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Numerical modelling of soil-pile interaction in a subsiding soil

Modélisation numérique de l'interaction sol-tas dans un sol affaissé

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ABSTRACT: This paper presents a suitable constitutive and interface model of soil-pile interaction in a subsiding soil (gyttja) using PLAXIS 2D software. The model was calibrated by simulating the results of soil settlement, excess pore water pressure, drag force and unit shaft resistance along the uncoated steel test pile obtained in the field triggered by placing a 2 m high fill embankment around the piles. Such calibrated model could serve as a basis for further numerical simulations of the soil-pile interaction for both single-pile and pile groups.

RÉSUMÉ: Cet article présente un modèle constitutif et d'interface approprié de l'interaction sol-tas dans un sol affaissant (gyttja) à l'aide du logiciel PLAXIS 2D. Le modèle a été calibré en simulant les résultats du tassement du sol, de la pression excessive de l'eau interstitielle, de la force de traînée et de la résistance unitaire de l'arbre le long du pieu d'essai en acier non revêtu obtenu sur le terrain, déclenché en plaçant un remblai de 2 m de haut autour des pieux. Un tel modèle calibré pourrait servir de base à d'autres simulations numériques de l'interaction sol-pieu pour des groupes de pieux simples et de pieux.

Keywords: PLAXIS 2D; soft soils; interface; drag force; instrumentation.

1 INTRODUCTION

An expansion of the Randers Harbour, Denmark, is forthcoming. The first step of the site development plan was to employ vertical drains across a vast area to accelerate consolidation process. Later steps will involve piles installed through compressible gyttja surcharged by fill. Placing fill induces soil subsidence and develops negative skin friction along piles (Fellenius, 1984). Therefore, an experimental programme has started in the Randers Harbour to investigate force distribution along uncoated and bitumen coated, steel and precast concrete driven piles installed in gyttja (Kania et al., 2023). Vibrating wire piezometers, magnetic extensometers and strain gauges along piles were employed.

The aim of this paper is to validate a numerical soil and interface model of soil-pile interaction using PLAXIS 2D software (Bentley Systems, 2022). The soil model was calibrated by simulating field results obtained after placing a 2 m high fill embankment. The soil-pile interaction was examined focusing on the results from the uncoated steel pile.

2 MATERIALS AND METHODS

2.1 Soil and ground water conditions

The boreholes and results of two CPTU soundings carried out prior to the pile installation indicated a soil profile consisting of an 1.1 m old fill underlain by an 8 to 10 m thick gyttja (a post-glacial organic clayey sandy silt of marine origin) layer on top of a post-glacial, marine, medium dense sand. The ground water table was located 0.4 m below the ground surface.

The average unit density, ρ, of gyttja was 1,430 kg/m³ ranging from 1,390 to 1,510 kg/m³. The water content, w_n, ranged from 70% to 110% and was close to the liquid limit. The undrained shear strength, $c_{u;FVT}$, determined from the field vane shear tests (adjusted by plasticity index factor of $\mu = 0.77$) ranged from 13 to 35 kPa with an average of 21 kPa. Consolidated undrained triaxial tests indicated an angle of friction $\varphi'_{tr} = 34°$ and an effective cohesion $c' = 8$ kPa.

Two oedometer tests performed on specimens from 2.6 and 4.5 m depth showed an average compression index, C_c, of 1.14, swelling index, C_s, of 0.15 and creep index for secondary compression, C_α, of 0.014. An average initial void ratio, e_{init}, was 2.9. Both specimens were found to be overconsolidated with an average preconsolidation stress of about 20 kPa, which corresponds to an *OCR* at 2.6 and 4.5 m depths of about 2.1 and 1.7, respectively. An average coefficient of permeability, *k*, between initial and final (after placing the fill) effective stress was $9.5 \cdot 10^{-10}$ m/s.

2.2 Test setup and programme

Two instrumented Ø 406 diameter (8 mm wall thickness) closed-end steel piles (with and without bitumen coating) and two instrumented 400 mm square precast concrete piles were installed at the test site. All piles were instrumented with vibrating wire strain gauges and distributed fibre optic sensors. The steel and precast concrete driven piles were installed to the approximate depth of 13.8 m and 12.8 m, respectively.

The ground monitoring system consisted of vibrating wire piezometers and magnetic extensometers was installed at different depths prior to the pile installation.

