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The anisotropic stiffness and shear strength characteristics of a stiff glacial till

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ABSTRACT

Glacial tills are widespread across North America, northern and central Asia, and northern Europe where they are also found under the Baltic, North and Norwegian Seas. Their geological and geotechnical characterisation is important to a wide range of onshore and offshore engineering projects. One aspect of tills on which little has been reported is their mechanical anisotropy. This paper reports coordinated hollow cylinder apparatus (HCA) tests, triaxial shearing and small-strain stress probing experiments, supported by index testing, on high-quality samples of a natural low-to-medium plasticity, high *OCR*, stiff clay-till from the Bolders Bank Formation at Cowden, near Hull in the UK. Material variability and sampling bias is inevitably introduced by the till's erratic gravel particles and fissure systems, and these aspects are addressed carefully. The experiments investigated the till's stiffness and shear strength anisotropy from its limited linear elastic range up to ultimate failure, showing that stiffnesses are higher in the horizontal direction than in the vertical and that higher undrained shear strengths develop under "passive" horizontal loading than "active" vertical loading. Comparisons are made between the till's patterns of anisotropy and those applying to previously studied sediments and reference is made to in-situ stiffness measurements. The important implications of anisotropic behaviour for geotechnical design and the interpretation of field tests are emphasised.

KEYWORDS:

Glacial till; anisotropy; shear strength; stiffness; probing; laboratory tests

INTRODUCTION

Accurate knowledge of how anisotropy affects mechanical behaviour from any initial linear elastic response through to failure plays a key role in enabling representative numerical analyses of ground deformations and soil-structure interaction, see Hashash & Whittle (1996), Addenbrooke *et al.* (1997), Zdravković *et al.* (2001, 2002), Jardine *et al.* (2004) or Avgerinos *et al.* (2016). Examples of improved characterisation of sedimentary clays has been achieved through advanced laboratory investigations of anisotropy include studies by Gasparre *et al.* (2007), Nishimura *et al.* (2007), Brosse *et al.* (2016) and Brosse *et al.* (2017) on four geologically aged highly over-consolidated stiff-to-hard UK clays; Nishimura (2014a, 2014b)'s research on six Japanese sedimentary clays and Ratananikom *et al.* (2013)'s experiments on lightly over-consolidated Bangkok Clay. However, the key experimental evidence required to inform such analyses appear to be lacking for high yield stress ratio (YSR), stiff glacial tills, which are distributed widely over parts of the UK (Davies *et al.*, 2011), northern Europe (Clarke *et al.*, 2008) and North America (Fullerton *et al.*, 2003), as well as parts of northern and central Asia, and under the Baltic, North, Norwegian and other Seas. Understanding regarding how anisotropy affects slopes, earthworks, deep excavations, tunnelling or foundations in glacial tills will be valuable, especially in applications where existing methods perform poorly against field measurements, as is the case for lateral loading analyses of offshore piles; Byrne *et al.* (2017), Jeanjean *et al.* (2017) or Giuliani *et al.* (2017).

Hight & Jardine (1993) emphasise the importance to characterisation studies of high-quality sampling and testing, which can be hampered in glacial tills by their often variable and composite nature, as imparted by glacial erosion, transportation and deposition (Clarke, 2018). Field sampling and laboratory element specimen formation can be difficult and scale

1 effects may impact strength and stiffness measurements, particularly when intense fissuring is
2 encountered. Suitable representative element volumes (REV) need to be identified carefully.
3 The tills' glacial histories may also lead to in-situ effective stress conditions and histories that
4 may not be captured by simple descriptions of 1-D over-consolidation ratios (*OCR*) or axially
5 symmetric K_0 stresses.

6 This paper reports on the mechanical anisotropy of glacial clay till, sampled at Cowden,
7 near Hull in northern England from the Bolders Bank Formation, which is present under large
8 areas of the North Sea (Davies *et al.*, 2011). Ushev (2018), Ushev *et al* (2019) and Ushev &
9 Jardine (2020) report on the till's small-to-large strain behaviour under monotonic and cyclic
10 triaxial loading, oedometer compression and interface shearing, and present detailed
11 comparisons between intact and reconstituted behaviour. The intact till's mechanical
12 anisotropy is explored from the linear elastic range up to failure through coordinated hollow
13 cylinder (HCA), triaxial shearing and probing tests, incorporating multi-directional laboratory
14 and in-situ shear wave velocity measurements. While recognising that glacial deposits may
15 have more complex patterns of anisotropy, the till's behaviour is interpreted within the
16 classical cross-anisotropic framework. Detailed appraisals are also presented of potential
17 effects of sampling and specimen preparation techniques when gravel particle inclusions and
18 fissures are present. The experimental outcomes allow comparisons to be made with glacial
19 formations found in other regions, such as the lightly over-consolidated freshwater Chicago
20 glacial clays studied by Cho & Finno (2010).

21 The study formed part of the PISA Joint Industry Project described by Byrne *et al.*
22 (2017) and Zdravković *et al.* (2020a) which included numerical analyses and lateral loading
23 tests on instrumented monopiles, with diameters up to 2m, at the Cowden site, that were
24 designed to improve design for monopiles driven to support large turbines at many North and

Baltic Sea glacial till offshore windfarm sites, see Schroeder *et al.* (2015, 2020) or Manceau *et al.* (2019). The laboratory research described herein aimed to resolve the significant mismatch noted by Zdravković *et al.* (2020b) between stiffness trends found from triaxial undrained compression tests on Cowden till and those inferred from field and laboratory geophysical testing and observed in the PISA pile tests.

SITE CONDITIONS AND TRIAXIAL STRENGTH AND STIFFNESS PROFILES

Powell & Butcher (2003) and Zdravković *et al.* (2020a) set out general aspects of the Cowden site ground conditions. While the phreatic surface was located at 1.0 m depth, the profile is under-drained between 5 and 12 m. Figure 1 presents Ushev & Jardine's (2020) profiles of water contents and index properties, while Table 1 summarises the index properties, critical state parameters and shear strengths he determined over the depth ranges of interest. The profile comprises predominantly, low plasticity glacial clay till to around 12 m depth, below which lies a silty sand layer, then deeper till and chalk. Ushev (2018) confirmed a transition at 4.8 m between red/brown upper weathered till and lower, darker grey-brown, un-weathered layers and reports on how weathering affects index and mechanical properties.

Grading curves of rotary cored intact Cowden till specimens from five depths are given in Figure 2. The stony till showed on average 7% gravel content and a combined silt and clay fraction ranges from 58 to 73% over the depth ranges considered in this paper. Block samples showed gravel contents up to 12%, as discussed later. However, reference to Clarke (2018) indicates that matrix dominated behaviour should be expected in all cases, with the gravel content not affecting the till's overall shearing resistance. Systematic testing of soil mixtures, as by Jafari & Shafiee (2004), demonstrates that adding 20% or more gravel has no

discernible impact on static or cyclic undrained shearing response or pore-pressure behaviour of clays. However, it is natural to expect that adding significant fractions of large rigid particles should increase mass stiffness and modify the positions of virgin compression and critical state lines in void ratio-effective stress space. Following Gens & Hight (1979), Ushev (2018) eliminated the effects of gravel content when defining the critical state parameters listed in Table 1 by expressing these in terms of intergranular void ratio.

It is notoriously difficult to measure in-situ lateral stresses in strong glacial till. Considering the upper weathered layer's apparent over-consolidation ratios and applying empirical correlations developed for monotonically over-consolidated waterborne sediments led Powell & Butcher (2003) to interpret relatively high K_0 values (up to 2.8) for Cowden. However, Jardine (1985) argued that glacial deposition, which may involve glacio-tectonic pushing, basal shearing, freeze-thaw cycles, aerial exposure leading to desiccation in addition to vertical loading by ice, could not be represented as a simple 1-D compression process. The site's CPT and undrained shear strength profiles appear incompatible with uniform K_0 loading. Also, the existence of extensive vertical fissuring indicated prior desiccation, which does not impose high K_0 . Furthermore, attempts to impose high K_0 values in laboratory tests led to excessive axial straining; Ushev (2018). These considerations led the PISA team assuming a lower K_0 of 1.5 uniformly down to 7 m, falling to 1.0 at greater depths when setting in-situ effective stress conditions for numerical modelling and stress-path laboratory testing (Zdravković *et al.*, 2020a, 2020b).

Figure 3 presents the elastic shear stiffness (G_0) profiles defined by Ushev's (2018) multi-axis laboratory bender element measurements on intact specimens re-consolidated to the above in-situ K_0 stresses and groundwater conditions. The shear moduli determined in vertical and horizontal planes (G_{vh} , G_{hv} and G_{hh}) plot relatively close together. However,

Ushev (2018) and Ushev & Jardine (2020) showed that intact Cowden till only manifests elastic behaviour in triaxial compression tests over very small stress or strain excursions involving axial strains $< 0.002\%$. Regression analyses of the initial linear portions of the tests' q - ε_v traces, as described by Ushev (2018) identified the till's initial elastic undrained vertical Young's moduli (E_v^u). Figure 4 shows how secant E_v^u degrades abruptly at larger strains down to small proportions of the initial 'elastic' maxima, with steeper stiffness decay under compression than in extension. Ushev (2018) showed that the E_v^u decay curves present modest variation with depth when plotted as $[E_v^u/p_{\text{ref}}] / [p'/p_{\text{ref}}]^N = f(\varepsilon_v)$, where $p_{\text{ref}} = 101.3 \text{ kPa}$, p' is the applied mean effective stress at corresponding ε_v level, exponent $N = 0.5$ and axial strain ε_v is equal to the shear strain invariant ε_s under undrained triaxial conditions. Maximum Young's moduli profiles from undrained triaxial compression and extension tests, $E_{v0}^{u,\text{TXC}}$ and $E_{v0}^{u,\text{TXE}}$, are plotted in Figure 3 as equivalent shear stiffnesses with $G = E_{v0}^u/3$, as expected for isotropic soils. The triaxial 'G' values evidently fall far below the bender element G_{vh} and G_{hh} trends.

Undrained triaxial compression and extension shear strength (S_u^{TXC} and S_u^{TXE}) profiles indicated, on average, 25% higher strengths in compression than extension, as shown in Figure 5 after Ushev & Jardine (2020). The till displayed a ductile response in compression, with continuous barrelling until ultimate failures developed at axial strains greater than 25%. In contrast, axial extension led to non-uniform necking failures after less than 10% axial strain, so the $S_u^{\text{TXC}}/S_u^{\text{TXE}}$ ratio could give a misleading impression of 'anisotropy'.

RATIONALE FOR INVESTIGATING MECHANICAL ANISOTROPY

Class A (Lambe, 1973) predictions were made by Zdravković *et al.* (2020b) for the PISA pile tests by advanced three-dimensional Finite Element analyses that incorporated non-linear

1 stiffness-strain functions and critical state modelling parameters interpreted from the Cowden
2 ground investigations. Good agreement was found for the field load-displacement curves,
3 including slightly conservative ultimate peak load values, with stiffness relationships that
4 were scaled to match the G_{vh} and G_{hh} traces from in-situ seismic or laboratory bender element
5 tests. The same agreement could not be achieved by applying the triaxial Youngs modulus
6 data directly.

7 Noting that earlier triaxial and HCA testing programmes on stiff, high OCR,
8 sedimentary clays had indicated far greater horizontal than vertical stiffness (Gasparre *et al.*
9 2007, Brosse *et al.* 2016), it was decided to conduct triaxial and HCA tests and establish
10 whether comparable stiffness anisotropy existed that might help reconcile the laboratory
11 triaxial data with the pile lateral loading behaviour observed in the field. Taking HCA
12 samples to failure while controlling the intermediate principal stress ratio ($b = (\sigma_2 - \sigma_3) / (\sigma_1 - \sigma_3)$)
13 and the major principal stress orientation α defined with respect to the vertical could also
14 explore any shear strength anisotropy. Ushev (2018) describes parallel studies that examined
15 how mean effective stress increases caused by pile installation, as identified in field tests by
16 Bond & Jardine (1991), Lehane & Jardine (1994) and Pellew & Jardine (2008) may also have
17 affected the till's stiffness locally around the PISA pile shafts.

18 **SAMPLES AND FABRIC CHARACTERISTICS**

19 Intact till specimens were prepared from either nominally 100 mm outer diameter cores taken
20 with a triple-barrel, wireline, Geobore-S system down to around 13 m, or high-quality block
21 samples taken in the top 3.5 m of strata. The overall rotary coring recovery rate (42%) was
22 affected by the till's hard gravel inclusions; layers presenting more than 10% gravel could not
23 be recovered satisfactorily.

Micro-scale observations reported by Powell & Butcher (2003) did not identify any clearly orientated fabric in the till. However, the upper weathered till contained frequent vertical to sub-vertical planar, moderately spaced and open fissures. Ushev (2018) and Liu (2018) concentrated on specimens from below 1.5 m to avoid the most heavily weathered and highly variable layers. The till may have been subjected to directionally applied glacial shearing and developed anisotropy that could deviate from the usually assumed cross-anisotropic pattern. However, in the absence of any clear evidence regarding anisotropy in horizontal planes, the conventional transverse-isotropic framework was adopted to design and interpret the tests.

