driven pile will, between the hammer blows, reach a status of nearly stationary conditions whereby the soil only carries the weight of the pile and possibly part of the pile driving equipment (e.g. follower and hammer). To demonstrate the difference between jacking and driving a dynamic FE analysis was made for a pile with cross section that was similar to those used for the static analysis presented in Fig. 3. The pile, driven to 20 m depth, is given a 10 ms blow resulting in 23 mm extra penetration. The left hand and the right hand side of Fig. 4 show the vertical displacement of the tip during the first 0.025 s respectively the first 0.4 s after the start of the blow. The pile tip starts to move downwards before the soil next to the pile tip starts to move.

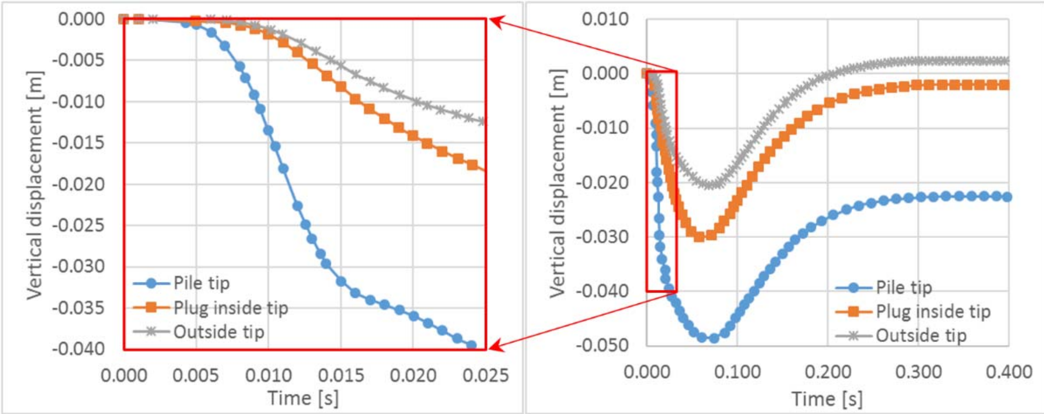

Fig.4. Vertical displacement of pile tip and neighbouring points in the soil after a hammer blow

The huge differences between the vertical total stresses under the driven pile at t=0 s and t=0.06 s on the one hand and the jacked pile on the other hand are shown in Fig. 5 below.

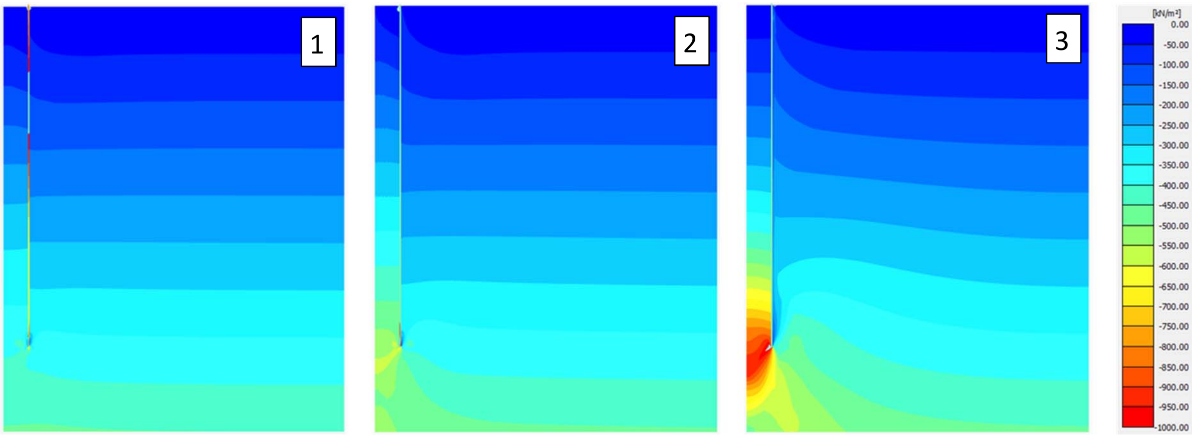

Fig.5. Vertical total stress: 1. driven pile t=0s, 2. driven pile t=0.06s and 3. jacked pile after 0.25 m push.

