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# INVESTIGATION OF GEOTRIBOLOGICAL BEHAVIOUR IN THE SOIL-PILE INTERFACE CONSIDERING CLAY BASED ON NUMERICAL AND LABORATORY INVESTIGATIONS

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**Abstract.** The contact behavior between soil and structures is an important aspect in many geotechnical applications. One example is the contact between pile and soil during pile installation which especially for open-ended pipes can lead to soil plug formation. Within the present research, contact behavior between clayey soil and pile is investigated by means of numerical and laboratory experiments focusing on the contact behavior within tubular piles. First, the contact between kaolin clay and steel is experimentally investigated with respect to overburden pressure in the so-called Geo-Tribometer developed at HSU. The results of the experimental investigations show some unexpected results leading to the assumption that the contact failure surface inside the soil specimen changes with differing overburden pressure. Additional numerical simulations are carried out for better understanding of contact stress development. Second, further laboratory investigations using soil-filled tubular piles show that adhesion-like effects significantly influence the contact behavior between soil plug and internal surface of the tube. For estimation of the adhesion values, numerical simulations by means of finite element analyses are carried out showing that as expected with increasing soil's overburden pressure adhesion effect increases. The results are finally discussed with respect to transferability from small scale in numerical and laboratory investigations toward prototype scale.

## 1 INTRODUCTION

The calculation of vertical stresses below a soil plug inside a tubular pile is strongly dependent on the contact behaviour between internal soil plug and inner pile surface. Especially for clay soil the phenomenon of soil plugging is up to now not fully understood.

An empirical approach for non-cohesive soil material can be found in [1] based on silo-theory after [2]. For undrained conditions the vertical stress below the soil plug can be estimated after eq. (1):

$$\sigma_{\mathrm{v}} = p + (\gamma_{\mathrm{w}} + \gamma')z + 4\beta \frac{z}{D} p + 2\beta \frac{z}{D} \gamma' z \tag{1}$$

with

$$\beta = \frac{\tau_i}{\sigma'_{\mathrm{v}}} = \tan\varphi \cdot K_0 \tag{2}$$

The vertical stress can be calculated based on the own weight of the soil plug together with frictional contact. Further, a surface load $p$ can be considered. For drained conditions, the effective vertical stress can be calculated as follows.

$$\sigma'_{\mathrm{v}} = p + \left(\mathrm{e}^{\frac{4\beta z}{D}} - 1\right)\left(p + \frac{D\gamma'}{4\beta}\right) \tag{3}$$

Not considered in this approach are possible adhesive or adhesion-like effects which are often discussed in the framework of the bonding potential in tunneling works. Especially for clay soils adhesion effects can be significant based on the minerals within the clay. Therefore, it is possible to add an adhesive term in eq. (1) and (3) by incorporating shear stress from adhesive effects $c_{\mathrm{a}}$, which are considered constant over height and circumferential area of contact. Simultaneously, the surface load $p$ is neglected.

$$\sigma_{\mathrm{v}} = (\gamma_{\mathrm{w}} + \gamma')z + 2\beta \frac{z}{D} \gamma' z + 4c_{\mathrm{a}} \frac{z}{D} \tag{4}$$

$$\sigma'_{\mathrm{v}} = \left(\mathrm{e}^{\frac{4\beta z}{D}} - 1\right)\frac{D\gamma'}{4\beta} + 4c_{\mathrm{a}} \frac{z}{D} \tag{5}$$

Within this paper, some numerical and experimental investigations are presented to better understand the contact behavior between clay and steel tubular piles.

## 2 GEO-TRIBOMETER

### 2.1 Experimental device and exemplary results

The Geo-Tribometer developed by the Professorship of Geotechnics at Helmut-Schmidt-University (HSU) is a device based on a standard oedometer test connected with a ring shear torsion apparatus after [3] to investigate the shear friction behavior of a soil specimen inside a tubular ring. The device is fully described in [4] such that in this paper only a brief description of the apparatus is given. For a visualization of the test setup refer to Figure 1.

The main element for friction measurement is the highly sensitive multi-component sensor (1) which detects both forces and moments in three directions of a cartesian coordinate system. In the present research, mainly the momentum $M_z$ is of interest as this can directly be correlated to contact friction stresses. The soil sample itself is placed inside a pressure cell (2) with diameter 75 mm. The pressure plate is placed on a rotation frame (3) which is connected to a precision planetary gearbox (4), reducing incremental steps for rotation velocity. An electric stepper motor is used for powering the planetary gear (5). Motor and all additional setup are mounted on a steel frame linked to a base plate (6).