PRINCIPLES, APPARATUSES AND PROGRAMME

Linear elastic cross-anisotropic stiffness framework

Treating the till's initial linear elastic behaviour as being cross-anisotropic and strain-rate independent, following Kuwano & Jardine (2002), Gasparre *et al.* (2007) and Nishimura (2014b), allowed moduli from dynamic and static procedures to be considered as compatible and inter-changeable. The cross-anisotropic compliance relationship under drained triaxial conditions is then:

$$\begin{Bmatrix} \Delta \varepsilon_v \\ \Delta \varepsilon_h \end{Bmatrix} = \begin{bmatrix} \frac{1}{E'_v} & -\frac{2\nu'_{hv}}{E'_h} \\ -\frac{\nu'_{vh}}{E'_v} & \frac{1-\nu'_{hh}}{E'_h} \end{bmatrix} \times \begin{Bmatrix} \Delta \sigma'_v \\ \Delta \sigma'_h \end{Bmatrix} \quad [1]$$

Where $\Delta \sigma'_v$ and $\Delta \varepsilon_v$, $\Delta \sigma'_h$ and $\Delta \varepsilon_h$ are the effective stress and normal strain increments in the vertical and horizontal directions, while E'_v and E'_h are the corresponding drained Young's moduli. ν'_{vh} is the Poisson's ratio for horizontal normal strain caused by vertical strain while ν'_{hh} is the ratio for the horizontal strain response caused on one horizontal (h_1) axis by

horizontal straining on a second, independent, orthogonal horizontal (h_2) axis and ν_{hv}' is the ratio for vertical normal strain induced by horizontal straining. Five independent parameters, E_v' , E_h' , ν_{hv}' (or ν_{vh}'), G_{vh} , G_{hh} (or ν_{hh}'), are required to characterise such behaviour.

Under undrained conditions, the number of independent parameters further reduces to three (E_v^u , E_h^u , G_{vh}), incorporating the following additional restrictions (Gibson, 1974).

$$\nu_{vh}^u = \frac{1}{2} \quad [2]$$

$$\nu_{hh}^u = 1 - \nu_{hv}^u = 1 - \frac{1}{2} \frac{E_h^u}{E_v^u} \quad [3]$$

Shear moduli in the vertical (G_{vh} and G_{hv}) and horizontal (G_{hh}) planes can be measured non-destructively by bender elements. The Young's moduli and Poisson's ratios can be determined or derived by imposing a set of small stress or strain increments (probes) uniaxially in the vertical or horizontal direction, interpreting these with either a drained approach (see Lings (2001), Kuwano (1999), Kuwano & Jardine (2002) or Brosse *et al.* (2016)), or through the combined drained and undrained method proposed by Nishimura (2014a, 2014b).

The drained approach requires highly accurate local axial (or vertical) and radial (or horizontal) strain increment measurements. The drained Young's moduli and Poisson's ratios are found as:

$$E_v' = \frac{\Delta \sigma_v'}{\Delta \varepsilon_v} \quad [4]$$

$$E_h' = \frac{4AG_{hh}}{A + 2G_{hh}} \quad [5]$$

Where $A = \Delta \sigma_h' / \Delta \varepsilon_h$, as measured from radial probing tests. The undrained Young's moduli, E_v^u and E_h^u , can then be derived (Lings, 2001, Brosse *et al.*, 2016).

$$E_v^u = \frac{E_v' [2(1 - \nu_{hh}') E_v' + (1 - 4\nu_{vh}') E_h']}{2(1 - \nu_{hh}') E_v' - 4(\nu_{vh}')^2 E_h'} \quad [6]$$

$$E_h^u = \frac{E_h' [2(1 - \nu_{hh}') (E_v')^2 + (1 - 4\nu_{vh}') E_v' E_h']}{[1 - (\nu_{hh}')^2] (E_v')^2 + (1 - 2\nu_{vh}' - 2\nu_{vh}' \nu_{hh}') E_v' E_h' - (\nu_{vh}')^2 (E_h')^2} \quad [7]$$

The E_h' and E_h^u expressions are evidently more complex and sensitive to experimental errors. Recognising the difficulties in undertaking reliable radial strain measurements (Ackerley *et al.*, 2016), Nishimura (2014a, 2014b) proposed a combined drained and undrained probing approach that relies only on the generally higher quality axial strain data. Undrained vertical probes determine the undrained vertical Young's modulus, E_v^u and allow the drained horizontal Young's modulus, E_h' , to be derived as:

$$E_h' = \frac{4G_{hh} (E_v^u - E_v')}{E_v^u + 4a^2 E_v^u E_v' G_{hh} + (1 - 4a E_v') G_{hh} - E_v'} \quad [8]$$

Where $a = \nu_{hv}' / E_h' = - \Delta \varepsilon_v / (2 \Delta \sigma_h')$. The undrained horizontal Young's modulus, E_h^u , can be determined similarly using Equation [7].

The current study incorporated multiple drained and undrained stress probes in conjunction with high resolution radial strain measurements, allowing both approaches to be applied.

Quantification of full-strain multi-axial undrained Young's moduli in HCA

Hollow Cylinder Apparatus (HCA) element testing offers a robust approach for assessing mechanical anisotropy, see Hight *et al.* (1983), Nishimura *et al.* (2007) or Brosse *et al.* (2016), (2017). Four external boundary stresses are imposed on specimens: axial force (F_a), torque (T) plus inner (p_i) and outer (p_o) cell pressures, as illustrated in Figure 6. The applied external actions can be averaged across the HC specimen, and the vertical, radial, circumferential and

torsional shear stresses (σ_z , σ_r , σ_θ and $\tau_{z\theta}$) and strains (ε_z , ε_r , ε_θ and $\gamma_{z\theta}$) are derived as set out by Hight *et al.* (1983). HCA testing offers four degrees of freedom that enable independent control of the p , q , α and b stress parameters.

$$p = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \quad [9]$$

$$q = \sigma_1 - \sigma_3 \quad [10]$$

$$\alpha = \frac{1}{2} \tan^{-1} \left(\frac{2\tau_{z\theta}}{\sigma_z - \sigma_\theta} \right) \quad [11]$$

$$b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \quad [12]$$

The HCA systems can be programmed to follow desired consolidation or shearing stress paths. The current study employed a controlled- $\alpha_{d\sigma}$ stress path shearing scheme, with $\alpha_{d\sigma}$ representing the direction of the axis of the major principal stress increment controlled as

$$\alpha_{d\sigma} = \frac{1}{2} \tan^{-1} \left(\frac{2\Delta\tau_{z\theta}}{\Delta(\sigma_z - \sigma_\theta)} \right) \quad [13]$$

Assuming cross-anisotropic conditions, the r - θ plane in HCA tests is equivalent to the horizontal plane in triaxial tests, except that $E_h = E_r = E_\theta$ and $\nu_{hh} = \nu_{r\theta} = \nu_{\theta r}$. In principle, linear and non-linear stiffness parameters and their degradation trends can be measured directly through drained incrementally uniaxial HCA tests, as described by Zdravkovic & Jardine (1997) or Gasparre et al (2007). However, equivalent undrained Young's moduli can also be determined directly over the full strain range $\alpha_{d\sigma}$ tests through Brosse's (2012) equations:

$$E_v^u = \frac{2\Delta\sigma_z - \Delta\sigma_r - \Delta\sigma_\theta}{2\Delta\varepsilon_z} \quad [14]$$

$$E_{hr}^u = \frac{\Delta\sigma_r(\Delta\sigma_r - 2\Delta\sigma_z) - \Delta\sigma_\theta(\Delta\sigma_\theta - 2\Delta\sigma_z)}{\Delta\varepsilon_r(\Delta\sigma_r + \Delta\sigma_\theta - 2\Delta\sigma_z) + \Delta\varepsilon_z(\Delta\sigma_\theta - \Delta\sigma_z)} \quad [15]$$

$$E_{h\theta}^u = \frac{\Delta\sigma_r(\Delta\sigma_r - 2\Delta\sigma_z) - \Delta\sigma_\theta(\Delta\sigma_\theta - 2\Delta\sigma_z)}{\Delta\varepsilon_\theta(\Delta\sigma_r + \Delta\sigma_\theta - 2\Delta\sigma_z) + \Delta\varepsilon_z(\Delta\sigma_r - \Delta\sigma_z)} \quad [16]$$

$$G_{z\theta} = \frac{\Delta\tau_{z\theta}}{\Delta\gamma_{z\theta}} \quad [17]$$

The above approach allows multiple undrained moduli to be determined efficiently in single HCA tests that apply or measure all stress and strain components simultaneously as the test progresses from small strains to ultimate shear failure; Brosse *et al.* (2016).

Test equipment and environment

Modified Bishop & Wesley (1975) cells were employed for the triaxial experiments on 200mm high, 100mm diameter specimens. Jardine *et al.* (1984), Kohata *et al.* (1997) and Tatsuoka *et al.* (1999) showed that pre-failure straining is over-recorded in routine triaxial testing, which can be avoided by deploying local strain sensors. The systems shown in Figure 7 were employed with: (a) ‘floating’ type LVDT local axial strain sensors (Cuccovillo & Coop, 1997) spaced 120° apart and gauged over the specimen's central 70 mm portion; (b) ‘fixed’ LVDTs to measure local radial deformations from three radially spaced points around the specimen's middle height section through the Ackerley *et al.* (2016) hinged L-shape system. A mid-height sensor was employed to track local pore water pressure generation and dissipation in the central zone; three pairs of platen-mounted or lateral ‘T-configuration’ piezoceramic bender element shear-wave transducers were also installed.

The HCA tests were performed with the computer-controlled Imperial College Resonant Column Hollow Cylinder Apparatus (ICRCHCA), employing a similar configuration to Nishimura (2006) and Brosse (2012). The testing system is equipped with a

Hardin oscillator to monitor shear stiffness ($G_{z\theta}$) and damping ratio non-destructively, independently of the consolidation or loading conditions. The HCA specimens were 180 to 195 mm high, with 71 mm outside and 38 mm inside diameters.

The HCA and triaxial systems employed compressed air to apply axial load, cell and back pressures through either a double-acting Bellofram or Imperial College type air/water interfaces, which minimise any air diffusion during long-term testing. Direct air-water contact occurred in the HCA outer cell chamber as the Hardin oscillator is non-submersible. Undesirable air diffusion was countered, following Nishimura (2006), by gradually refreshing the cell chamber water with de-aired water at regular intervals throughout testing, while maintaining constant specimen stress conditions. The applied pressures were regulated through electro-pneumatic Manostat valves controlled through stepper motors set to deliver 0.07 kPa per step. HCA torque was applied mechanically through a rotary table regulated with a stepper motor, to provide 6.25×10^{-5} degrees rotation per step. HCA specimen deformations were gauged globally by a cantilevered axial displacement transducer, two volume-gauges, and platen-to-platen torsional rotations were sensed through non-contacting proximity transducers, measuring against a shaped cam fixed to the specimen top. System compliance errors were eliminated through careful calibration. The HCA system provided nominal resolutions and precision ranges of $(1-4) \times 10^{-4}\%$ for all the global strain components, which were less precise and representative than those achieved with high-resolution triaxial local strain sensors and were unable to resolve down to the strain levels at which the Cowden till manifests its elastic stiffness maxima in triaxial tests. It is also recognised that global systems tend to over-record local straining. The triaxial and HCA test laboratories' temperatures were controlled to 21°C with $\pm 0.3^\circ\text{C}$ fluctuations over daily periods. The triaxial specimens and instruments were enclosed within a pressurised de-aired water volume around

20 times larger than the specimen. Bubble wrap and aluminium foil insulation was wrapped around the cells during probing to reduce temperature variations further to around $\pm 0.1^{\circ}\text{C}$. Liu (2018) and Ushev (2018) give full details of the apparatuses, instruments and their performance.

Challenges in specimen preparation

Nishimura (2006) developed effective procedures for preparing HCA specimens of natural stiff to very stiff (fissured) plastic London clay which allowed him to recover the same triaxial compression response in HCA tests as conventional triaxial experiments on 100mm diameter, 200mm high, cylindrical samples. Liu (2018) outlines the similar procedures he applied to alleviate disturbance with the Cowden till. Additional measures and perseverance were required to cope with gravel content and the systems of fissuring encountered in samples trimmed from 350mm $\times$ 350mm cubical blocks cut at around 2.9 m depth from sampling pits. X-ray CT scanning was undertaken that could locate gravel particles and cavities with maximum dimensions greater than 10mm when scanning half or smaller blocks. However, abundant smaller hard gravel particles, fissures and micro-cracks could not be resolved sufficiently well at useful scales, so HCA and triaxial specimen formation proceeded by making repeated careful attempts with specially hardened saws and other tools. The Cowden HCA specimens' 16.5 mm wall thickness rendered them far more susceptible to surface defects, fissures, hard gravel inclusions and collapses during trimming than 100 mm diameter solid cylinder triaxial samples. Attempts had to be abandoned when heavily fissured or stony zones, which tended to collapse or crumble, were encountered. Where required, small external pits and cavities with nominal dimensions < 10 mm were patched with manually remoulded till and several hours were required to prepare each HCA specimen. The average success rate in forming acceptable solid cylindrical test specimens from block

1 samples was 50%, which reduced to around 20% for HCA specimens with the desired final
2 dimensions and quality. Recognising that Cowden HCA samples were more likely to be
3 affected deleteriously by trimming than plastic London clay, two undrained triaxial
4 compression HCA tests were performed to assess the impact of specimen preparation on
5 undrained stiffness and shear strength.