During the rapid advancement of the pile tip, after having received the hammer blow, the vertical total stresses inside and outside do differ, but much less than when the pile is jacked. This explains why the driven pile is hardly plugging compared to the jacked pile. In cases where the radial outward displacement of the soil is hampered by high horizontal stresses, as well as a higher stiffness and higher strength at pile tip level, the IFR values will increase and plug heave can occur as onsite experiences, as reported below, confirm.

OBSERVATION ON SITE AND ANALYSIS

Public project data indicating occurrences of plug heave are scarce. Within literature, only one case provided sufficient background information to verify hypotheses relating to mechanisms that could play a role in the prediction of plug heave (Shioi et al. 1992). Besides that, non-public field data from pile installation for an offshore wind farm was available to the authors for which at a large number of locations plug heave occurred.

Plug heave analysis offshore wind farm data

In order to understand the mechanism of plug heave, the authors analyzed an offshore wind farm for which a vast amount of information was available comprising soil investigation data (CPTs, Boreholes, laboratory tests), pile data (pile geometry, pile installation records) as well as as-built survey data (measured levels of soil in the piles, prior to dredging works).

Some general specifications:

- Complete data entries totaling approximately 200 piles over all jacket locations
- ≈ 3 m diameter open ended piles, with D_p/t in the range of 55 – 70
- Pile penetrations ranging from 20 to 50 m below seabed

Soil conditions comprise soft holocene sediments with underlying glacial till (both highly OC clays as well as densely packed sands/gravels), structureless chalk (grade D, UCS ≈ 0.1 MPa) followed by structured low to medium density chalk (grade A, UCS ≈ 0.5 MPa). First predictions, based on an initial assumption of $\eta = 0.5$, produced a relatively large error when comparing postdicted to measured plug heave (Fig. 8) with the trend line showing an offset compared to the ideal 1:1 relation between predicted and observed plug heave. Further study indicated that the Holocene was relatively strongly correlated to the total amount of plug heave. This seemed remarkable since that layer is relatively thin (on average 1.5 m) compared to the total embedment. Upon further searching for the possible mechanism causing the impact of the top layer, the influence of the piling template was assessed.

Piling operations would start once the piling template was placed on the seafloor, preloaded per mudmat such that the preload would exceed the ULS load under operational conditions, and corrected within requirements in terms of horizontality. With this preloading, the mudmats would penetrate, depending on thickness and strength of the Holocene layers, up to 1 m. Due to the shape of the piling template's mudmats, being a rectangular footing with a centralized opening (4 x 4 m) in which the piles are stabbed and subsequently driven, any initial soil displacement in the soft Holocene sediments would partly emerge as an upward soil movement within the centralized opening. For the geometry of the mudmats of the piling template, simple analytical predictions were found to agree with 3D FE analyses (Fig. 6): for this geometry approximately 1.5 times the mudmat penetration was emerging as soil heave within the centralized opening. A mudmat design in which the skirt height around the centralized opening is large compared to the skirt height on the outside perimeter of the mudmat could reduce this soil heave. The same applies to a tapered footing which forces soil displacement towards the outside of the mudmat during penetration.