The test programme consisted of two phases. The first phase focused on the pile installation effects and the second phase on placing of the fill. The embankment footprint area was 10 x 19 m and a slope of 45°. All piles were sleeved using a plastic pipe prior to placing of the fill to avoid load transfer from the fill to the piles. This paper addresses only the effects of the second phase.

Details about the test setup and programme are reported in Kania et al. (2023).

2.3 Numerical model

PLAXIS 2D V22.02 (Bentley Systems, 2022) was used for the numerical analysis. The uncoated steel test pile (STP1) was considered as a single pile in an axisymmetric model. The model had a radius of 1.5 m corresponding to the half spacing between the piles. The lower boundary was located 10 times the pile diameter below the pile toe elevation. A 15-noded triangular mesh elements with medium distribution and enhanced mesh refinement were employed in the model. The obtained average mesh quality was 0.9.

The soil stratigraphy and parameters are shown in Table *1*. The old fill and sand layer were modelled using a drained Mohr-Coulomb soil model. Their parameters were adopted from Belinchon et al. (2016), who performed a numerical analysis of a previous test setup located next to the one described in this paper. The gyttja layer was modelled using an undrained (type A) Soft Soil Creep model following the observations from the oedometer tests regarding the secondary compression. The model is described by modified compression index, $\lambda^* = \frac{C_c}{(1+e_0)\ln 10}$, swelling index, $\kappa^* = \frac{C_s}{(1+e_0)\ln 10}$, and creep index, $\mu^* = \frac{C_\alpha}{(1+e_0)\ln 10}$. An average coefficient of permeability obtained within the change in effective stress was initially used in the model. The flow conditions along the vertical boundaries of the model within gyttja layer were set as "closed" to prevent water flow across it.

Table 1. Model soil stratigraphy and parameters.

Parameter	**Old fill**	**Gyttja**	**Sand**
y_{min} [m]	0.0	-1.1	-9.5/-10.9$^{1)}$
y_{max} [m]	-1.1	-9.5/-10.9$^{1)}$	-17.86
γ [kN/m^3]	18.0	14.3	16.0
e_{init} [-]	0.5	2.898	0.5
E'_{ref} [kN/m^2]	10000	-	30000
ν [-]	0.25	0.15$^{2)}$	0.3
c'_{ref} [kN/m^2]	6	8	1
φ' [°]	25	34	35
k_x [m/day]	0.6	$0.0821 \cdot 10^{-3}$	0.6
k_y [m/day]	0.6	$0.0821 \cdot 10^{-3}$	0.6
λ^* [-]	-	0.1269	-
κ^* [-]	-	0.0333	-
μ^* [-]	-	0.0016	-

$^{1)}$ Two different levels due to inclination of the layer

$^{2)}$ ν_{ur}

The steel pile was modelled as a solid cluster using a Linear Elastic material model with an equivalent saturated unit weight of 16.7 kN/m^3 (to represent a tubular steel pile) and stiffness of 210 GPa.

Interface elements, to simulate the soil-pile interaction, were created by adopting the method used by Belinchon et al. (2016). They were placed along the vertical and horizontal pile cluster limit and extended by 0.5 m in both directions at the pile toe level in order to avoid non-physical stress oscillations. The soil-pile interaction was defined by a strength reduction factor R_{inter}.

The fill was represented by uniformly distributed load with a magnitude of 36 kPa (applied in two 18 kPa load steps simulating half- and full completion of the fill embankment) located along the top, horizontal boundary between the pile cluster and the vertical boundary of the model.

The first construction stage represented the initial conditions. The second stage simulated completion of a half of the fill embankment (i.e. 1 m high). The next stage was a short (1 day) consolidation stage before a full embankment fill was completed (i.e. 2 m high). The last stage was a consolidation stage until all excess pore pressures in the geometry have reduced to a predefined minimum value of 1 kPa.

3 ANALYSIS

To calibrate the soil model, the dissipation of excess pore water pressure and ground settlement obtained in the field and in the finite element model were compared. In order to assess the soil-pile interaction the strength reduction factor was modified. Due to inclination of the gyttja layer, the bottom of the layer for calibrating the soil and soil-pile interaction model was assumed at 9.5 m and 10.9 m depth, respectively.

