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Shear box tests to explore the soil-structure interface of model piles created using 3D printing techniques

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ABSTRACT: Current research on piled foundations has focused on methods of reducing the size of piles whilst maintaining or increasing their capacity to conserve materials and reduce the amount of underground space needed. Generally, the ultimate capacity of a pile in clay is achieved through shaft friction between the pile and the soil. The proportion of undrained shear strength, S_u, of a soil that is mobilised by a pile shaft depends on the adhesion factor, α. For bored piles in clay, α varies between 0.35-0.50, suggesting that a large proportion of S_u along the pile shaft length is not mobilised. Recent advancement in research have investigated this complex soil-structure interaction by using 3D printing techniques to fabricate model piles, with a repeatable and known surface roughness, for subsequent use in centrifuge model testing. However, this technique has only provided the overall response of the foundation. At City St George's, University of London, a series of 100 x100 mm direct shear box tests were undertaken on samples of 3D printed plastic and 3D printed metal to investigate the influence of this surface roughness. 3D printed samples were placed within one half of the shear box with the designed surface roughness exposed to the shear plane. The other half of the sample consisted of pre-consolidated Speswhite kaolin clay. Tests were conducted at a vertical stress of 124 kPa to represent *in-situ* stress conditions at the mid-point depth along the length of a 9 m long prototype pile.

KEYWORDS: 3D Printing, piles & Piling, shear box.

1 INTRODUCTION

The end bearing and shaft resistance capacity of piles are mobilised through the resultant settlement that occurs following an applied head load. Bored piles in clay mobilise a relatively small amount of their load capacity from end bearing. This is because, in a clay, the required applied loads typically exceed the safe working capacity of the pile (Burland, 2023).

The shaft capacity of a bored pile in clay is achieved through the mobilisation of the friction between the pile surface and the soil. This is governed by the proportion of the undrained shear strength, S_u, that is predicted to be mobilised along the pile shaft. The proportion is defined as the dimensionless adhesion factor, α. When considering conventional bored piles in clay, the value of α has been shown to vary between 0.35-0.50, suggesting that a large proportion of the undrained shear strength of the clay along the length of the pile shaft is not being fully mobilised.

Studies by Panchal *et al.* (2019), Sabaliauskaite *et al.* (2024) and Vardanega *et al.* (2012) suggest that a reduction in the pile stiffness or a greater pile compression could be accounted for to arrive at a higher overall pile capacity. Panchal *et al.* (2019) and Sabaliauskaite *et al.* (2024) used geotechnical centrifuge modelling techniques to establish this relationship using scaled model piles in homogeneous soil samples. Studies such as Lalicata *et al.* (2021) have used cast *in-situ* resin to form the model pile instead of solid piles fabricated in metal.

However, Sabaliauskaite *et al.* (2024) used 3D printing techniques to fabricate the model piles with a known and repeatable roughness profile. This would have not been possible with the traditional method of sandblasting, machining or adhering sand particles to the outer surface of a model pile. The model piles in Sabaliauskaite *et al.* (2024) were formed from plastic (OnyxOne) and aluminium with 3D printing technology. Sabaliauskaite *et al.* (2024) also states that the Roughness parameter, R_a, for a concrete block was 12.06 µm and a uniform pattern was included on the outer surface of the model shaft. This pattern had a measured R_a of 11.98 µm.

In this paper a series of 100 x 100 mm standard direct shear box tests were undertaken to quantify the interface behaviour of the roughness profile.

2 TESTING

2.1 *Shear box apparatus*

Figure 1. Standard direct shear box test apparatus at City St George's, University of London.

City St George's, University of London has access to a 100 x100 mm standard direct shear box apparatus (see Figure 1). This apparatus uses a compressed air line to provide the vertical stress to a sample via a 7-bar regulator and a 70 mm diameter piston attached to a 450 mm lever arm. There is a 5 kN load cell which provides feedback to the logging/control computer to ensure the vertical stress is maintained throughout testing. Two 50 mm potentiometers measure the vertical and horizontal displacement. The loading rate can be set via a series of gears.

2.2 *Testing*

Seven tests were undertaken in this paper. Three tests on a 3D printed aluminium interface plate, two tests on the equivalent interface plate printed in OnyxOne plastic and two tests without an interface plate (i.e. sample was completely soil thereby creating a clay-clay interface).