6 *Specimen saturation*

7 Stiff low plasticity till samples can dry rapidly during laboratory trimming. Evaporation was
8 alleviated by trimming in a high humidity environment and assessed by checking the
9 specimens' degrees of saturation (S_r). Other information was obtained when isotropic
10 confining pressures were applied to induce a positive pore pressure in the specimens which
11 indicated their initial suction or mean effective stress (p_0'), which was taken as the applied
12 cell pressure minus the measured pore pressure. The same p_0' was maintained over the
13 saturation stage during which back pressure was elevated to typically 250-400 kPa so that B
14 values greater than 0.95 could be reached. In some cases where the targeted B value could not
15 be achieved initially, a two-stage consolidation scheme was followed within which the back
16 pressure was raised further to saturate the specimen during the swelling stage of the
17 reconsolidation. This procedure induced minor errors in the recorded consolidation strains.

18 As shown in Tables 2 and 3, the triaxial samples had high initial degrees of saturation
19 S_r , while the average S_r (88.2%) of the successfully formed (and tested) HCA specimens was
20 significantly lower due to their far greater trimming times and larger surface area to volume
21 ratios. The HCA specimens' p_0' values were also around 25% higher than those of the 100
22 mm triaxial specimens prepared from adjacent blocks.

Reconsolidation procedure and testing programmes

The reconsolidation process typically involved reducing the samples' p_0' (applied at 2 kPa/hour) while moving to the estimated in-situ stresses. Constant effective stresses were maintained until the residual creep strain rates met the criteria specified in Table 4. A minimum pause period of three weeks was required prior to the probing tests. Such pauses are essential to measuring representative stiffnesses; Gasparre *et al.* (2014). Similar pause stages were imposed between multiple probing tests to ensure that any residual straining had ceased before another stress probe was executed.

The triaxial probing tests' initial conditions are summarised in Table 2, along with those for the 'standard' K_0 -consolidated undrained compression (KUC) and extension (KUE) tests performed on samples from nearby depths, noting the different K_0 conditions applied for the specimens from upper (2 to 5.35 m) and deeper horizons. The probing comprised small (typically 2 kPa) drained vertical (dv'), horizontal (dh') and undrained vertical (dq) stress increments at rates of 0.1-0.2 kPa/hour. Some tests were repeated to explore different probing options and assess data variability. Each suite of probing tests on single specimens took 2-2.5 months to complete, while the KUC/KUE tests typically extended for three weeks.

The initial vertical undrained compression and extension moduli determined from the KUC and KUE tests ($E_{v0}^{u, TXC}$ and $E_{v0}^{u, TXE}$) on nominally identically specimens should, if the till was elastic, be equal (Brosse *et al.*, 2016). However, Ushev (2018) observed that triaxial compression tangents appeared stiffer initially than in extension, with $E_{v0}^{u, TXC}/E_{v0}^{u, TXE} = 1.23$ on average in his paired tests on rotary core samples from identical depths, although their general trends with depth were similar. It is possible that, despite the long periods allowed in the KUC/KUE tests for creep straining to diminish, non-zero rates of residual straining,

(which tended to show axial extension when $K_o > 1$), led to equal and opposite under-measurement of vertical stiffnesses under extension and overestimation in compression.

The Cowden till's shear strength and stiffness anisotropy were further examined through a suite of undrained HCA $\alpha_{d\sigma}$ -controlled and simple shear (SS) tests on specimens from a single depth of 2.9 m. As summarised in Table 3, the ultimate failure states were controlled to keep the intermediate principal stress parameter (b) close to 0.5, while covering a range of final α_f values between 0 to 90°. These conditions are far from those where stress-strain non-uniformities cause concern (Symes, 1983). When $b = 0.5$, the stress variables (t , s' , q , p') are linked simply to the Cauchy stress invariants, and the effective stress path inclinations $d(\sigma_z - \sigma_\theta)/d(2p')$ do not vary with α ; Brosse *et al.* (2017). Two undrained triaxial compression (HCA TXC) tests were also performed.

As illustrated in Figure 8, the main ($\alpha_{d\sigma}$ -controlled) HCA test series involved applying undrained stages in which b was reduced from 1 to 0.5 at a rate of -0.1/hour while maintaining constant q , p and α , after which at least 24 hours was allowed for creep to stabilise. Shearing stress paths involving constant $\alpha_{d\sigma}$ directions were then applied by controlling the dominating the strain component (either axial strain or torsional shear strain) at fixed deviatoric strain rates of around 0.15%/hour. Given the low initial in-situ effective stresses, principal stress axis rotation took place rapidly over the early stages of shearing.

The HCA simple shear (HCASS) tests involved shearing directly from in-situ stresses, without applying any b -change. The radial and circumferential strains (ε_r and ε_θ) were kept close to zero by maintaining constant specimen and inner cavity volumes (V and V_i), while mechanical and servo-control systems prevented axial straining, allowing only torsional rotation to occur. As shown later, the tests culminated with b and α values close to 0.5 and 45°, as would, $b = 0.5$ constant- $\alpha_{d\sigma}$ tests set to give $b = 0.5$ and $\alpha_f = 45^\circ$. An additional HCA

1 simple shear test was performed on a shallow (0.5m depth) specimen to characterise the
2 upper (higher plasticity) weathered layer, which was important to the PISA lateral pile
3 loading response.

4 **SAMPLING BIAS IMPLICIT IN HCA TESTING**

5 One important issue is the potential impact of the sampling bias that resulted from the till's
6 variable gravel contents. The Cowden till samples are categorised in Table 5 according to the
7 degree to which they could be cored in the field or formed into triaxial or HCA test
8 specimens in the laboratory. The key controlling feature was the samples' percentage, by
9 mass, of gravel particles exceeding 2 mm size. HCA testing was only possible with Group D
10 specimens, whose contents fell below 4%. Noting the spread of distributions shown in Figure
11 2, the HCA programme was unintentionally biased towards lower-than-typical gravel
12 samples.

13 Comparing the dynamic shear stiffnesses measured in the vertical plane (G_{vh} or $G_{z\theta}$) by
14 triaxial bender element (G_{vh}^{BE}), HCA resonant column ($G_{z\theta}^{RC}$) and in-situ downhole seismic
15 CPT (G_{vh}^{SCPT}) techniques allowed checking of how sampling and specimen preparation
16 affected stiffness. While the triaxial G_{vh}^{BE} data generally matched the in-situ SCPT G_{vh}^{SCPT}
17 traces, the HCA $G_{z\theta}^{RC}$ values were around 45% below the triaxial G_{vh}^{BE} trend. It was
18 concluded that high frequency bender element wave propagation through the solid triaxial
19 specimens was more representative of in-situ conditions than the lower frequency resonant
20 column $G_{z\theta}^{RC}$ measurements made on hollow samples. The latter may have been more
21 susceptible to local inhomogeneities relating to fissures, gravel or infilled local voids.

22 The HCA specimens tested under undrained 'triaxial' conditions with equal inner and
23 outer cell pressures (the HCA TXC tests) provided indications of the small-strain vertical

undrained compression Young's moduli $E_{v0}^{u,HCA}$, which fell around 38% below the maxima $E_{v0}^{u,TXC}$ determined in triaxial tests at smaller, locally measured sensors. This discrepancy is larger than the 23% discrepancy in E_{v0}^u found in triaxial extension, which may have been the result of residual axial creep rates. However, the HCA global sensors' inability to resolve the elastic stiffness maxima and their tendency to over-record strains due to specimen-end defects probably contributed, along with the HCA specimens' lower-than-typical gravel contents. Comparable discrepancies between HCA and triaxial stiffnesses were noted in parallel studies on stiff plastic marine sedimentary clays by Brosse *et al.* (2016).

The till's shear strength characteristics were also affected. Figure 9 shows that the HCA TXC specimens' undrained shear strengths ($S_u^{HCA,TXC}$) were around 25% lower than the S_u^{TXC} values of standard triaxial tests on 100mm diameter samples from similar depths, this divergence is similar to that between triaxial compression S_u^{TXC} and extension S_u^{TXE} shear strengths, which was ascribed earlier to the latter's necking failure mode.

In summary, the HCA tests tended to be more susceptible than triaxial samples to having reduced gravel contents; suffer greater preparation disturbance; and be more sensitive to geometrical imperfections and local inhomogeneities. However, comparisons between monotonic triaxial tests run with both systems showed that the impact on shear strength and stiffnesses was not overwhelming; the HCA tests could be trusted to give credible information on the mechanical anisotropy of the matrix dominated till.

CROSS-ANISOTROPIC ELASTIC STIFFNESSES

The spread of the triaxial dv' , dh' and dq stiffness probing tests undertaken to examine stiffness anisotropy is summarised in Table 6. The symbols, $\pm$, $+$ or $-$ indicate whether (respectively) full load-unload cycles, half-cycle loading or unloading probing increments

had been applied. Provided that the strain excursions do not exceed the specimen's initial linear elastic range, the $\pm$, + or - styles of probing should give identical outcomes.

Figures 10 and 11 plot examples of the stress-strain loops recorded, covering the axial ($+\Delta\sigma_v'$ and $-\Delta\sigma_v'$) and radial ($+\Delta\sigma_h'$ and $-\Delta\sigma_h'$) probes in test PB2, while Figure 12 plots the specimen's response over full-cycle probes. Also shown are best fit linear regression lines and their slopes. Similar undrained probes provided information on vertical undrained Young's moduli (E_v^u). Given the very small strains considered, the individual data points inevitably show scatter. However, the cycles appear fully reversible and the probes gave similar linear slopes, regardless of the sign or size of the load increment over the ranges considered. The scatter reflects the difficulties of determining stiffness and Poisson's ratios at strains less than 0.002%, see Nishimura (2014a) and Ackerley *et al.* (2016).

The triaxial probing and bender element measurements provided the drained and undrained cross-anisotropic compliance parameters tabulated in Table 7. The undrained E_{v0}^u values determined by different routes fell relatively close together (except at 2 m depth), and the E_h^u parameters were consistent between the alternative approaches, with average deviations of 4% from the respective mean values. As noted earlier, the indirectly assessed E_h' and E_h^u values are more sensitive to minor variations in measurements than the E_v' and E_v^u outcomes. Table 7 confirms that Cowden till manifests anisotropy with elastic moduli that are stiffer under horizontal than vertical loading, with in most cases $G_{hh} > G_{vh}$, $E_h' > E_v'$ and $E_h^u > E_v^u$. The ratios, G_{hh}/G_{vh} , E_h'/E_v' and E_h^u/E_v^u , derived from the triaxial probing and HCA tests are plotted against depth in Figure 13, where they show generally consistent trends, despite the earlier discussed difficulties of determining small-strain stiffness by HCA testing. Also shown are the shear modulus ratios (G_{hh}/G_{vh} and G_{hh}/G_{hv}) determined from in-situ down-hole and cross-hole shear velocity tests reported by Powell & Butcher (2003). In

general, the till's stiffness anisotropy appeared to diminish with depth, which could be important for full-scale offshore monopiles, whose tips are often driven far deeper than 20m in such strata.

NON-LINEAR STIFFNESS CHARACTERISTICS OVER FULL STRAIN RANGE

The till marked elastic-plastic stiffness degradation with strain under HCA conditions is demonstrated in Figures 14-16, by plotting E_v^u , E_h^u and G_{vh} against: vertical strain ε_z ; average horizontal strain $(\varepsilon_r + \varepsilon_\theta)/2$; and torsional shear strain $\gamma_{z\theta}$, respectively. Least square linear regressions applied up to 0.005% strain provided estimates for the initial stiffness values. It is acknowledged that higher stiffnesses are likely to have applied within the initial linear, smaller-strain, portions of the test curves that could not be resolved with the global HCA strain sensing arrangements.

Despite scatter over their initial sections, the undrained Young's moduli exhibited consistent degradation trends. Narrow spreads were observed for each component, suggesting that the cross-anisotropic stiffness components are relatively insensitive to the shearing path followed, even over the non-linear range. The two HCA triaxial tests (CATC and CATC2), which employed $b = 1.0$, showed encouraging overlapping E_v^u trends that fall slightly below those of most $b = 0.5$ tests. The E_h^u curves plot above the E_v^u data trends, but also show greater variation because they are more sensitive to experimental errors. The monotonic torsional shear stiffnesses $G_{z\theta}$ presented in Figure 16 indicate similar degradation trends once $\gamma_{z\theta}$ exceeds 0.005%.