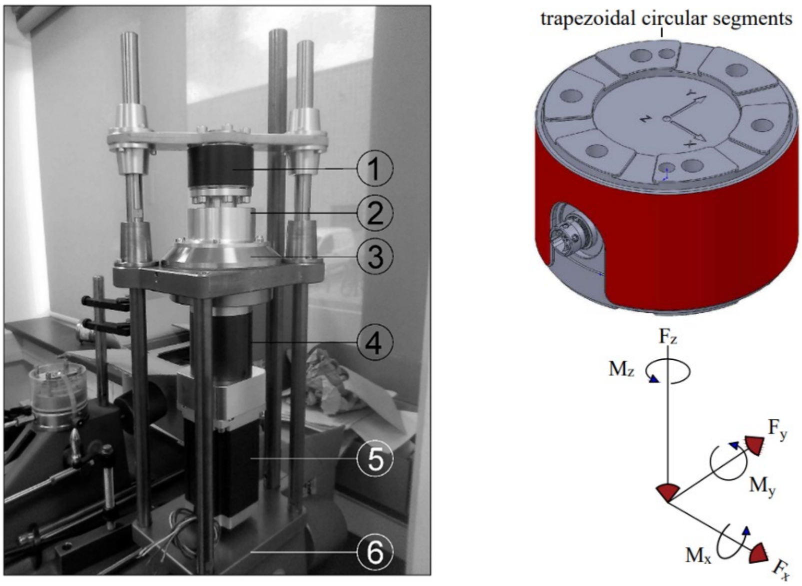


**Figure 1**: Experimental setup of the Geo-Tribometer (left) and multi-component sensor (right)

In the current research, Geo-Tribometer tests considering kaolin clay with a water content of approx. 42 % are conducted to investigate the contact behavior between kaolin and the internal steel surface of the sample container. The height of the soil samples is set to approx. 10 mm per test. Tests were carried out considering vertical stresses on the soil sample of 50, 75, 100, 150 and 200 kN/m². In Figure 2 exemplary results for low vertical stress levels of 50 kN/m² and 75 kN/m² are depicted.

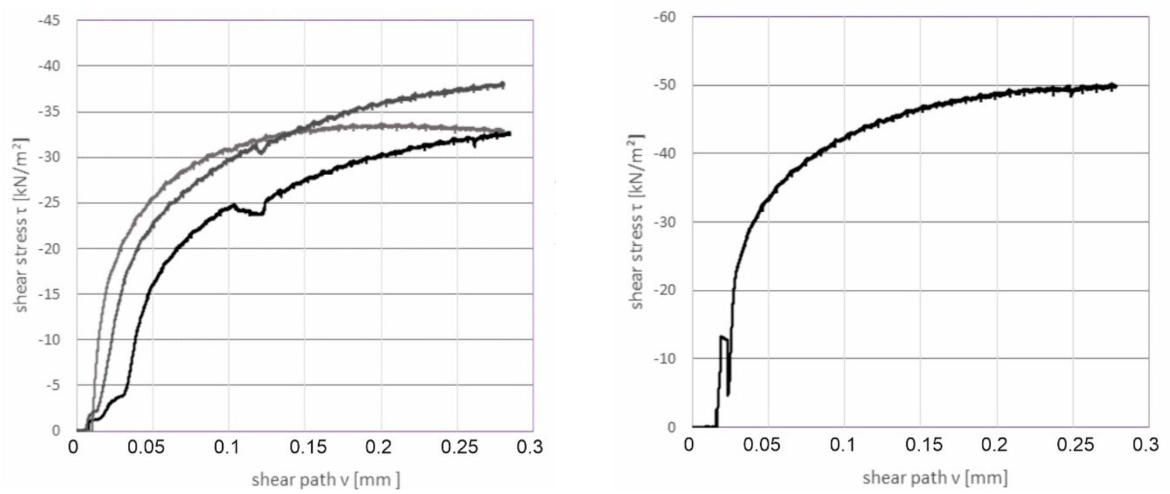


**Figure 2**: Exemplary results of Geo-Tribometer tests with kaolin clay at 50 kN/m² (left) and 75 kN/m² (right) vertical stress

Regarding the experimental results, it becomes evident that the shear stress increases heading towards a maximum mobilized shear stress at a rather low shear path of about 0.1-0.2 mm. For tests with vertical stress level of 50 kN/m² the maximum shear stresses are between 32 kN/m² and 38 kN/m². As expected with increasing vertical stresses the mobilized maximum shear stress at 75 kN/m² stress level increases up to approx. 50 kN/m². Regarding higher vertical stress level in several tests the shearing behavior changed significantly, see Figure 3.