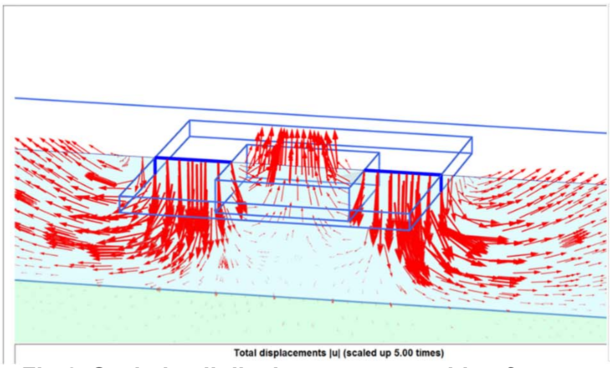

Fig.6. Scaled soil displacements resulting from penetration of mudmat

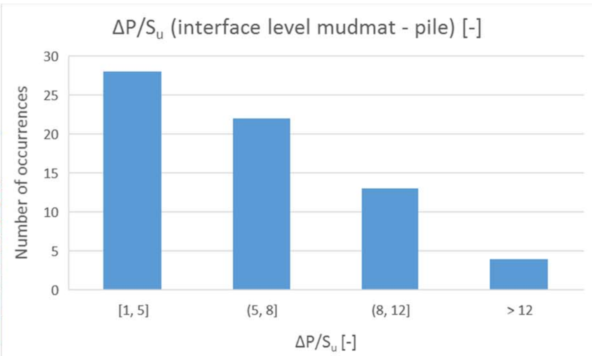

Fig.7. Distribution ΔP/S_u at interface mudmat/pile over jacket locations

Accounting for this the overall quality of the prediction was greatly improved (Fig. 9). However the scatter between predicted and measured values remained quite large. By means of back calculation, a best fit was acquired over the various soil units by varying the parameter η after:

$$\eta_{fitted} = \eta_{initial} * multiplier$$

where η_{fitted} is the back calculated soil volume distribution factor and $\eta_{initial}$ is the initially assumed 50/50 soil volume distribution on the inside and outside of the pile. Table 1 presents the outcome of this assessment.

Table 1. Back calculated multipliers over η for various soil units

Soil unit	Plug heave, corrected for template penetration	Plug heave, corrected for template penetration, loosening (ΔR_e) and dilation (ψ) effects
	R^2 = 1.00; Σ_{res} = 49	R^2 = 1.00; Σ_{res} = 43
	Multiplier over $\eta_{initial}$ = 0.5	Multiplier over $\eta_{initial}$ = 0.5
Holocene	1.01	1.00
Glacial Till	1.33	0.95
Fluvo Glacial Till	1.07	0.99
Chalk grade D	1.06	0.99
Chalk grade A	1.17	0.97

From the back calculation process followed that the soil units just below the mudmat (mostly the top Glacial Till layer; Table 1, second column) were found to be important to the total plug heave. Fig. 3 shows that one can expect the IFR to be in excess of 1 for $\Delta P/S_u$ > 2 to 4. A large number of locations had a $\Delta P/S_u$ in the range of 5 to 12 in the shallow layers due to the presence of the increased stress below the mudmats (Fig. 7). This means that within the influence zone of the mudmat stress an increased η-factor is to be expected. Moreover, at multiple pile locations the Glacial Till comprise very dense sands (mixed with gravels) that are likely to exhibit dilation and a reduction in relative density (loosening) from pile installation.

To investigate to what extent effects from volume changes within the soil plug would improve the postdictions, earlier experiences with suction pile installation in loose to very dense sands were incorporated in terms of initial relative density (R_e) and expected change in relative density (ΔR_e). A range of ΔR_e = 0 to +5% (compaction) for (very) loose material to ΔR_e = -10% to -20% (loosening) for (very) dense material was implemented. It is assumed that for the given soil conditions and pile geometry, sufficient dissipation of pore pressures could develop to facilitate volumetric strains induced from the (cyclic) pile installation in the permeable layers.

When the impact of a potential change in relative density and effects of dilatancy (Ψ) in the densely packed Tills are considered, the back calculated multipliers over η move more towards 1.0 (Table 1, right column), although effects in the Glacial Till layer seem to overshoot a bit.