Sabaliauskaite *et al.* (2024) tests 16 mm diameter, 180 mm long, model piles at 50 times Earth's gravity using geotechnical centrifuge modelling techniques. The model piles are situated within Speswhite kaolin clay in a rectangular soil container. These model piles are analogous to 0.8 m diameter, 9 m long, piles at prototype scale.

To ensure consistency with this work, all shear box tests were also performed using Speswhite kaolin clay. In addition, the vertical stress applied to all samples was 124 kPa which represented the horizontal stress at a 4.5 m depth (which is at a depth of half the prototype pile length). Since the tests were undrained, the vertical stress applied in the shear box should not affect the strength measured. The vertical stress was calculated using Equation (1):

$$\sigma_h' = \sigma_v' \times K_o \quad (1)$$

Where σ_h' is the effective horizontal pressure, σ_v' is the effective vertical pressure, and the coefficient of earth pressure at rest, K_o, was estimated using the correlation for overconsolidated soils suggested by Mayne and Kulhawy (1982) along with the proposed relationship in Eurocode 7:

$$K_o = (1 - sin\phi)\sqrt{\mathrm{OCR}} \quad (2)$$

Where the angle of friction for Speswhite kaolin clay, ϕ, was taken as 23°.

The loading rate was kept at a constant 1.20 mm/minute for all the tests to be considered undrained. Ritchie (2024) states the vertical permeability of reconstituted Speswhite kaolin ranging between 1.85 to 2.05 x 10^{-9} m/s. The horizontal permeability was given as ranging between 2.0 and 2.2 x 10^{-9} m/s. Therefore, the loading rate was considered much faster than the rate with which the sample could dissipate excess pore pressures.

2.3 *Interface plates*

Two interface plates were fabricated using the aforementioned 3D printing techniques: one in plastic (OnyxOne) and one in aluminium. These plates were 100 (w) x 100 (l) x 2.7 (t) mm. The Roughness Parameter which was applied to the top surface of these plates was calculated using Equation (3).

$$R_a = \frac{1}{2}\int_0^L |y(x)|dx \; ; R = \frac{R_a}{D_{50}} \quad (3)$$

Where R_a is the centreline average, R is the relative roughness (R_a/D_{50}), L is the measurement length, D_{50} is the average particle size. A roughness offset distance of 0.25 mm was chosen for the interface plates as this value corresponds with the relative roughness value of model piles used in a series of centrifuge tests (Sabaliauskaite *et al.*, 2024). The interface plates are shown in Figure 2.

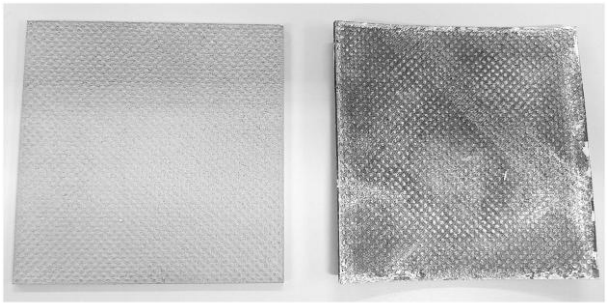

Figure 2. 3D printed interface plates

These interface plates were placed within the bottom half of the shear box on top of spacers to ensure the shear plane was level with the top surface (i.e. only the roughness was proud). The top half of the shear box was filled with a preformed Speswhite kaolin clay layer, as shown in Figure 3.

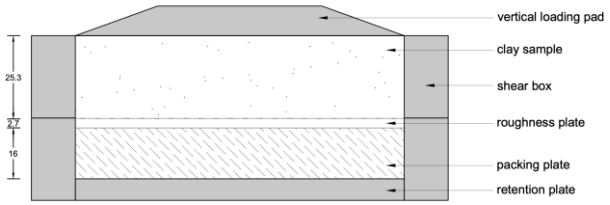

Figure 3. Cross-section of shear box for interface tests

2.4 *Soil*

Speswhite kaolin clay was used for the series of tests. A slurry of 110 % water content was prepared and placed within a rectangular soil container with internal dimensions of 550 (w) x 200 (d) x 375 (h) mm. The slurry was placed between layers of filter paper and porous plastic to allow for two-way drainage. The soil container was placed within a hydraulic controlled consolidation press for approximately one week. The sample was first consolidated to a vertical stress of 500 kPa and swelled to 250 kPa the day before testing to ensure consistency with the stress history from Sabaliauskaite *et al.* (2024).