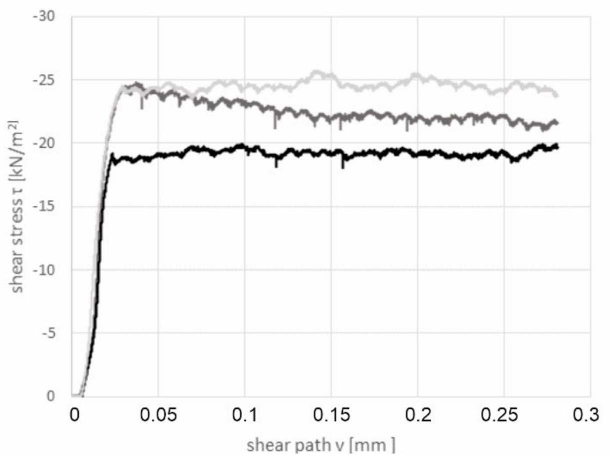


**Figure 3**: Exemplary results of Geo-Tribometer tests with kaolin clay at 200 kN/m² vertical stress

Considering the results in Figure 3 it seems that at higher stress levels the shear failure mechanism changes. Probably, during shearing at lower stress levels failure occurs within the soil sample itself whereas at higher stress levels failure is located directly at the interface between clay and steel. This has to be investigated in further research to identify the reason for the differing stress-strain behavior. In the current research, simplified numerical simulations of the Geo-Tribometer tests are carried out to investigate the Geo-Tribometer experiments in more detail.

## 2.2 Comparative numerical study

A comparative numerical study is done using commercial code *Abaqus/Standard*. A numerical model considering the main dimensions of the Geo-Tribometer is built, see Figure 4.

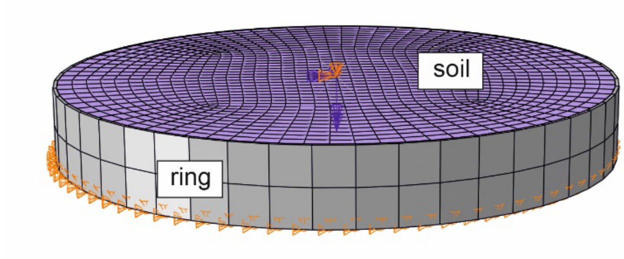


**Figure 4**: Numerical model for simulating the Geo-Tribometer tests with kaolin clay

The soil is modelled as a deformable volume modelled with a simple Drucker-Prager constitutive model. The parameters are based on experimental oedometer and triaxial tests on kaolin with water content comparable to the experiments and are summarized in table 1. Drained conditions are considered due to a very low shearing velocity.

The steel ring on the other hand is modelled as a rigid body. Contact between steel ring and soil is set to a friction coefficient $\mu = 0.2$ as a first assumption considering a Coulomb-friction behavior. During simulation first gravity loading is applied and afterwards pressure loading is placed on top of the soil sample. The tribometry test itself is simulated by rotating the ring with a constant velocity of 0.0625 °/min similar to the experiments. A total simulation time of 720 s is modelled such that a final rotation of 0.75° is applied on the ring.

**Table 1**: Drucker-Prager constitutive parameters for kaolin

| Mass density $\rho_s$ [t/m3] | Young's Modulus $E$ [kN/m²] | Poisson's Ratio $\nu$ [-] | Angle of Friction $\beta(\varphi)$ [°] | Dilation Angle $\psi$ [°] | Yield Stress $d(c)$ [kN/m²] | Abs. Plastic Strain |
|---|---|---|---|---|---|---|
| 1.6 | 3.000 | 0.2 | 29.35 | 0 | 15 | 0 |

To better understand the deformation behavior of the soil sample during the Geo-Tribometer experiments, the logarithmic strain distribution at maximum torsion of 0.75° are shown in Figure 5.

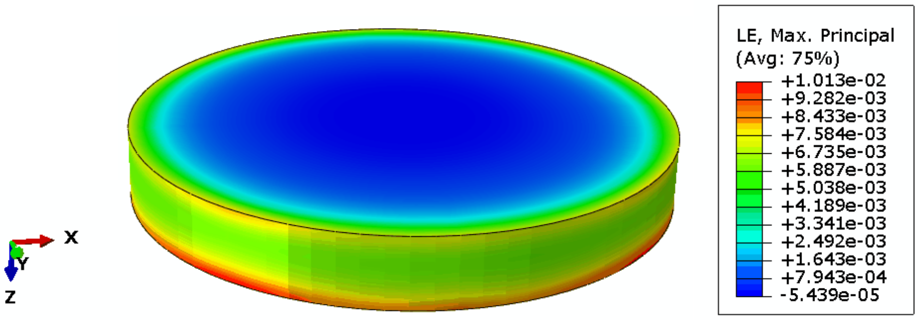


**Figure 5**: Resulting logarithmic strain distribution for a numerical simulated Geo-Tribometer test with 50 kN/m² vertical stress at 0.75° ring rotation

First, it can be seen that as expected the maximum strains are mobilized close to the ring whereas the center parts of the sample remain undistorted. Thus, the expected shearing behavior between soil and ring is well represented in the numerical simulations. Second, regarding the strain distribution at the contact surface the strains are distributed non-uniform with a minimum along the center line.