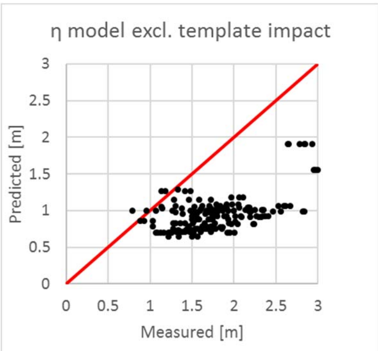

Fig. 8 Case excluding template impact
(η = 0.5; R^2 = 0.54; $\Sigma_{residual}$ = 158)

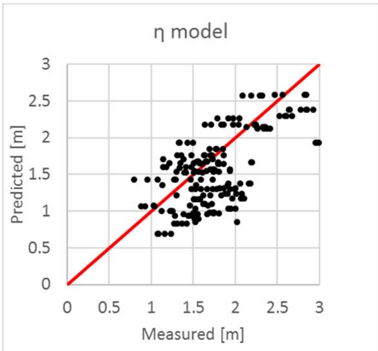

Fig. 9 Case including template impact
(η = 0.5; R^2 = 0.88; $\Sigma_{residual}$ = 52)

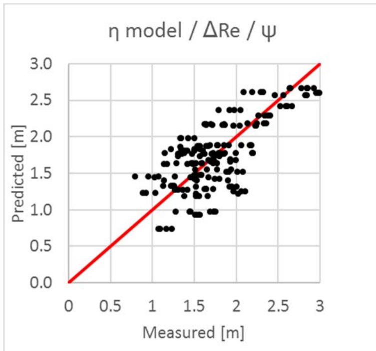

Fig. 10 Case including template impact, ΔRe and dilation effects
(η = 0.5; R^2 = 1.01; $\Sigma_{residual}$ = 44)

Figure 10 presents an overview of the model's performance including effects from soil volume change. Use has been made of η = 0.5 for all soil layers. Even though the prediction of plug heave is improved, the sum of residuals remains relatively large. At some jacket locations, significant challenges with followers being stuck in the pile (plug) occurred due to excessive plug heave and limited stick up length. This prevented proper measuring of the actual plug rise and these locations could therefore not be included in the analysis. An overview of measured versus predicted plug heave according to the various models is presented in Fig. 11.

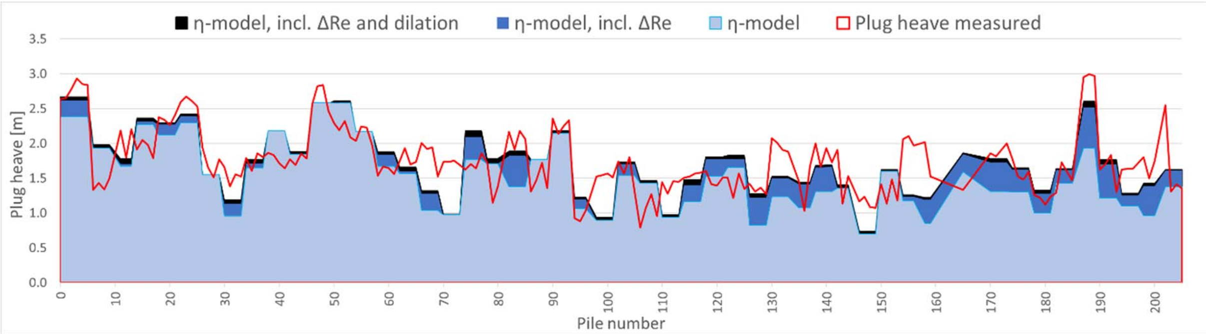

Fig. 11 Measured versus predicted plug heave after various models

When using the simplified approach as described here to arrive at a cautious estimate for plug heave, it is recommended to implement an uncertainty factor (safety factor). From the observed scatter a factor in the order of 1.5 can be a reasonable estimate to arrive at upper bound design values.