3.1 Excess pore water pressure dissipation

The measured (at 5.8 m depth) and simulated dissipation of excess pore water pressure with time after starting construction of the fill embankment is presented in Figure 1 The field records show unexpectedly low maximum excess pore water pressure of 17 kPa at the end of constructing the fill embankment and indicate stable readings after about 80 days after placing the fill. In general, a faster dissipation of excess pore water pressure was expected based on the results from the previous test setup reported by Sørensen (2015). This can be explained in part by the existence of vertical drains about 10 m from the test site. However, erroneous readings of the piezometers cannot be ruled out since all three piezometers installed in gyttja showed similar trend. In order to simulate the accelerated consolidation time and match the ground settlement with time, the coefficient of permeability was increased 5 times, i.e. $k_x=k_y=0.41 \cdot 10^{-3}$ m/day. Another way of simulating the observed settlements, without changing the coefficient of permeability, was to create a 7 cm thick vertical sand drain along the boundary of the model. The sand drain had a coefficient of permeability equal to 0.01 m/day. Both simulations are shown in Figure 1.

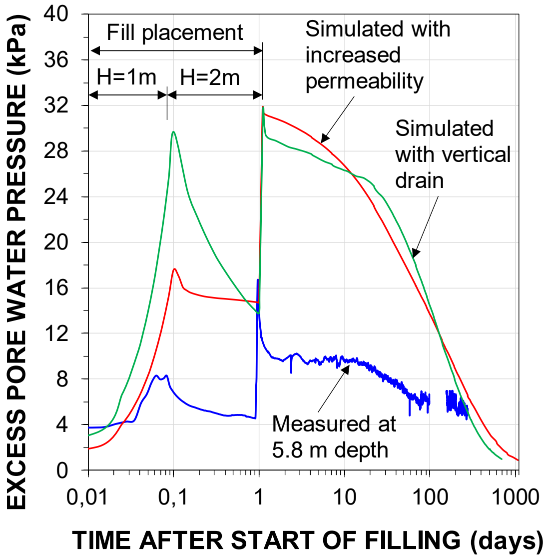

Figure 1. Excess pore pressure dissipation with time due to fill placement (measured data based on Kania et al. (2023)).

3.2 Ground settlement

The apparent preconsolidation of the gyttja layer in the upper part was assumed to decrease with depth. Therefore, the OCR and the preconsolidation margin used in the PLAXIS models were reduced to 1.5 and 10 kPa, respectively. Figure 2 shows the measured and simulated soil settlement profile 277 days after placing the fill. The model with vertical sand drain served as a basis to compare two different ways of generating initial preconsolidation stress in PLAXIS. As shown in the figure, both methods provide good agreement with the measured data. By using the OCR method, the simulated soil settlement profile differs from the observed one within the gyttja layer, however, it has a perfect match at the top of the gyttja layer. In contrast, by using the preconsolidation margin method the soil settlement is slightly different only in the upper part of the curve. This difference can be explained in part by the fact that an OCR is a ratio function, while a preconsolidation margin is a stress difference.

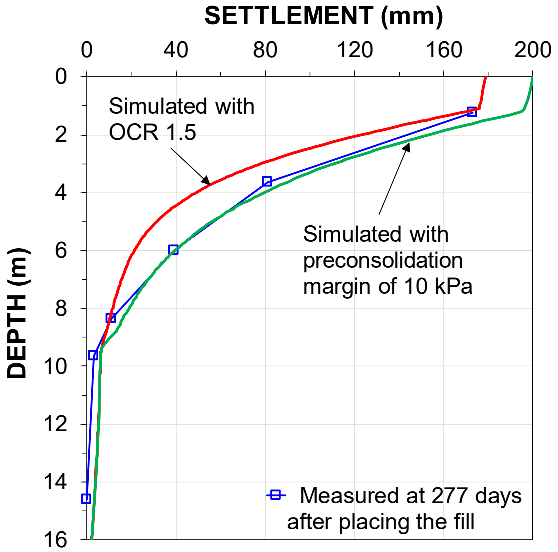

Figure 2. Measured and simulated settlement 277 days after placing the fill (measured data based on Kania et al. (2023).

3.3 Drag force distribution and negative skin friction along the uncoated steel pile

To calibrate the soil-pile interaction model, the strength reduction factor, R_{inter}, was adjusted using the PLAXIS model with the vertical sand drain and the preconsolidation margin of 10 kPa. The simulated and measured drag force distributions obtained 277 days after placing the fill are presented in Figure 3. As shown in the figure, the R_{inter} of 1.0 provided 2 times higher drag force than the measured value. However, a good agreement was obtained using the R_{inter} of 0.45. This finding is consistent with Potyondy (1961), who