Main aim of the numerical simulations was to investigate which of the experimental stress-strain relationship is reproduced in the numerical simulation. Therefore, in Figure 6 shear stress development over shear path is depicted for Geo-Tribometer tests with vertical stresses from 50 kN/m² up to 200 kN/m². Regarding the results in Figure 6 it is evident that the general evolution of shear stress over shear path is similar independent from vertical stress state. Therefore, it seems that the numerical simulations with boundary conditions, as applied in the current study, are only able to reproduce the shearing behavior as identified in Geo-Tribometer experiments at low vertical stress state. Therefore, further research considering e.g., more complex interface friction models like hypoplastic interface models will be carried out in future.

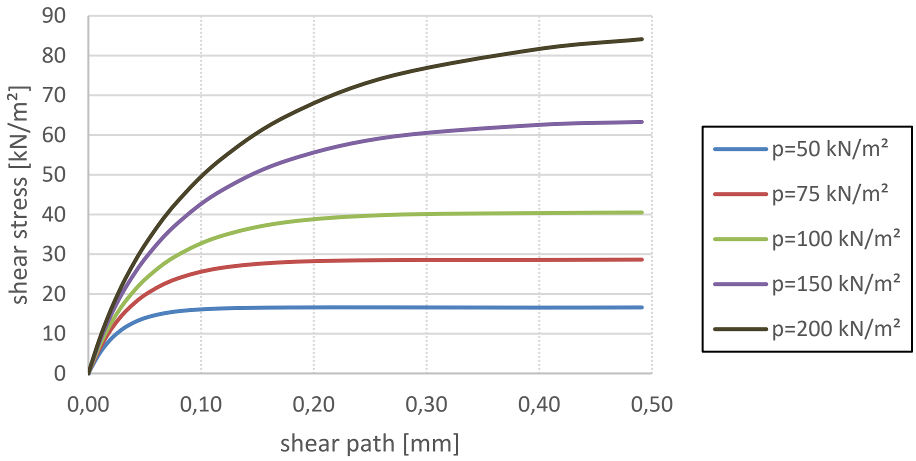


**Figure 6**: Numerically simulated shear stress mobilization over shear path for different vertical stresses considering a friction coefficient of $\mu = 0.2$

Comparing the results of the numerical results shown in Figure 6 with the experimental results from Figure 2 leads to the conclusion that the assumed friction coefficient of $\mu = 0.2$ is too low to quantitatively reproduce the experimental results. Therefore, a parametric variation of friction coefficient is done with values of $\mu = 0.3$ as well as $\mu = 0.33$, see Figure 7.

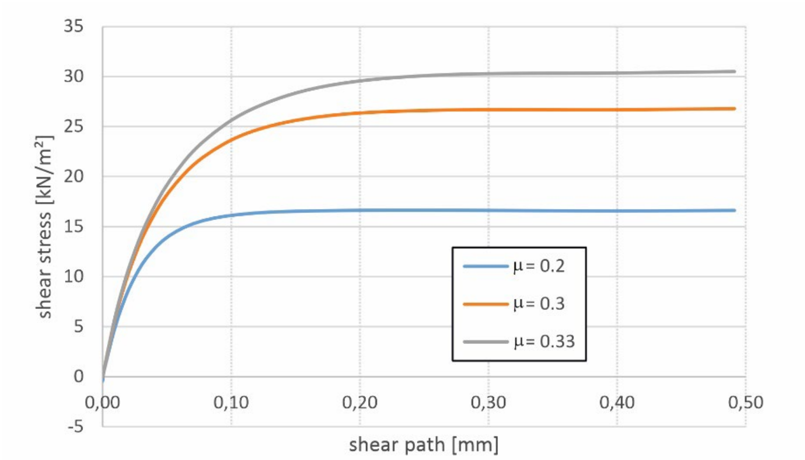


**Figure 7**: Shear stress mobilization for different friction coefficients $\mu$ at 50 kN/m² vertical stress

The simulation with a friction value of $\mu = 0.33$ leads to a maximum mobilized shear stress of approx. 30 kN/m² which is close to the experimental results. Furthermore, considering this relatively high friction coefficient of $\mu = 0.33$ the shear path necessary to mobilize maximum shear stress is about 0.25 mm which is also close to the experimental results.