Plug heave analysis Trans-Tokyo Bay Highway

The mechanisms as described above, were used to see if back calculation of plug heave at the Trans-Tokyo Bay Highway (Shioi et al. 1992) would deliver a reasonable estimate. Shioi reported the following data: a typical CPT profile (see Fig. 12), pile geometry (D_p = 2.0 m, t = 34 mm; D_p/t = 59), penetration (31 m) and a measured plug heave (2.4 m).

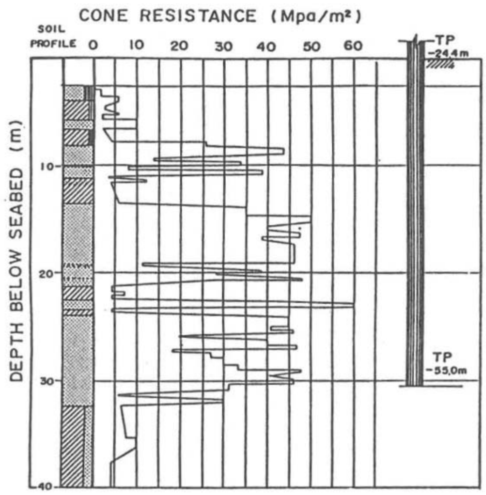

Fig. 12. Typical CPT profile [Shioi et al. 1992]

For a large part of the embedded pile length, very dense sand layers are present. Loosening effects in these strata have been considered, whereas no volume expansion in the very stiff clays have been incorporated (although these layers may exhibit heave as well). Processing this data resulted in the following estimates:

Contribution of steel/soil displacement (η = 0.5):	1.10 m
Contribution of compaction and loosening (ε_v(*) = 2.9 to 4.5%):	0.70 to 1.40 m
Contribution of dilation effects (ψ = 7 to 16 deg; ε_v = 0.2 to 0.3%):	0.05 to 0.10 m
Together total predicted plug heave range is approximately:	1.9 to 2.6 m

Note(*): ε_v is volumetric strain. Range of values based on an $e_{max} - e_{min}$ from 0.3 to 0.6.

Although no information on type of sand was available to the authors, within a specified uncertainty bandwidth (of which the difference in minimum and maximum void ratio is an important driver), it followed that the actual plug heave of 2.4 m fell within the predicted range of 1.9 to 2.6 m.

SUMMARY AND CONCLUSIONS

Although there are a handful of (largely unpublished) cases known where plug heave phenomena of impact driven open ended piles have resulted in challenges offshore, no guidance is available in codes, guidelines and literature how to predict plug heave. By writing this paper, the authors have tried to create awareness of plug heave and potential consequences to design and operations. While accurate prediction of plug heave is shown to remain a challenge, providing insight in the mechanisms that may play a role in plug heave as well as providing insight in the uncertainty of these factors is easier.

It is demonstrated that the degree of plug heave depends on the type of installation. The driving parameter is the differential pressure between the outside and inside of the pile. Extremes are found for non-driven piles and range from plugging jacked pile to suction installed piles that experience reverse end bearing failure. For driven piles dynamic FE analyses demonstrated that, even though stresses on the in- and outside of the pile do differ during rapid advancement of the pile tip, inertia of the soil near the pile tip tends to result in an η factor around 0.5.

It was shown that one can arrive at a reasonable estimate of plug heave assuming $\eta = 0.5$ and accounting for potential volume changes in the soil plug. Use of a piling template on a softer seabed may lead to an 'initial' extra heave followed by larger filling ratios over the first self-weight penetration stage of the installation. This phenomenon might not only be limited to prepiling templates, but could also be relevant in cases where piles are driven in close vicinity of spud cans when using a jackup vessel. Based on the available field data it is concluded that when using $\eta = 0.5$ to arrive at a best estimate of plug heave, a multiplier in the order of 1.5 can be a reasonable estimate to arrive at upper bound design values.

Authors acknowledge that further investigation is to be carried out in order to improve insights and subsequently improve predictions. Any contribution from parties that wish to provide useful data from projects and/or wish to participate in this field is therefore greatly encouraged.

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