The initial ratio of horizontal to vertical Young's moduli (E_{h0}^u/E_{v0}^u) averaged 1.74 at strains $< 0.005\%$, while G_{hh0}/G_{vh0} was 1.64, indicating overall stiffer behaviour in horizontal planes, as in the triaxial probing tests. The derived undrained Poisson's ratios $\mu_{hh0}^u = 0.111$

and $\mu_{hv0}^u = 0.889$ deviate significantly from 0.5, as expected for ideal isotropic elastic materials. Overall, the HCA tests confirm that stiffness anisotropy persists beyond the till's linear elastic ranges.

UNDRAINED SHEAR STRENGTH ANISOTROPY

Stress-strain behaviour over large strains

As noted earlier, undrained triaxial compression tests manifested ductile responses and trended to stable critical states as axial strains approached or exceeded 30%. The maximum axial strains achievable in the HCA tests apparatus were smaller, at around 15%, and stress and strain non-uniformities become more marked in HCA tests at large strains. The discussion below recognises that the shear strength data may not be fully representative of behaviour at comparably large strains.

The shear strengths were considered in terms of Tresca undrained shear strengths $S_u (= q/2)$, secant Mohr Coulomb stress ratios t/s' (taking $c' = 0$) and the ultimate ratio of q/p' . The latter two criteria are exchangeable with $q/p' = 2t/s'$ when $b = 0.5$. Figure 17 presents overall trends of deviatoric stress ($q = \sigma_1 - \sigma_3$) against maximum shear strain ($\varepsilon_1 - \varepsilon_3$) for the seven HCA tests on samples from 2.9m depth. Five of the curves were predominantly ductile, without manifesting any peak up to $(\varepsilon_1 - \varepsilon_3) = 20\%$, while the two high α_f tests (CA6705 and CA9005) developed peaks at 8-9% shear strain. The deviatoric stresses were still climbing slowly at the end of the other tests. Strain localisation or rupture was rarely observed, and the recorded stress-strain responses were treated as representing continuous element behaviour.

Figure 17 reveals anisotropic shear strength trends. The passive test (CA9005) with $\alpha_f = 90^\circ$ showed the highest undrained shear strength ($S_u = q_{\max}/2$), while the sample that underwent active ($b = 0.5$) compression (CA0005) exhibited the lowest. The undrained shear

strengths of the specimens experiencing mainly torsional shearing (CA2305, CASS and CA6705) at intermediate α_f values spanned these limits, although CA2305 and CASS showed relatively steep final stress-strain curves and may have developed greater ultimate strengths if further straining had been possible. It is also interesting to note that the two HCA “triaxial” compression tests developed apparently higher shear strengths than the ‘plane-strain’ α_{ds} -controlled test CA0005. These trends are opposite to those expected for low OCR clays or silts: see for example Hashash & Whittle (1996), or Zdravković *et al.* (2001, 2002).

The evolving trends of t/s' ($= q/(2p')$) against shear strain are plotted in Figure 18, also indicating the corresponding mobilised ϕ_{mob}' angles. As in the ‘Tresca’ analyses above, the highest t/s' ($= q/(2p')$ at $b = 0.5$) was noted with $\alpha_f = 90^\circ$ and lowest at $\alpha_f = 0^\circ$, with the intermediate α_f results lying in-between. The high α_f tests: CASS (48.2°), 67° , 90° , showed relatively stable t/s' trends, while marked peaks were observed in the two lower $\alpha_f = 0^\circ$ and $\alpha_f = 23^\circ$ tests (CA0005, CA2305). The effective stress paths projected into the $(\sigma_z - \sigma_\theta)/2 - p'$ plane plotted in Figure 19 show paths that all curve to the right after following initially contractant patterns. The Cowden till dilates strongly as it is sheared towards large strain critical state conditions, in contrast to the high OCR plastic marine clays tested by Brosse *et al.* (2017) which bifurcated after relatively modest shear strains and started to develop residual shear structures after reaching their peak resistances.

Undrained shear strength anisotropy

The shear strength anisotropy revealed from the above stress-strain traces is depicted further by projecting the stress paths in the $\tau_{z\theta} - [(\sigma_z - \sigma_\theta)/2]$ plane, as shown in Figure 20. Tests on identical isotropic samples would follow a half circle in this space, but the Cowden till’s stress paths, and the annotated 10% shear strain and maximum shear strain (ultimate) state points, extend greatly to the left and are skewed on the right-hand side. Taken together with

the S_u against α_f plot given in Figure 21, these data confirm that the highest shear strength is developed under nominally plane strain compression conditions ($\alpha_f = 90^\circ$) and the lowest at $\alpha_f = 0^\circ$, under active plane strain stresses. The maximum S_u values is 1.37 times the minimum, indicating mildly anisotropic Tresca failure characteristics.

Variations were also seen between undrained shear strengths from standard triaxial compression and extension tests. Ushev (2018) reported the opposite trend for a mean S_u^{TXC}/S_u^{TXE} ratio of 1.25 from his nine tests on specimens from comparable depths. However, his triaxial extension tests all developed necking failures and strains localised before any stable global failure could be achieved. They also imposed different b values in compression and extension and so give a misleading indication of the till's shear strength anisotropy.

The till's anisotropic shear strength characteristics are also summarised in terms of stress ratio t/s' and mobilised shear resistance angle (ϕ_{mob}') at 10% shear strain, peak and ultimate states in Figures 22 and 23. As with the Tresca criterion, the minimum t/s' develops at $\alpha_f = 0^\circ$, and the maximum at $\alpha_f = 90^\circ$. A ratio of 1.19 exists between the maximum and minimum, with the highest value corresponding to remarkably high secant ϕ_{mob}' angle. Intermediate t/s' values are found between the limits. The anisotropy in S_u follows a more accented pattern than that for t/s' , indicating that both fabric anisotropy and a tendency for the till's dilatancy to vary with α contribute to the overall directional dependency of undrained shear strength.

Figure 23 shows the ϕ_{mob}' values developed in the $b = 0.5$ HCA tests were markedly higher than those observed applying under triaxial compression. It is reassuring that the HCA 'triaxial' compression tests reached ultimate mobilised ϕ' angles of around 27° , matching the critical state ϕ' values from routine intact triaxial compression tests by Ushev (2018). It is also interesting to note that the $b = 0.5$ "plane strain" HCA active tests gave only slightly (3°)

higher ϕ' values than the $b = 0$ triaxial compression conditions after reaching similarly large strains.

Undrained HCA simple shear (HCASS) behaviour

Design procedures based on conventional simple-shear tests have been proposed for multiple applications, including embankment foundations and laterally loaded offshore piles (Jeanjean et al 2017). However, only two components of the stress and strain tensors can be tracked in such tests, so limiting their interpretation for critical state based geotechnical modelling. However, these difficulties were overcome fully in the HCA simple shear tests on Cowden till. Liu (2018) reports the details of the stress paths followed, showing that σ_z , σ_θ and σ_r remained relatively stable under torsional loading, leading to near vertical stress paths in the $\tau_{z\theta}$ - $[(\sigma_z - \sigma_\theta)/2]$ plane as shown in Figure 24. Figure 25 demonstrates how the major principal stress direction α , the b ratio and deviatoric stress q varied throughout. The b - α paths plot only slightly below the dashed $b = \sin^2(\alpha)$ curve, which applies when the inner and outer cell pressures are identical. The trends confirmed little changes developed in σ_r and σ_θ from their initial (equal) values under torsional shearing, a factor that alleviates stress non-uniformity in HCA testing (Hight *et al.*, 1983). Rapid rotation of the major principal stress direction from 90° took place after the onset of shearing, leading to ultimate α values between 45° to 50° , and ultimate b values of around 0.45 under the conditions imposed.

DISCUSSION ON IMPLICATIONS FOR PRACTICAL ANALYSIS

The triaxial and HCA experiments were conducted to assess whether Cowden till shows a stiffer response to horizontal rather than vertical loading. The laboratory tests verified this outcome and so helped to explain the field tendency for the PISA monopile lateral loading tests to indicate stiffer, non-linear, behaviour than expected from triaxial compression tests.

1 The patterns of anisotropy, which were shown to be broadly compatible with those applying
2 in stiff, geologically-aged, high *OCR* marine clays have significant implications for a broad
3 range of applications from offshore foundations, to urban deep basement design and tunnel
4 settlement analysis, as demonstrated for example by Addenbrooke *et al.* (1997).

5 The patterns of shear strength anisotropy exhibited in the HCA experiments differed
6 greatly from those understood for, and applied in, low *OCR* clay stability analyses for
7 foundation problems ranging from deep excavations (Hashash & Whittle, 1996) to low
8 length-to-diameter ratio offshore suction caissons or embankments on soft clay: see
9 Zdravković *et al.* (2001, 2002). Shear strengths developed in the ‘passive’ zone and in
10 transitional ‘simple shear’ zones of such problems are likely to be greater in Cowden till than
11 in the ‘active’ region and fall closer to the shear strengths developed under triaxial conditions.
12 These outcomes are compatible with Zdravković *et al.* (2020b)’s successful prediction of the
13 ultimate bearing pressures developed by the PISA monopiles through critical state modelling
14 based on Ushev’s (2018) triaxial compression tests.

15 SUMMARY AND CONCLUSIONS

16 The mechanical anisotropy of Bolders Bank glacial till sampled at Cowden (UK) has been
17 explored through suites of locally instrumented triaxial tests that incorporated multi-axial
18 bender element systems and high-resolution measurements of the till’s anisotropy at very
19 small strains. Parallel HCA tests allowed the till’s anisotropy to be examined from small
20 strains up to failure. Ten main conclusions are drawn:

- 21 1. The till only manifested fully recoverable, and markedly anisotropic, linear elastic
22 stiffness over a very small strain range in triaxial probing tests.

2. The main programme of tests on block samples showed far higher drained and undrained stiffnesses under horizontal than vertical loading, with G_{hh0}/G_{vh0} , E_{h0}'/E_{v0}' , E_{h0}^u/E_{v0}^u ratios greater than unity
3. The till's degree of stiffness anisotropy tended to be greatest at shallow depth and diminish with depth.
4. Complementary HCA experiments provided further valuable information, confirming that stiffness anisotropy persisted as the undrained moduli, E_h^u , E_v^u and G_{vh} degraded after exceeding their initial linear ranges.
5. HCA testing proved challenging. Specimen formation was only feasible in blocks with gravel contents below 4% and the procedures adopted led to significantly lower stiffness and shear strength S_u than comparable tests in triaxial apparatus.
6. However, the discrepancies between HCA and triaxial tests were not overwhelming, and the till's relative stiffness anisotropy did not appear to be affected unduly.
7. The till, which shows a ductile response under undrained triaxial shearing, manifested a pattern of shear strength anisotropy which differed considerably from the patterns expected with low *OCR* sediments.
8. The minimum strengths, as defined by either Tresca or Mohr-Coulomb criteria, developed at $\alpha = 0^\circ$ and the maximum at $\alpha = 90^\circ$. The torsional shear mode ($\alpha_f \approx 45^\circ$) leads to strengths in between the active (vertical) and passive (lateral) compression modes. However, the average shear strength developed over the full range of α values did not deviate greatly from the triaxial compression value S_u^{TxC} .
9. The till's Tresca shear strength anisotropy was more marked than that interpreted from the Mohr-Coulomb criterion parameters, indicating that both fabric anisotropy and a

1 tendency for the till's dilatancy to vary with α contribute to the overall directional
2 dependency of undrained shear strength.

3 10. The observation that higher ultimate shear strengths and stiffness apply (over the full
4 strain range) under horizontal rather than vertical loading has important practical
5 implications for cases involving principal stress axis rotation, as is common in
6 applications ranging from deep excavations, to tunnelling, earthworks and offshore
7 foundations; see Hashash & Whittle (1996), Addenbrooke *et al.* (1997), Zdravković *et*
8 *al.* (2001), Byrne *et al.* (2017), Jeanjean *et al.* (2017) or Jardine (2020).