Concluding this section, the following results can be summarized:

- The numerical model with the considered boundary conditions of this research is able to reproduce a steady increase of shear stress over shear path.

- Considering a rather high friction coefficient of $\mu = 0.33$ the experimental results can be recalculated with sufficient accuracy considering both maximum mobilized shear stress as well as stress-strain development. It is assumed that adhesive or adhesion-like effects could be the reason for the rather high skin friction mobilized at the soil-steel-interface which typically is rather smooth.
- The bi-linear failure behavior as encountered during experiments with higher vertical stress cannot be reproduced in the current numerical simulations.
- It is assumed that higher valued constitutive models like (visco-)hypoplasticity as well as interface formulations have to be considered in future simulations to better represent the above-mentioned bi-linear stress-strain behavior.

## 3 PIPE SHEAR TEST

### 3.1 Experimental setup and exemplary results

Another approach to investigate contact behavior between soil plug and tube was chosen by means of pipe shear tests. In this case, kaolin clay first was consolidated inside steel tubes with different diameter (36 mm, 50 mm and 64 mm). Consolidation was done under different vertical stresses of 63, 104 as well as 201 kN/m². At the end of consolidation, water content of approx. 52 % was evident with a standard deviation of about 1.6 %. To investigate friction between clay and steel tube, the kaolin sample within the steel tube was placed under a one-axial press and the soil sample was pressed out of the tube considering different velocities (0.1 mm/s, 0.2 mm/s as well as 50 mm/s). This leads to a test series of 29 tests in total. During the tests reaction force as well as soil displacement were measured. The test setup including sample preparation as discussed before are depicted in Figure 8.

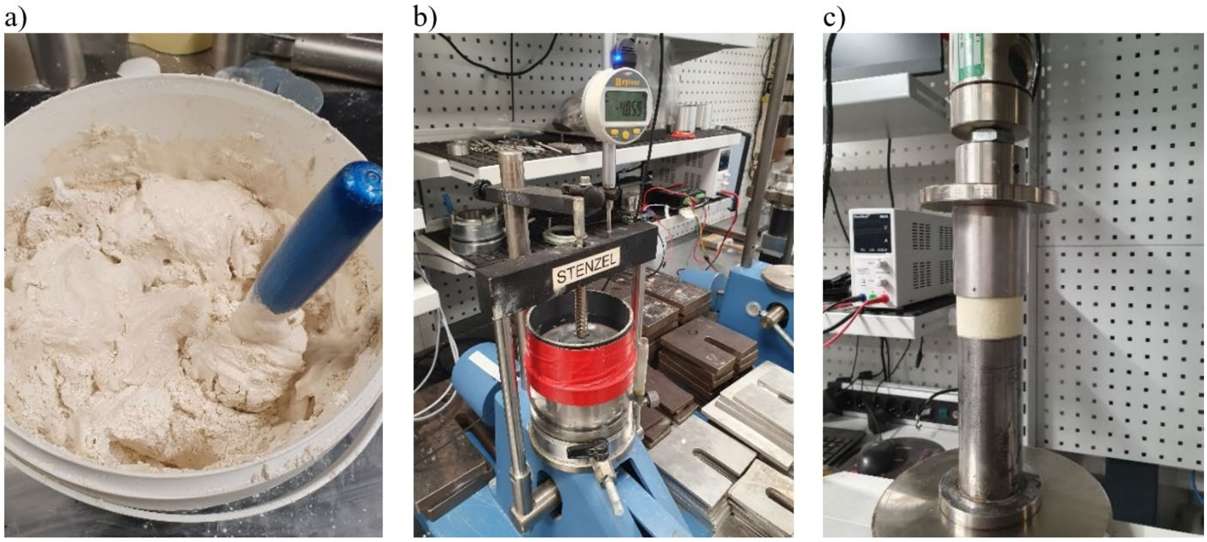


**Figure 8**: a) kaolin preparation, b) consolidation of the sample, c) pipe shear test

The results for pipe shear tests with kaolin consolidated under 100 kN/m² vertical consolidation stress are depicted in Figure 4. Furthermore, in this figure the results with calculations after eq. (1) for undrained conditions and eq. (3) for drained conditions are shown. These calculation results do not match the experimental results at all. It seems that considering drained conditions gives more realistic predictions. Even though considering very high parameter values of $\beta = 0.5$ and $\gamma' = 20$ kN/m³ high discrepancies between analytical and experimental results are apparent. For this reason, undrained and drained calculations after eq. (5) and (6) considering adhesion-like effects with an additional friction component of $c_a = 4.5$ kN/m² and $c_a = 5$ kN/m² were performed respectively and are displayed in Figure 9.