2.5 *Sample preparation*

On the day of testing, the soil container was removed from the consolidation press and one face of the soil exposed. An angle section served as a shelf for taking horizontal cuts of the clay sample. The shelf was set at an initial depth of 30 mm from the top surface for extracting the clay for the first set of tests. The top 30 mm was divided into four sites. As shown in Figure 4, moving from left to right, two sites were used for testing on the metal interface plates (M1, M2), one for the plastic interface plate (P1) and the other site for a soil-soil (CC1) direct shear test.

The second half of the series of tests lowered the shelf another 30 mm and four more sites were identified. Again, from left to right, two sites were used for testing with the plastic interface plates (P2, P3), one with the metal interface plate (M3) and the other site was for a second soil-soil (CC2) direct shear test.

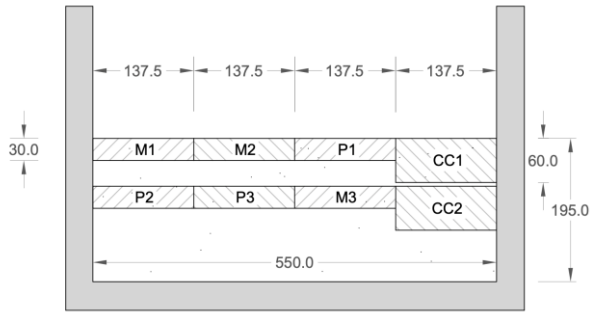

Figure 4. Schematic diagram of sample positions from consolidated clay sample

A flat, 6 mm thick, horizontal cutter was used to remove the sample for testing from the soil container. A template cutter was used to outline the shear box sample size and a wire saw was then used to trim the clay to 100 x 100 mm with minimum edge disturbance, as shown in Figure 5. With the respective interface plate placed into the bottom half of the shear box on spacers, trimmed sections of clay were then placed within the top half. Finally, a wire saw was used to ensure the clay was level with the top level of the shear box.

The shear box was placed within the apparatus frame with the top cap. The loading frame was positioned onto the top cap which has a concave fitting to ensure axial loading. The

potentiometers were placed, and the instrumentation connected. Finally, the logging/control computer programme was initiated which applied the specified vertical load to the sample and shearing could begin.

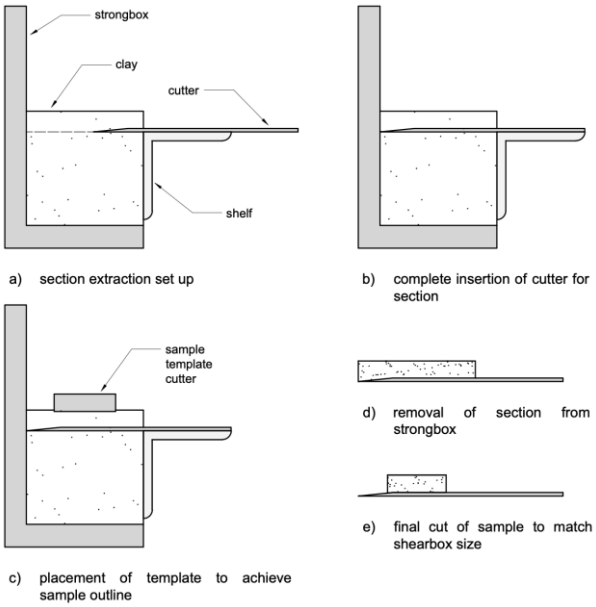

Figure 5. Sample extraction process

2.6 *Testing and post-test*

The sample was sheared at 1.20 mm/minute. Once 10 % horizontal strain, ε_h, (10 mm) had been reached the test was stopped. The entire sample was then removed from the shear box apparatus and water content measurements were taken. These samples had post-test water contents of 57.6 % on average. The individual water content values have been used to calculate estimated undrained shear strength values (following the procedure from Lalicata *et al.*, 2021) on Figure 6.