DATA AVAILABILITY STATEMENT

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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LIST OF NOTATION

PISA JIP	Pile soil analysis joint industry project
α	Angle between the vertical and the direction of σ_1 axis
$\alpha_{d\sigma}$	Angle between the vertical and the direction of $\Delta\sigma_1$ axis
α_f	Angle between the vertical and the direction of σ_1 axis at ultimate failure
$\varepsilon_z, \varepsilon_r, \varepsilon_\theta$	Axial, radial and circumferential strains respectively in HCA space
$\varepsilon_v, \varepsilon_h$	Vertical (axial) and horizontal (radial) strains respectively in triaxial space
$\varepsilon_1, \varepsilon_3$	Major and minor principal strain respectively
ε_s	Shear strain invariant
$\gamma_{z\theta}$	Torsional shear strain
$\tau_{z\theta}$	Torsional shear stress
ϕ'	Effective angle of shearing resistance
ρ_{bulk}	Bulk density
$\sigma_1, \sigma_2, \sigma_3$	Major, intermediate and minor principal stresses respectively
$\sigma_z, \sigma_r, \sigma_\theta$	Axial, radial and circumferential stresses respectively
σ_v', σ_h'	Vertical and horizontal effective stress respectively
$\nu_{vh}', \nu_{hv}', \nu_{hh}'$	Drained Poisson's ratios in the cross-anisotropic elastic model
$\nu_{vh}^u, \nu_{hv}^u, \nu_{hh}^u$	Undrained Poisson's ratios in the cross-anisotropic elastic model
b	Intermediate principal stress factor ($= (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$)
B	Pore pressure coefficient
CPT, SCPT	Cone penetration test, seismic cone penetration test
BE	Bender element test
e	Void ratio
E_v', E_h'	Drained vertical and horizontal Young's moduli respectively
E_v^u, E_h^u	Undrained vertical and horizontal Young's moduli respectively
G_{hh}	Shear modulus in horizontal plane
G_{hv}, G_{vh}	Shear moduli in vertical plane
$G_{z\theta} (=G_{vh})$	Shear moduli in vertical plane
G_0	Elastic shear stiffness

1	HCA	Hollow Cylinder Apparatus
2	K_0	Coefficient of earth pressure at rest
3	KUC	K_0 -consolidated undrained compression
4	KUE	K_0 -consolidated undrained extension
5	LVDT	Linear variable differential transformer
6	$M^{\text{TXC}}, M^{\text{TXE}}$	Critical state stress ratio under triaxial compression and extension
7	OCR	Over-consolidation ratio
8	p'	Mean effective stress ($= (\sigma_1' + \sigma_2' + \sigma_3')/3$)
9	p_0'	Initial mean effective stress
10	p_{ref}	Reference pressure
11	PB	Probing test
12	PL, LL	Plastic limit, Liquid limit
13	q	Deviatoric stress ($= \sigma_1 - \sigma_3$)
14	RC	Resonant column test
15	REV	Representative element volume
16	s'	$(\sigma_1' + \sigma_3')/2$
17	S_r	Degree of saturation
18	S_u	Undrained shear strength
19	t	$(\sigma_1 - \sigma_3)/2$
20	YSR	Yield stress ratio
21	F_a, T, p_i, p_o	Axial force, torque, inner and outer cell pressures applied in HCA tests
22	TXC, TXE	Triaxial compression and extension respectively
23	OD, ID, H	Specimen outer diameter, inner diameter and height, respectively

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1 **TABLES**

Table 1 Overview of natural Cowden till index properties, critical state parameters and shear strengths

Depth range [m]	wc [%]*	PL [%]	LL [%]	G_s	ρ_{bulk} [g/cm ³]	1-D compression				Triaxial KUC/KUE			
						YSR	λ	κ	v_1	M^{TXC}	M^{TXE}	S_u^{TXC} [kPa]	S_u^{TXE} [kPa]
1.5-2.5	18.0	17.6	38.2	2.723	2.17	24.7	0.077	0.027	2.020	1.09	1.0	124-168	103-121
2.5-3.5	16.1	17.4	36.4	2.716	2.21	11.5	0.073	0.025	1.853	1.0	1.0	129-164	118
4.0-6.0	16.9	15.1	33.2	2.707	2.19	6.69	0.06	0.023	1.790	1.0	1.0	107-127	70-83
10-12	17.4	15.5	34.5	2.708	2.18	2.77	0.068	0.027	1.833	1.0	1.0	140-175	118

2 Note: water content for rotary core specimens only.

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Table 2 Triaxial programme for small-strain probing and ‘standard’ KUC/KUE tests on natural samples

Depth [m]	Sample type	Test ref.	OD [mm]	Test type	Initial p_0' [kPa]	S_r [%]	e_0	σ_{v0}' [kPa]	σ_{h0}' [kPa]
2.0	Block	PB1	100	BE, dv'(2), dh'(2), dq(1)	388.3	96.0	0.461	33.0	49.5
2.05	Rotary core	KUC1	100	BE, KUC	94	100	0.440	34.9	52.4
2.2	Rotary core	KUE1	38	KUE	235.5	100	0.481	35.8	53.7
3.0	Rotary core	KUC2	38	KUC	144.5	100	0.427	45.7	68.6
3.4	Rotary core	KUE2	38	KUE	88.3	100	0.460	49.4	74.1
5.1	Rotary core	PB2	100	BE, dv'(3), dh'(3), dq(1)	85.0	95.3	0.436	69.9	104.9
5.1	Rotary core	KUC3	100	BE, KUC	40.0	100	0.441	69.9	104.9
5.35	Rotary core	KUE3	38	KUE	116.7	100	0.454	73.1	109.7
11.5	Rotary core	KUC4	38	KUC	86.0	100	0.438	198.0	198.0
12.0	Rotary core	PB3	100	BE, dv'(2), dh'(2), dq(1)	49.2	100	0.421	223.8	223.8
12.5	Rotary core	PB4	100	BE, dv'(2), dh'(2), dq(1)	113.7	99.9	0.406	212.3	218.0

Notes:

1. Probe type: dv' – drained vertical (axial) stress probe; dh' – drained horizontal (radial) stress probe; dq – undrained vertical (axial) stress probe; values enclosed by the brackets denoting the number of probes performed;
2. Bender element measurements were made within specimens of 100×200 (D $\times$ H, in mm) dimensions, but not within the smaller 38×76 mm samples;
3. e_0 : pre-shearing or pre-probing void ratio.

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Table 3 Sample and test conditions for the undrained HCA programme

Depth [m]	Test Ref.	Type	Initial p_0' [kPa]	S_r [%]	e_0	σ_{v0}' [kPa]	σ_{h0}' [kPa]	α_f [°]	b_0	b_u
2.93	CA0005	Constant $\alpha_{d\sigma}$	254.8	88.5	0.461	38.7	58.1	0	0.5	0.5
2.93	CA2305	Constant $\alpha_{d\sigma}$	265.5	93.7	0.428	38.7	58.1	23	0.5	0.5
2.87	CA6705	Constant $\alpha_{d\sigma}$	278.0	85.2	0.463	38.0	56.9	67	0.5	0.5
2.93	CA9005	Constant $\alpha_{d\sigma}$	248.2	88.4	0.463	38.7	58.1	90	0.5	0.5
2.93	CASS	HCA SS	188.8	88.5	0.460	38.7	58.1	48.2	1.0	0.47
2.87	CATC1	HCA TXC	261.2	85.1	0.471	38.0	56.9	0	1.0	0
2.87	CATC2	HCA TXC	265.5	87.5	0.441	38.0	56.9	0	1.0	0
0.50	CTSS	HCA SS	185.0	90.3	0.573	14.5	21.8	46.7	1.0	0.49

Notes:

1. All HCA specimens trimmed and drilled from intact soil blocks to similar dimensions (ID: 38±0.5 mm; OD: 71±0.5 mm; H: 180-195 mm).
2. α_f denotes the angle between the vertical and major principal stress direction at failure (ultimate state);
3. b_0 and b_u designate the intermediate stress factor prior to undrained shearing and at ultimate state respectively.

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Table 4 Creep strain criteria and shearing rates adopted in the experiments

Type	Strain rates [%/day]		
	Creep ($\leq$)	Shearing	Ratio
Triaxial 'standard' KUC/KUE shearing	0.005-0.01	5	500-1000
Triaxial drained & undrained probing	0.0002	0.007-0.014	35-70
HCA undrained shearing	0.012	3.6	300

Table 5 Gravel contents of Cowden till specimens in different preparation stages

Group	Sample status	No. of specimens	Specimen dry mass [g]	Gravel content [%]
A	Heavily fissured and weathered stony soil blocks; no scope for forming testable specimens; Geobor-S sampling impossible or very low recovery rate	2	2093.5~2667	8.5~12.2
B	Successfully rotary cored and recovered specimens using a Geobor-S sampler; representing the status of the 100 mm diameter, 200 mm height solid triaxial specimens; grading curves see Figure 2	5	$\approx$ 2000	6.3~8.2
C	Soil prisms and putative HCA specimen cylinders with attempts in manual trimming or mechanical drilling but failed; gravel fragments markedly smaller than those from Groups A and B	9	1015.5~2533.5	3.3~6.6
D	Successfully shaped, drilled and tested HCA specimens with minimum gravel particle sizes	4	891.5~1104.5	2.8~4.1

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Table 6 Summary of the triaxial probing test results

Test ref. (depth)	Stresses [kPa]		Drained axial (dv')				Drained radial (dh')				Undrained axial (dq)		
	σ_{v0}'	σ_{h0}'	Type	$\Delta\sigma_v'$	E_v' [MPa]	ν_{vh}'	Type	$\Delta\sigma_h'$	a [MPa ⁻¹]	F_h' [MPa]	Type	Δq	E_v^u [MPa]
PB1 (2.0 m)	33.0	49.5	+	1	71.5	0.23	+	1.4	0.0028	54.1			115.1
			-	1	90.9	0.33	-	1.4	0.0074	54.1			
			Average		81.2	0.28			0.0051	54.1			115.1
PB2 (5.1 m)	69.9	104.9	±	0.7	120.2	-	±	0.7	0.0039	102.8	±	1.4	132.1
			-	1.3	124.4		+	2.2	0.0027	92.8			
			+	2.2	111.6	0.36	-	2	0.0035	110.5			
			Average		120.0	0.36			0.0034	102.0			132.1
PB3 (12.0 m)	223.8	223.8	+	1.5	200.0	0.31	+	1.1	0.00083	153.3			229.0
			-	1.5	222.2	0.37	-	1.4	0.0013	133.3			
			Average		211.1	0.34			0.0013	143.3			229.0
PB4 (12.5 m)	212.3	218	±	1.8	204.5	1.09*	±	1.8	0.0018	274.3	±	1.8	212.0
			+	2.7	219.1	1.06*	±	1.8	0.0021	344.0			
			Average		211.8	1.08			0.0020	309.2			212.0

Notes:

1. $a = \nu_{hv}'/E_h' = -1/2 \times \Delta\varepsilon_v/\Delta\sigma_h'$; $F_h' = E_h'/(1 - \nu_{hh}') = \Delta\sigma_h'/\Delta\varepsilon_h$;

2. Poisson's ratios superscripted with "*" might be affected by residual radial straining.

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Table 7 Summary of the measured and derived cross-anisotropy stiffness parameters and Poisson's ratios

Test ref. (depth)	G_{vh} [MPa]	G_{hh} [MPa]	E_v' [MPa]	E_h' [MPa]	ν_{hh}'	ν_{hv}'	ν_{vh}'	E_v^u [MPa]	E_h^u [MPa]	ν_{hh}^u	ν_{hv}^u	Method
PB1 (2.0 m)	<u>131.9</u>	<u>153.7</u>	<u>81.2</u>	92.0	-0.70	0.47	0.41	82.2	214.3	-0.30	1.30	Kuwano & Jardine (2002)
	<u>131.9</u>	<u>153.7</u>	<u>81.2</u>	252.8	-0.18	1.29	0.41	<u>115.1</u>	263.3	-0.14	1.14	Nishimura (2014a, 2014b)
HCA (2.9 m)	<u>53.8</u>	70.6						85.1	152.9	0.111	0.89	Brosse <i>et al.</i> (2016)
PB2 (5.1 m)	<u>125</u>	<u>161</u>	<u>120.2</u>	155.0	-0.52	0.52	0.40	122.7	278.5	-0.14	1.14	Kuwano & Jardine (2002)
	<u>125</u>	<u>161</u>	<u>120.2</u>	271.7	-0.16	0.91	0.40	<u>132.1</u>	290.3	-0.10	1.10	Nishimura (2014a, 2014b)
PB3 (12.0 m)	<u>121.2</u>	<u>117.0</u>	<u>211.1</u>	177.8	-0.24	0.23	0.27	227.3	309.0	0.32	0.70	Kuwano & Jardine (2002)
	<u>121.2</u>	<u>117.0</u>	<u>211.1</u>	187.4	-0.20	0.24	0.27	<u>229.0</u>	309.7	0.32	0.67	Nishimura (2014a, 2014b)
PB4 (12.5 m)		<u>119</u>	<u>204.5</u>	254.9	0.07	0.46	0.37	219.1	308.5	0.30	0.70	Kuwano & Jardine (2002)
		<u>119</u>	<u>204.5</u>	199.5	-0.16	0.36	0.37	<u>212</u>	304.9	0.28	0.72	Nishimura (2014a, 2014b)

Notes:

- Underlined numbers represent variables from direct measurements; the others were from indirect derivation;
- Poisson's ratios: $\nu_{hh}' = E_h' / (2G_{hh}) - 1$; $\nu_{hv}' = -E_h' / 2 \times \Delta \varepsilon_v / \Delta \sigma_h'$ (in drained horizontal probing); $\nu_{vh}' = \nu_{hv}' \times E_v' / E_h'$; Other approaches may also be applied to derive these ratios, see Gasparre *et al.* (2007) and Nishimura (2014a).