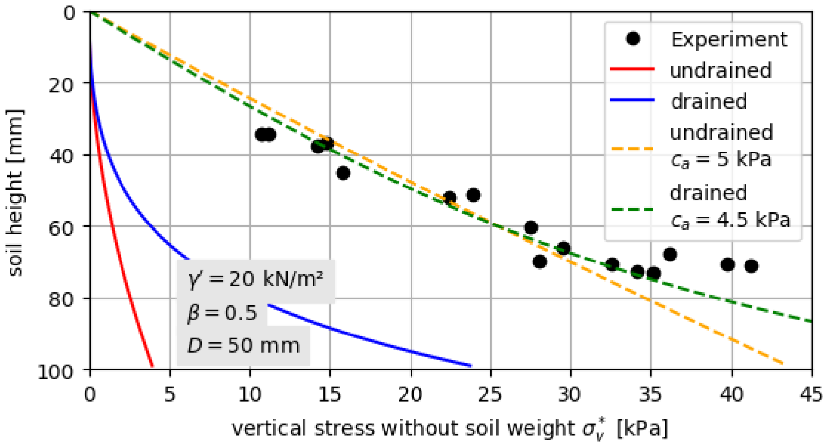


**Figure 9**: Comparison of vertical stresses received from experiments and calculated after eq (1) and (3)

It becomes evident that adhesion-like effects apparently have significant influence on the vertical resistance of the soil plug and must be considered in the analytical calculations to better represent the experimental results. Evidently drained conditions seem to be more appropriate to best represent the experimental results. All necessary parameters used in the analytical calculations are also depicted in Figure 9.

The additional shear stress $c_a$ may be due to adhesive effects. Another explanation is that after consolidation horizontal stresses remain inside the soil and act as additional normal stress onto the pipe wall. Adding a term for additional normal stresses from consolidation $\sigma_{c,h}$ to eq. (1) and (3) results in eq. (6) and (7). Beside already having $\beta$ as an unknown value, additionally adding a new unknown in $\sigma_{c,h}$ gives more room for interpretation and speculation.

$$\sigma_v = (\gamma_w + \gamma')z + 4\frac{\beta}{K_0}\frac{z}{D}\,\sigma_{c,h} + 2\beta\frac{z}{D}\gamma' z \tag{6}$$

$$\sigma_v' = \left(e^{\frac{4\beta z}{D}} - 1\right)\left(\frac{D\gamma'}{4\beta} + \frac{\sigma_{c,h}}{K_0}\right) \tag{7}$$

Which of these two effects has the higher influence on the measured shear stress has not been identified. For simplification, for the rest of this paper this additional shear stress is therefore being referred to as adhesion-like effects. Especially for the numerical study this is easier to work with. For the future investigations regarding parallel measurement of horizontal stresses are planned which is expected to solve this issue.

Regarding the back-calculated adhesion values with respect to the pressure applied during consolidation the results are depicted in Figure 10. It is evident that with increasing consolidation stress the adhesion-like effect increases. Furthermore, a small influence of plug height on the additional friction stress can be drawn out of the results.

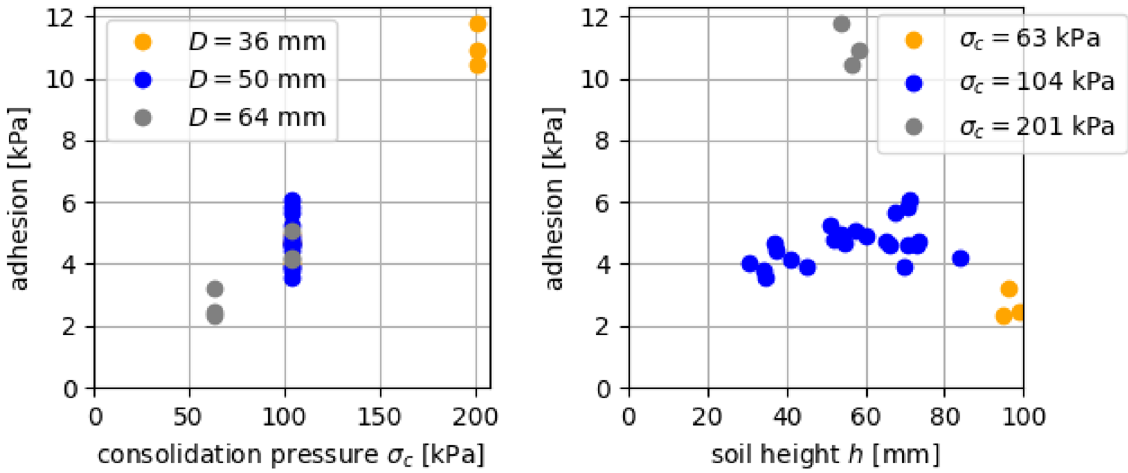


**Figure 10**: Back-calculated adhesion value with respect to consolidation pressure (left) and soil height (right)

Summarizing, the influence of adhesion-like effects seems to be a significant factor affecting plugging resistance such that beside the shown experimental results comparative simplified numerical simulations are carried out to verify the above-described results.