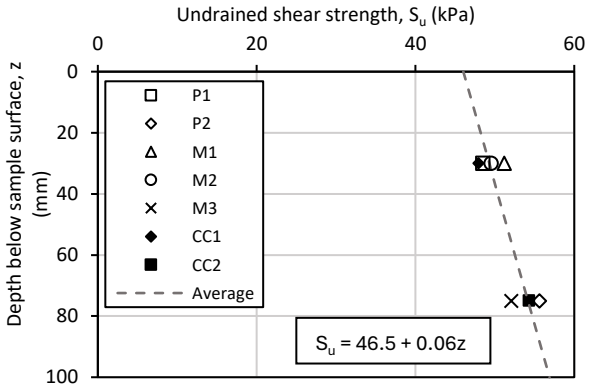

Figure 6. Undrained shear strength (S_u) with depth (z) profile calculated from water contents taken from each sample.

3 RESULTS

3.1 *Direct shear tests*

Figure 7 shows the normalised vertical and shear stress (τ/S_u) v. shear strain (γ_s) results from the seven direct shear tests on the top and bottom layers of Speswhite kaolin clay samples. The shear stress for the interface and clay-clay tests was normalised against the undrained shear strength calculated from the water contents of the samples.

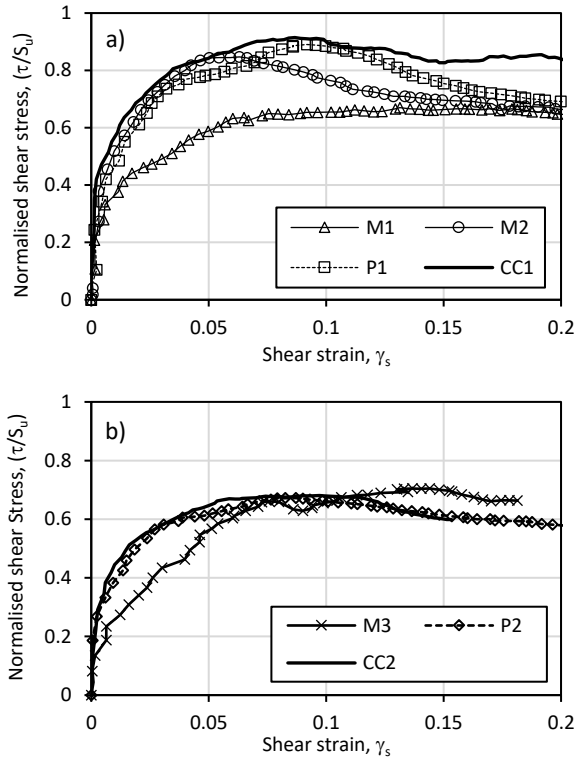

Figure 7. Normalised vertical and shear stress (τ/S_u) v. shear strain (γ_s) from the direct shear tests on the a) top layer, and b) bottom layer of Speswhite kaolin clay samples

For tests M2, P1, and CC1, about 90 % of the undrained shear strength of the soil was mobilised. Whereas tests M1, M3, P2, and CC2 show approximately 70 % mobilisation. In general, the tests conducted on the bottom layer of the sample show a lower proportion of mobilised undrained shear strength.

Figure 8 shows an overall undrained shear strength value of 43.1 kPa which can be compared with the mobilised undrained shear strengths from the tests range between 30 – 38.8 kPa (Figure 7).

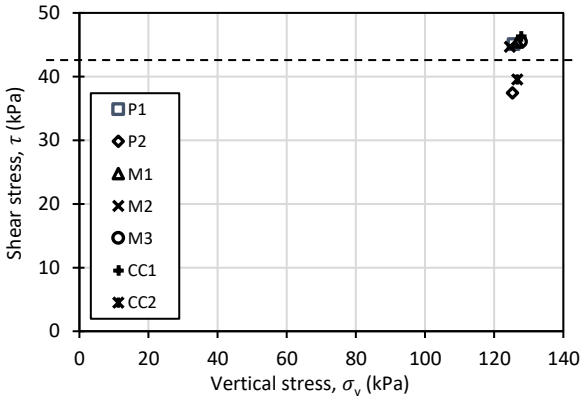

Figure 8. Shear stress (τ) v. vertical stress (σ_v) results from all seven tests.