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1 **FIGURES**

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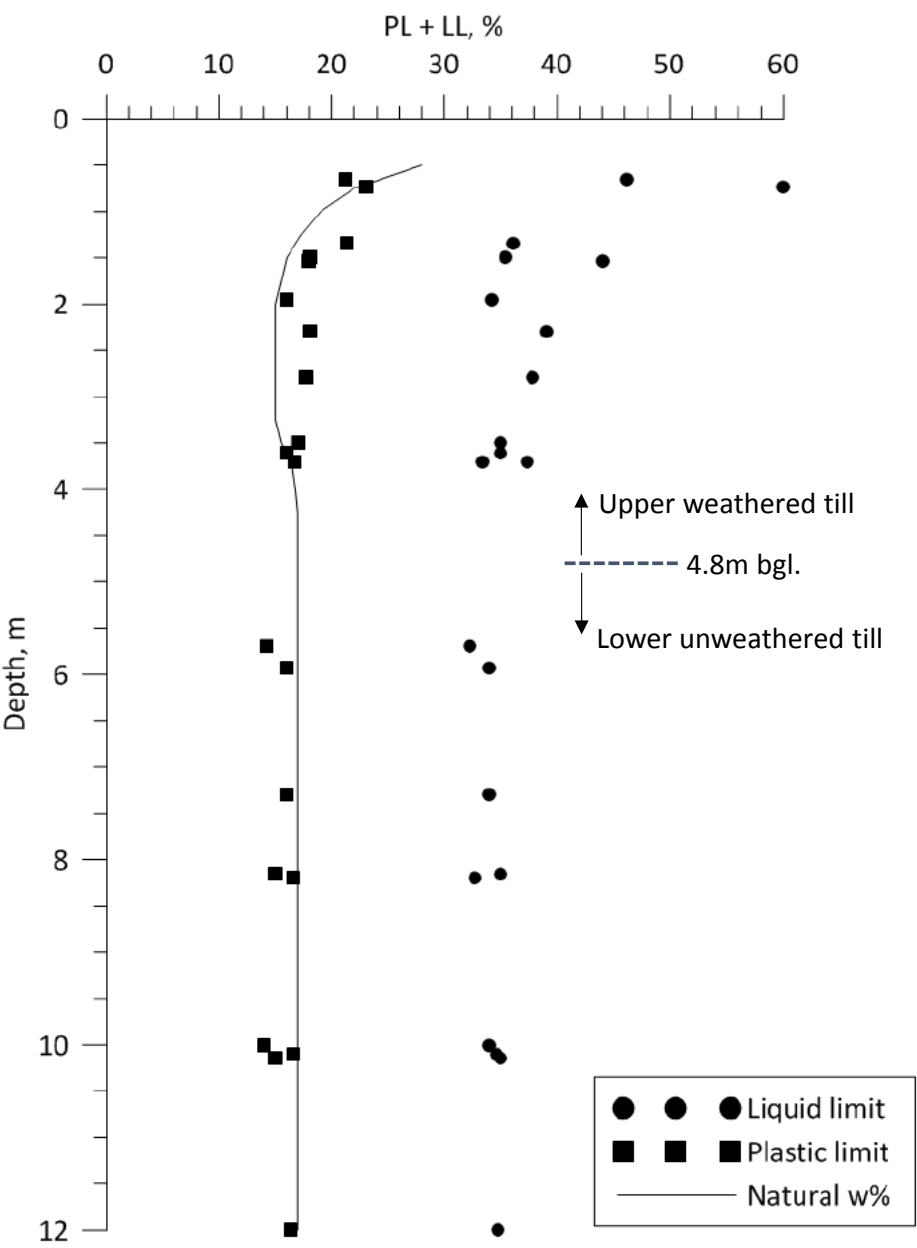


Figure 1 Index property profiles of Cowden till site (Ushev & Jardine, 2020)

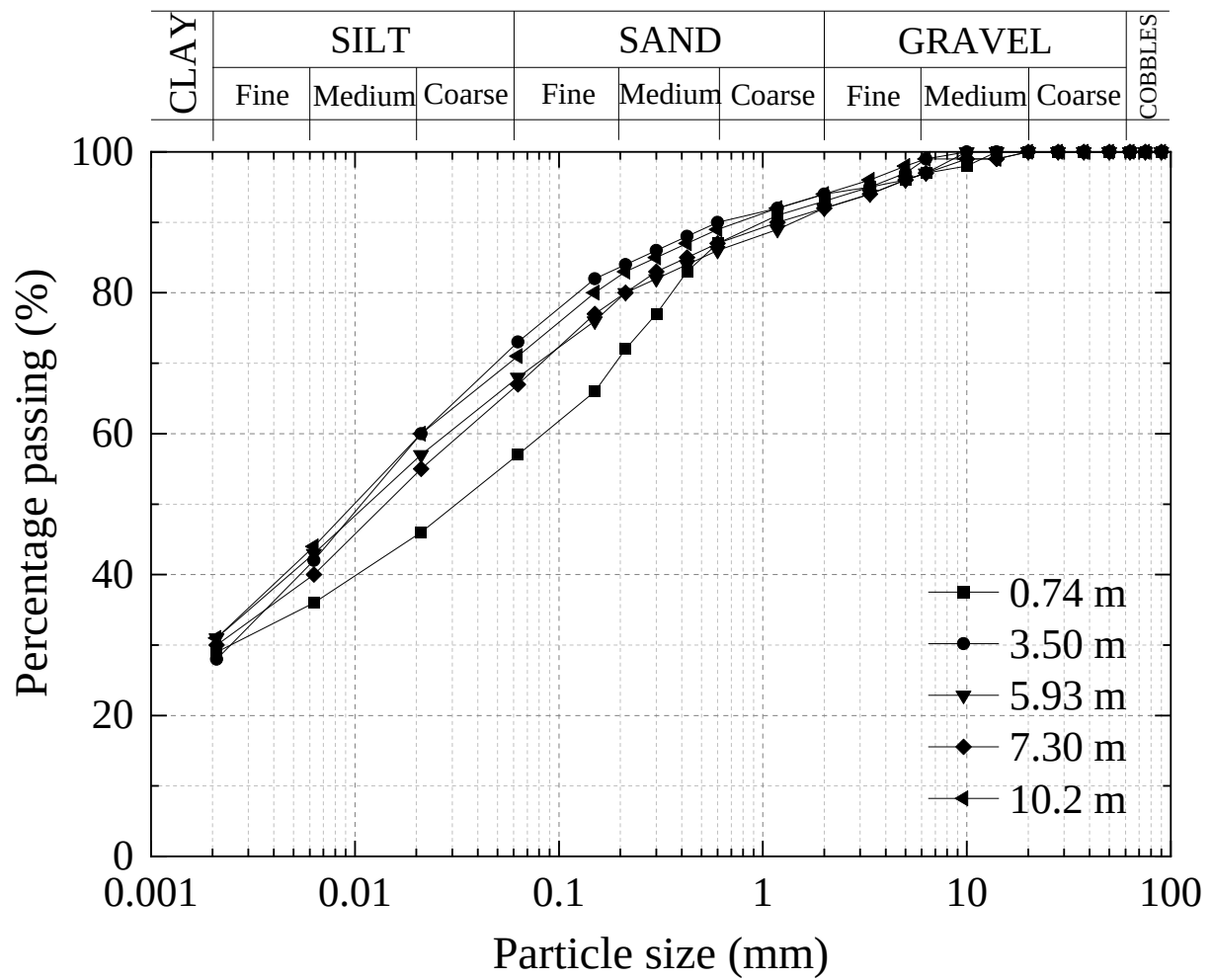


Figure 2 Grading curves of intact rotary cored Cowden till specimens from five depths

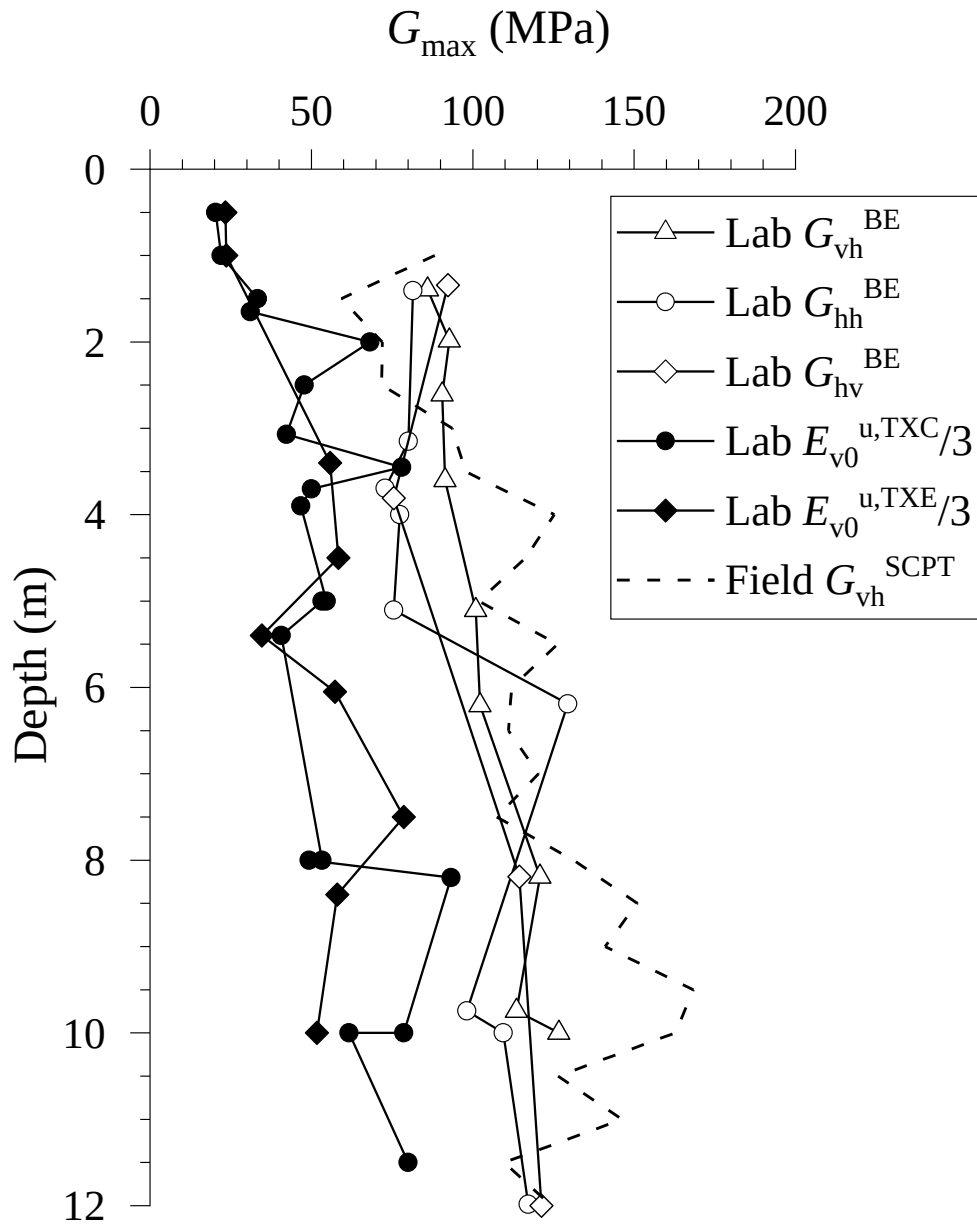


Figure 3 Maximum shear stiffness (G_0) profiles of Cowden till site determined from in-situ and laboratory approaches (Modified from Ushev (2018))