### 3.2 Comparative numerical study

The numerical model used for comparative simulations of the pipe shear tests is an axisymmetric model simulated using the commercial code Plaxis2D. The boundary conditions can directly be taken out of Figure 11. As seen, loading is modelled displacement controlled at the lower part of the model. The soil is modelled considering the Hardening-Soil model after [5] for a sufficiently realistic simulation of the soil's constitutive behavior. The parameters used in the simulations are summarized in table 2. Values for prehistoric overburden pressure POP is applied similar to those from experimental investigations. Only the cohesion considered in the numerical study has to be calculated with respect to the values including adhesion like stresses, see eq. (8).

$$c' = \frac{\sigma_{\mathrm{v,a}} D}{4z(1+\beta)} \tag{8}$$

Here $\sigma_{\mathrm{v,a}}$ is the vertical stress portion which is the deviation between the measured and the calculated vertical stress and which is seen to result from adhesive-like effects. In the numerical model increasing the cohesion parameter led to an increase in normal stresses on the pile wall. Therefore, the employed cohesion was reduced by the factor $\frac{1}{1+\beta}$ to not overestimate skin friction resistance, which is incorporated in eq. (8).

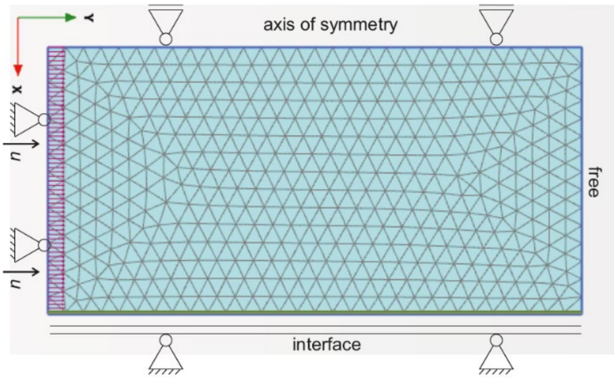


**Figure 11**: Numerical model for simulation of pipe shear test including boundary conditions and mesh

**Table 2**: Hardening-Soil parameters used in the numerical simulations of pipe shear test

| $\gamma/\gamma'$ [kN/m³] | $E_{\text{oed}}^{ref}$ [kN/m²] | $E_{50}^{ref}$ [kN/m²] | $E_{\text{ur}}^{ref}$ [kN/m²] | $p_{\text{ref}}$ [kN/m²] | $m$ [-] | $\varphi'$ [°] | $c'$ [kN/m²] |
|---|---|---|---|---|---|---|---|
| 20/10 | 200 | 200 | 1.200 | 200 | 0.9 | 11,3 | eq. (8) |

Exemplary results of skin friction development over pipe displacement (displacement rate 0.1 mm/min) for a test with a 36 mm diameter tube and consolidation stress of 104 kN/m² are shown in Figure 12 (left). Comparing, numerical and experimental results are in very good agreement up to maximum skin friction considering both maximum mobilized skin friction as well as the necessary displacement to mobilize maximum skin friction. Furthermore, Figure 12 (right) shows a comparison of maximum skin friction over all simulations and experiments. Using the numerical simulation, the experimental results can be reproduced with high accuracy.

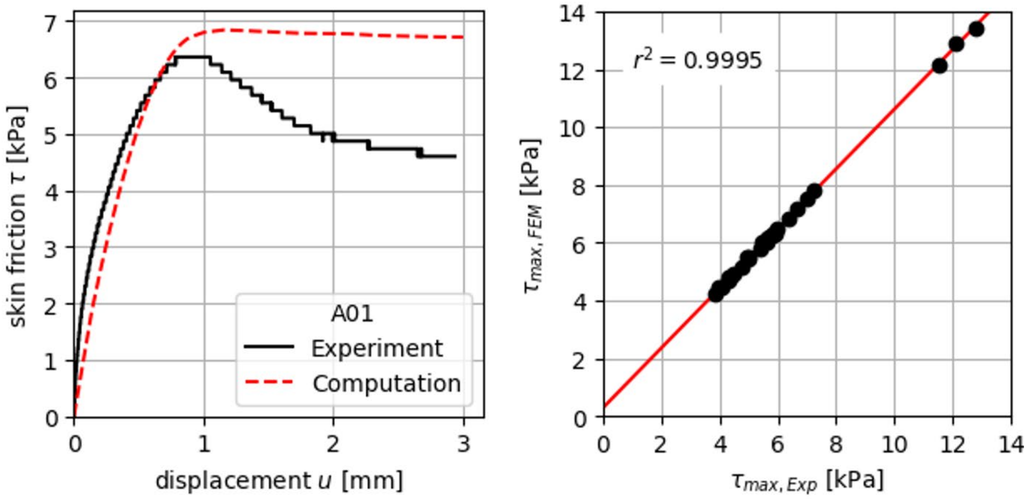


**Figure 12**: Numerical and experimental results of pipe shear test A01 with 36 mm diameter tube and 104 kN/m² consolidation stress (left) and comparison of maximum skin friction in computation and experiment for all investigations