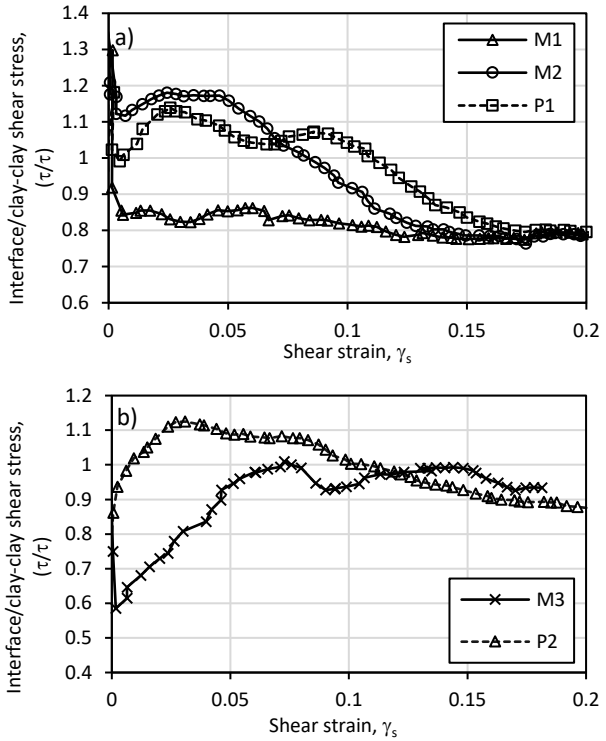

Figure 9. Normalised shear strength (τ/τ) against shear strain (γ_s) from the direct shear tests on the a) top layer, and b) bottom layer of Speswhite kaolin clay samples

Figure 9 shows the normalised shear stress (τ/τ) v. shear strain (γ_s) results from the five direct shear tests on the top and bottom layers of Speswhite kaolin clay samples. The shear stress for the interface tests was normalised against the shear stress for the clay-clay tests in their respective layers.

Interface plate M1 shows a consistently lower interface strength throughout the duration of the test, even at low shear strains, i.e. less than 2 %. However, interface plates M2 and P1 show a higher interface strength at lower shear strains and gradually reach a plateau at a shear strain of 20 %. This would imply an α value of 0.8. This α value is consistent for all three tests conducted on the top layer of the Speswhite kaolin sample.

Interface plates M3 and P2 show a lower interface strength at low shear strains, a slight increase at approximately 2.5 – 5 % of shear strain, and then a decrease to a plateau at 20 % with an α value of 0.9.

In both the bottom and the top layers, the OnyxOne interface plates (P1 and P2), mobilised a larger proportion of the interface strength at very small shear strains, i.e., less than 1 %. Although the effective α value was consistent between the two tests at larger shear strains, there is much greater variability at smaller values. These inconsistencies could either be due to variations in water contents of the samples or a function of how well the contact between the interface plate and the soil was established. While there is limited data to comment on the exact effects of the interface roughness, these tests show that the designed roughness for the pile tests in Sabaliauskaite *et al.* (2024) is mobilising almost all of the shear strength of the clay, therefore it can be assumed that the adhesion factor, α, for the pile roughness varies between 0.8 – 0.9.

4 CONCLUSIONS & FURTHER WORK

In this paper, a series of direct shear box tests were conducted at City St George's, University of London to investigate the soil-structure interface between Speswhite kaolin clay and 3D printed materials. 3D printing techniques have been used by Sabaliauskaite *et al.* (2024) to fabricate model piles, with a repeatable and known surface roughness, for subsequent use in centrifuge model testing. The main aim was to quantify the mobilised shear strength at the interface rather than the overall foundation response.

Five tests were conducted under a vertical stress of 124 kPa using two 3D printed interface plates, one printed using OnyxOne and one using aluminium, with both plates having an identical roughness profile. The normalised stress from these tests was compared to the normalised stress from two tests conducted on samples made entirely of Speswhite kaolin clay.

The interface tests showed that 70 – 90% of the undrained shear strength calculated from water contents was mobilised, or 80 – 90% of the interface strength when compared to the shear stress for the clay-clay direct shear tests. Comparison of the measured undrained shear strengths from the shear box tests and the calculated undrained shear strengths from water contents revealed that the latter method overpredicts the undrained shear strength by 17 %. It was also shown that there is a larger variation of mobilised interface strength at smaller strains due to interface-soil contact or variations in water contents between the soil samples. This variation in interface strength decreases with an increase in strain and ultimately reaches an α value of 0.8 – 0.9. Thus, this series of tests have demonstrated the quantification of the interface properties between Speswhite kaolin clay and the novel model piles fabricated using 3D printing techniques.

A further series of tests should be conducted to investigate how the variations of water contents between the samples affects the small-strain response between the interface plates and the soil. A methodology for consistent interface plate-soil contact should be established to eliminate inconsistencies between tests.

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