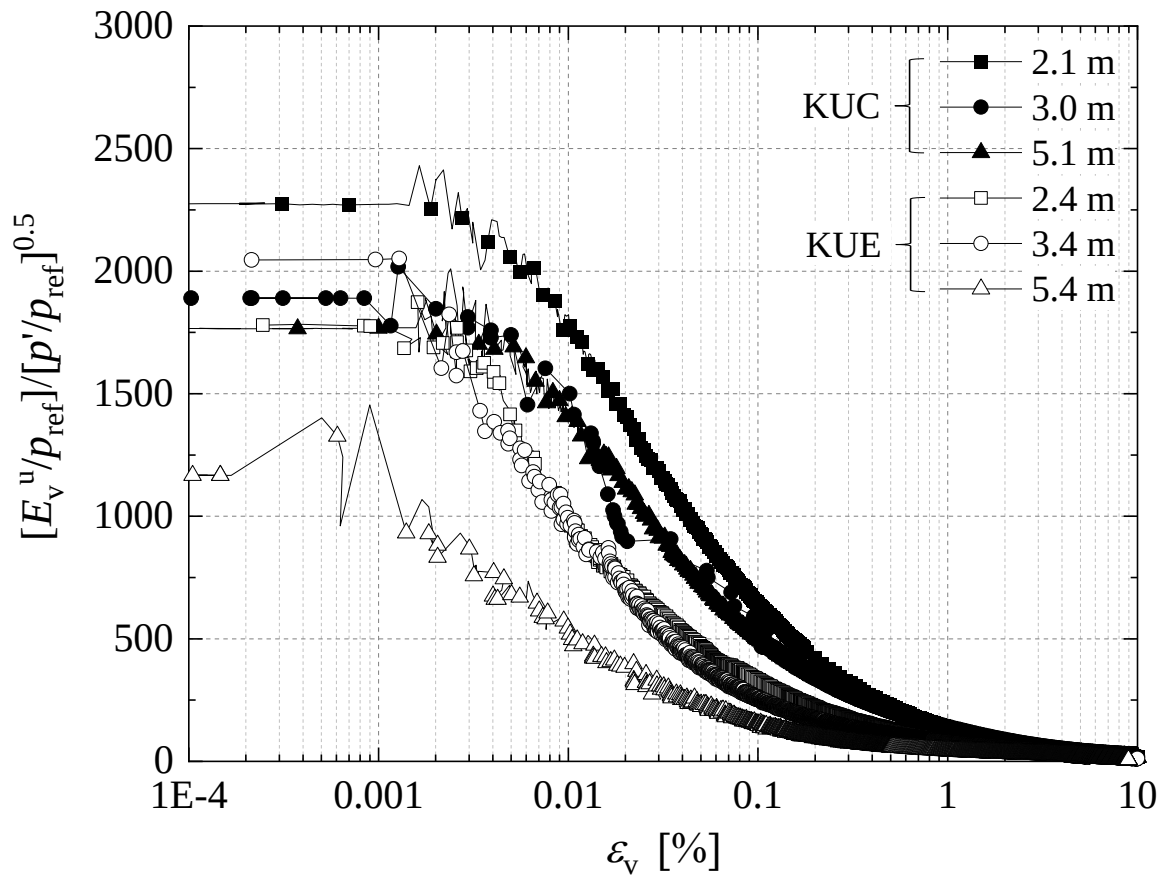


Figure 4 Degradation trends for normalised vertical undrained Young's moduli established from triaxial compression and extension tests on intact Cowden till specimens; after Ushev (2018)

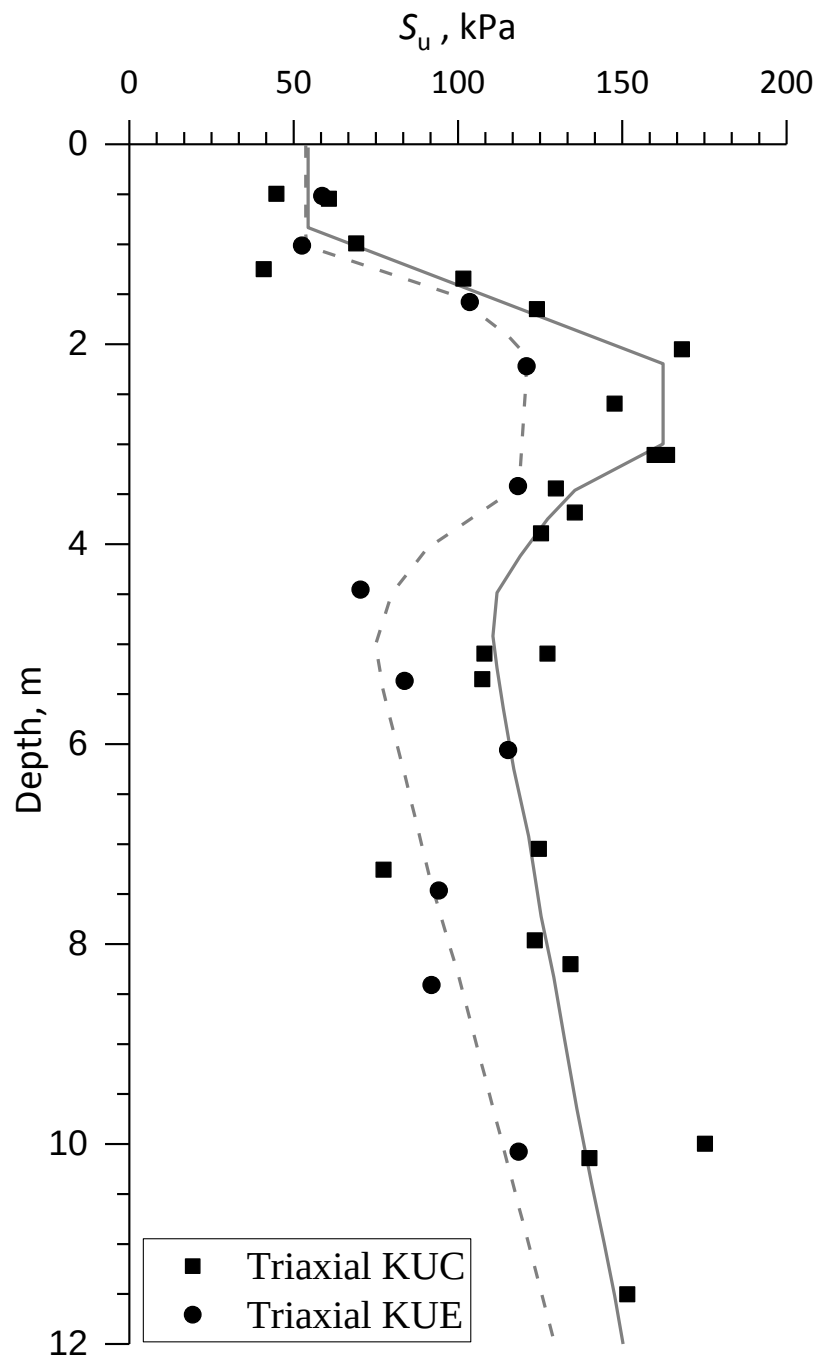


Figure 5 Undrained shear strength profiles of the Cowden till site established from anisotropically consolidated triaxial compression (KUC) and extension (KUE) tests (After Ushev & Jardine, 2020)

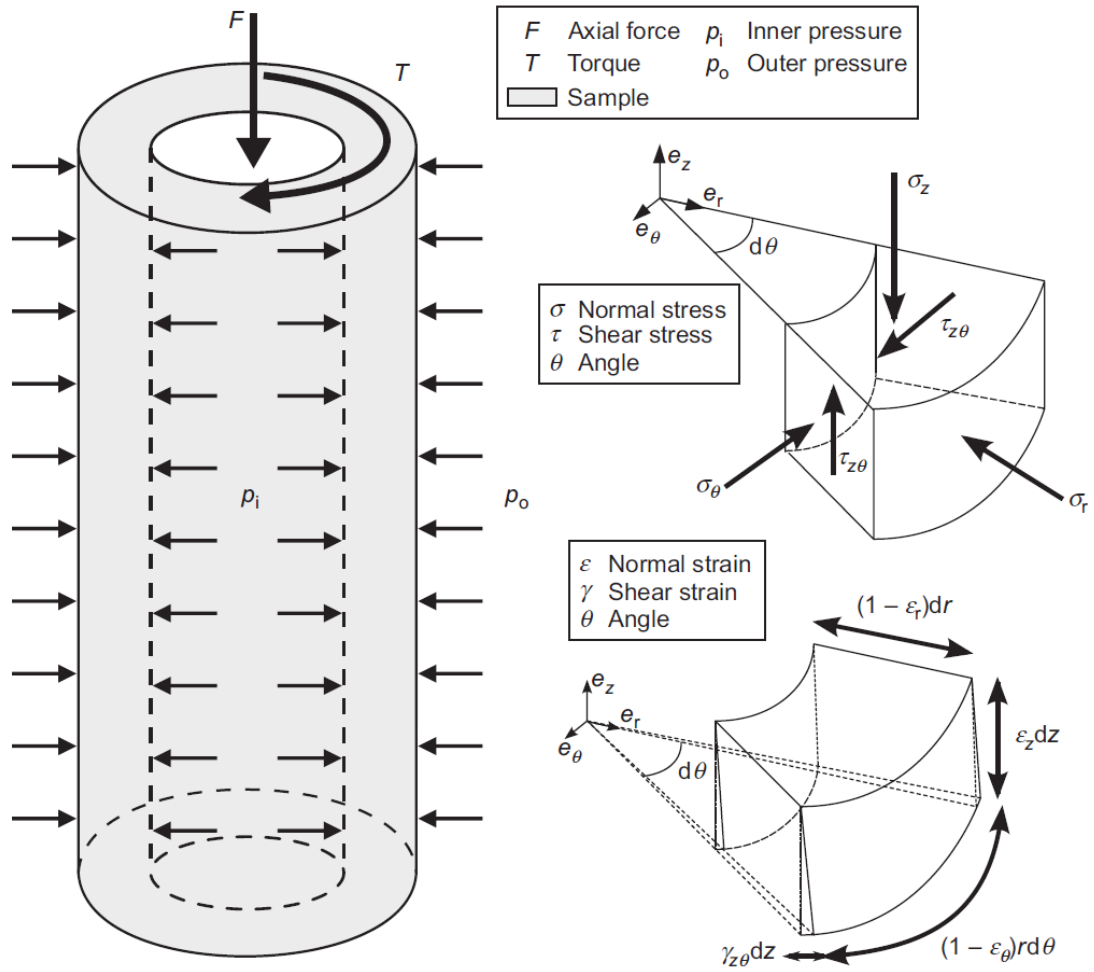


Figure 6 Stress and strain states imposed in a hollow cylinder sample (Brosse *et al.*, 2017)

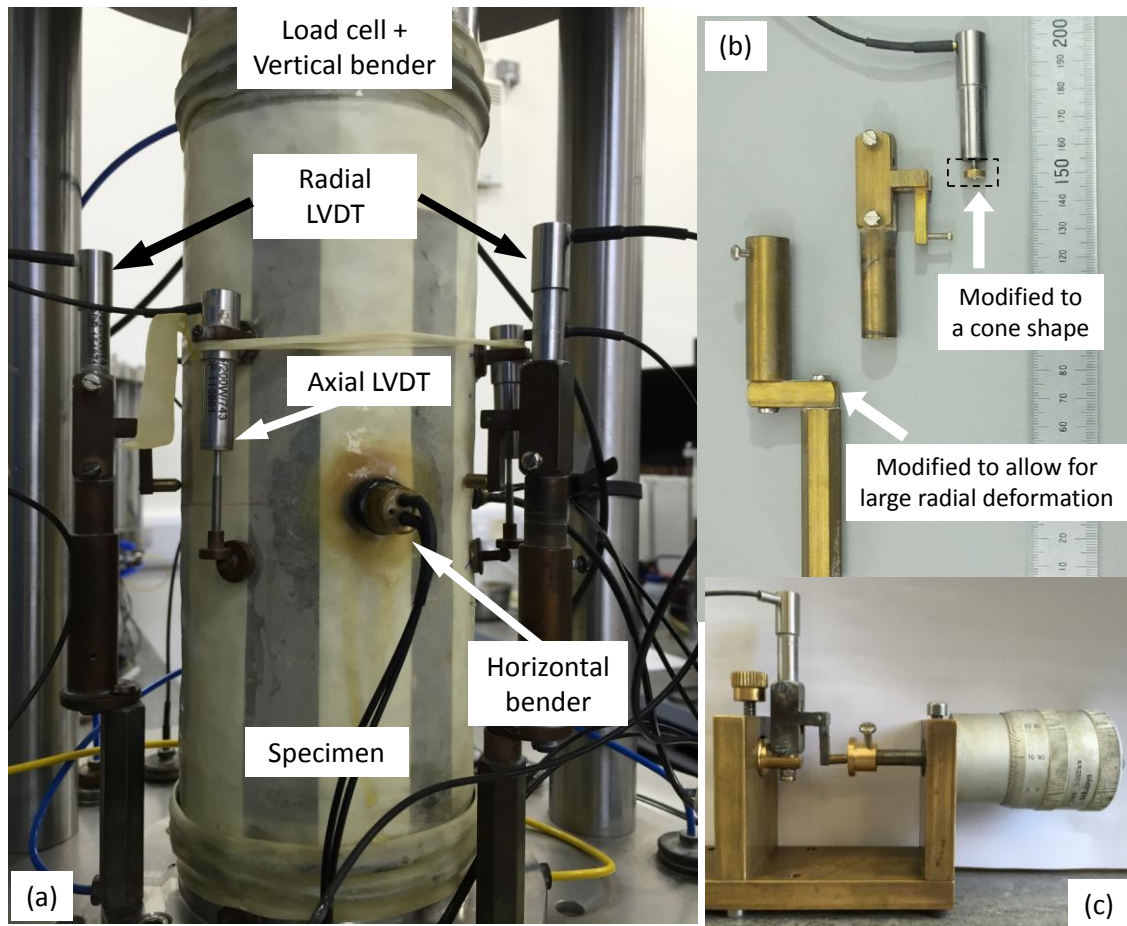


Figure 7 (a) Fully instrumented triaxial probing specimen; (b-c) configuration of the radial strain system developed by Ackerley *et al.* (2016) and calibration rig, also indicating relevant modifications implemented in this study