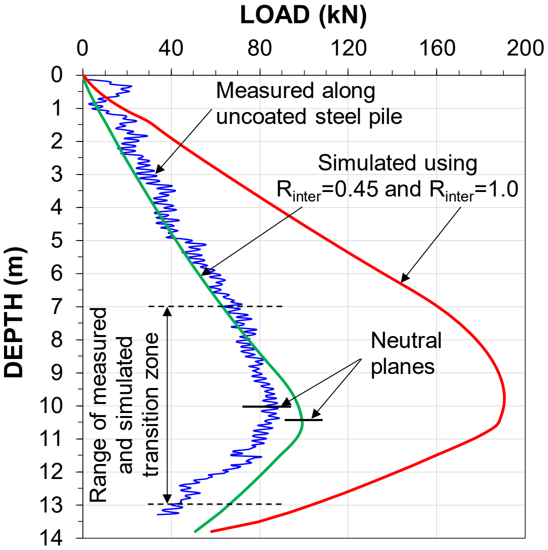

Figure 3. Measured and simulated drag force distributions (measured data based on Kania et al. (2023)).

proposed a strength reduction factor of 0.5 (average value) between rough steel and cohesive granular material. A neutral plane, a point of force equilibrium, and a transition zone, at which negative skin friction gradually changes into positive shaft resistance, were located at similar depths in both, measured and simulated drag force distributions.

The development of negative skin friction with time after fill placement is presented in Figure 4. Both measured and simulated values are comparable. However, measured negative skin friction started to increase about 8 days after placing the fill, while the simulated curve shows a logarithmic relationship shortly after placing the fill. A possible explanation for this is that the shearing rate effect was not correctly simulated in the PLAXIS model.

4 CONCLUSIONS

The purpose of the current study was to determine a suitable constitutive and interface model of soil-pile interaction in gyttja using PLAXIS 2D software. This study has shown that it was possible to simulate the ground settlement and derive a more realistic dissipation of excess pore water pressure. The second major finding was that good agreement between measured and simulated drag force distribution was obtained using the strength reduction factor of 0.45. The current study has only examined the behaviour of a single uncoated steel driven pile in an axisymmetric model. Further work needs to be done to establish a response of different piles, e.g. concrete driven piles or bitumen coated piles. It would be also interesting to assess the group effect using PLAXIS 3D software.

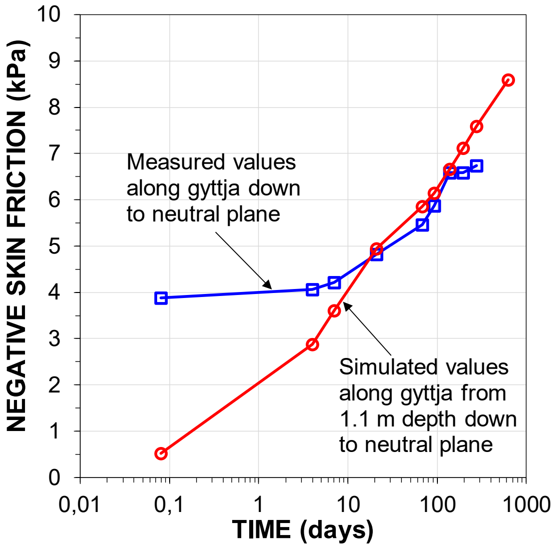

Figure 4. Measured and simulated development of negative skin friction with time after fill placement (measured data based on Kania et al. (2023)).

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REFERENCES

Belinchon, P., Sørensen, K. K. and Christensen, R. (2016). A case study of the interaction between a pile and soft soil focusing on negative skin friction using finite element analysis. In: *Proceedings of the 17th Nordic Geotechnical Meeting*, Reykjavik, Island, pp. 513-522.

Bentley Systems (2022). PLAXIS 2D, CONNECT Edition (V22.02).

Fellenius, B. H. (1984). Negative skin friction and settlement of piles. In: *Proceedings of the Second International Seminar, Pile Foundations, Nanyang Technological Institute*, Singapore, 12 p.

Kania, J. G., Sørensen, K. K. and Fellenius, B. H. (2023). The development of unit shaft resistance along driven piles in subsiding soil. *Canadian Geotechnical Journal*. https://doi.org/10.1139/cgj-2022-0694.

Potyondy, J. G. (1961). Skin friction between various soils and construction materials. *Géotechnique,* 11(4): 339-353, https://doi.org/10.1680/geot.1961.11.4.339.

Sørensen, K. K. (2015). Fuldskala pæleforsøg på Randers havn/Full-scale pile testing at Randers Harbour/, presentation at the Danish Geotechnical Society meeting (in English)