For further investigation a parametric study with varying boundary conditions was carried out. The varied parameters are summarized in table 3. The results of the parametric study are summarized in Figure 13.

**Table 3**: Varied parameters in the parametric study

| $a_{ij}$ | $j = 1$ | $j = 2$ | $j = 3$ |
|---|---|---|---|
| $i = 1$ | $D = 10$ mm | $D = 40$ mm | $D = 100$ mm |
| $i = 2$ | $h = 10$ mm | $h = 10$ mm | $h = 100$ mm |
| $i = 3$ | $c' = 3.5$ kN/m$^2$<br>$\sigma_c = 70$ kN/m$^2$ | $c' = 5$ kN/m$^2$<br>$\sigma_c = 100$ kN/m$^2$ | $c' = 10$ kN/m$^2$<br>$\sigma_c = 200$ kN/m$^2$ |

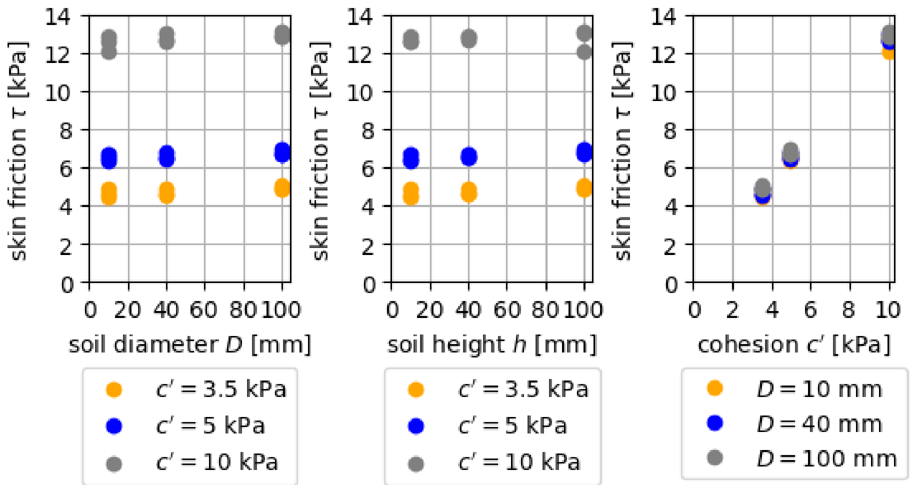


**Figure 13**: Max. skin friction in numerical simulations with respect to soil diameter, soil height and cohesion

Summarizing, it can be concluded that considering the numerical results the applied cohesion mainly influences mobilized skin friction. Beside this, sample diameter and sample height within the boundaries in the current numerical simulations do not have an influence.

## 4 CONCLUSION AND OUTLOOK

The present research gives a brief insight into the complex soil-structure interaction of clay material inside tubular piles. Two different experimental approaches to evaluate skin friction in the interface between these two materials are presented and briefly discussed. By means of numerical simulations both experiments could be recalculated and several effects encountered during the experiments were well reproduced in the numerical simulations.

Out of these experiments and simulations it becomes evident that contact friction behavior between clay and steel inside tubular piles is highly complex. It seems that adhesion-like effects lead to rather high skin friction values which could directly be measured and simulated in the pipe shear tests and which was further supported by the high friction coefficient necessary to recalculate the Geo-Tribometer tests in the comparative numerical simulations. The origin of these adhesive-like effects observed in the pipe shear tests may be related to horizontal stresses as leftovers from consolidation rather than true adhesion.

Nevertheless, the current investigations only form the basis for further detailed research to investigate the complex interface behavior between clay and steel inside tubular piles. Therefore, the following investigations will be carried out in future:

- Numerical simulations of the Geo-Tribometer tests considering higher valued constitutive models for soil and interface,
- Development of a pipe shear test apparatus to carry out pipe shear tests with varying confining pressure during the tests,
- Centrifuge modelling of tubular pile installation with measurement of internal stresses at soil plug level to transfer the laboratory experiments to real scale.

## ACHNOWLEDGEMENTS

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