Constant- α_{ds} stage: Incremental control with fixed p , b ($= 0.5$) and α_{ds} starting from $\alpha = 90^\circ$

$$\sigma_z = p_0 - \frac{1}{2} q_0 + \zeta \cos(2\alpha_{ds})$$

$$\sigma_r = p_0$$

$$\sigma_\theta = p_0 + \frac{1}{2} q_0 - \zeta \cos(2\alpha_{ds})$$

$$\tau_{z\theta} = \zeta \sin(2\alpha_{ds})$$

b -change stage:
Incremental control
with fixed p , q and α ($= 90^\circ$)

$$\sigma_z = p_0 - \frac{1}{3} q_0 (b + 1)$$

$$\sigma_r = p_0 + \frac{2}{3} q_0 \left(b - \frac{1}{2}\right)$$

$$\sigma_\theta = p_0 - \frac{1}{3} q_0 (b - 2)$$

$$\tau_{z\theta} = 0$$

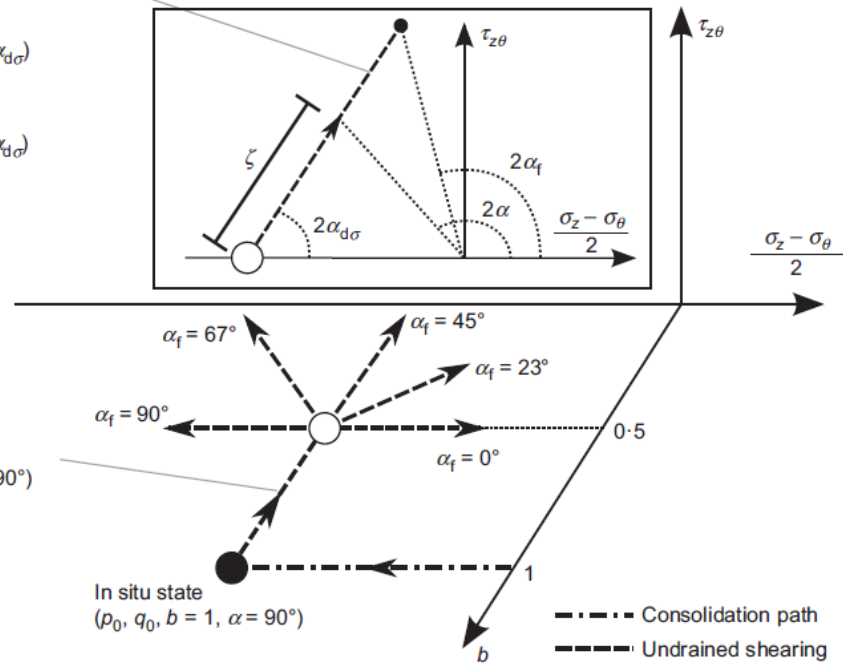


Figure 8 Undrained stress paths followed in the b -change and constant- α_{ds} stages from in-situ stresses in the HCA tests (adopted from Brosse *et al.* (2017))

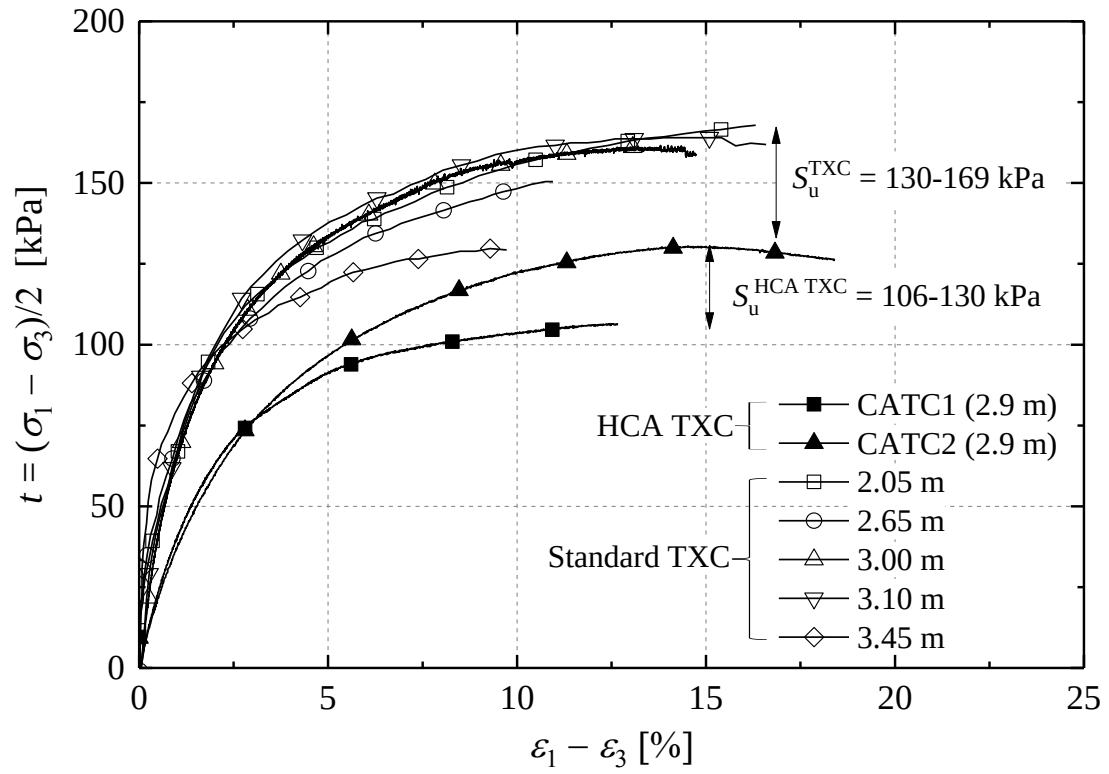


Figure 9 Comparison of shear strengths characterised from the HCA 'triaxial' tests (HCA TXC) and standard triaxial compression (TXC) tests

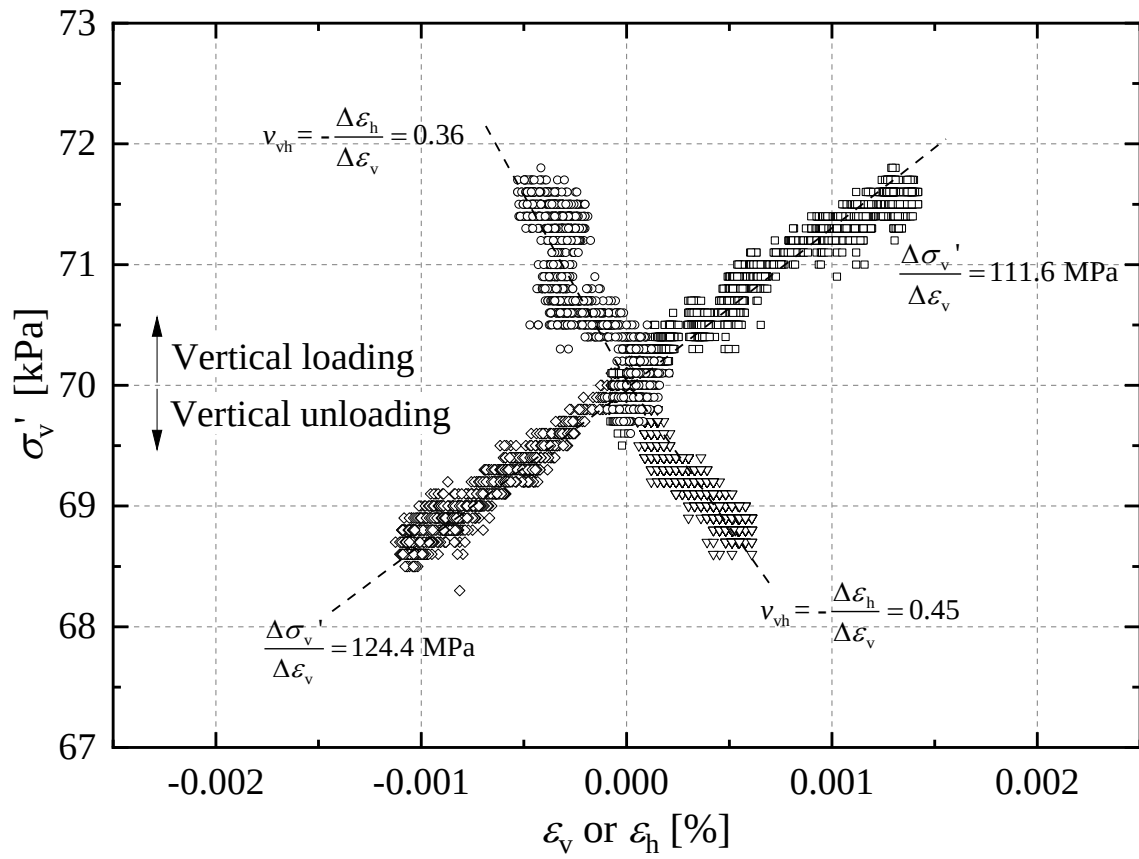


Figure 10 Stress-strain increments in the drained vertical loading (+dv') and unloading (-dv') probes in test PB2

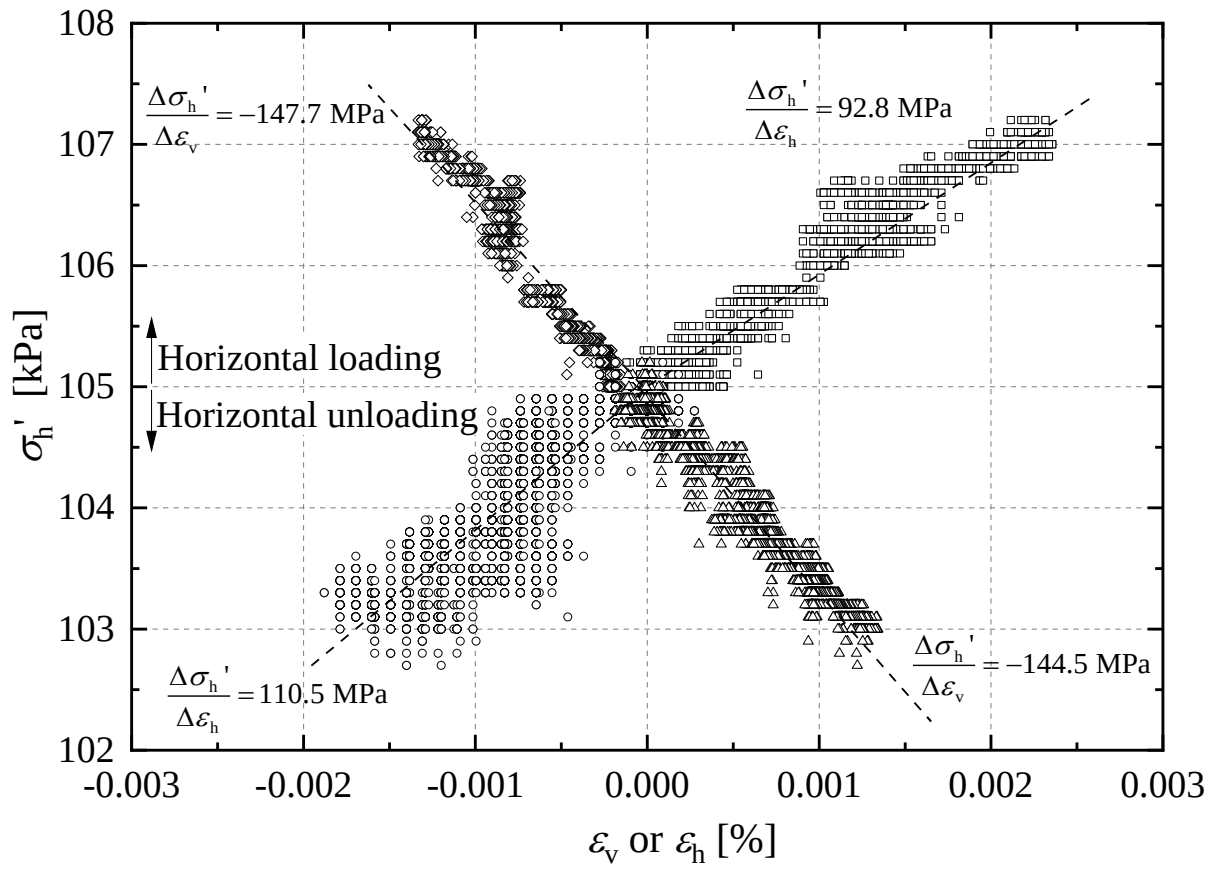


Figure 11 Stress-strain increments in the drained horizontal loading (+dh') and unloading (-dh') probes in test PB2

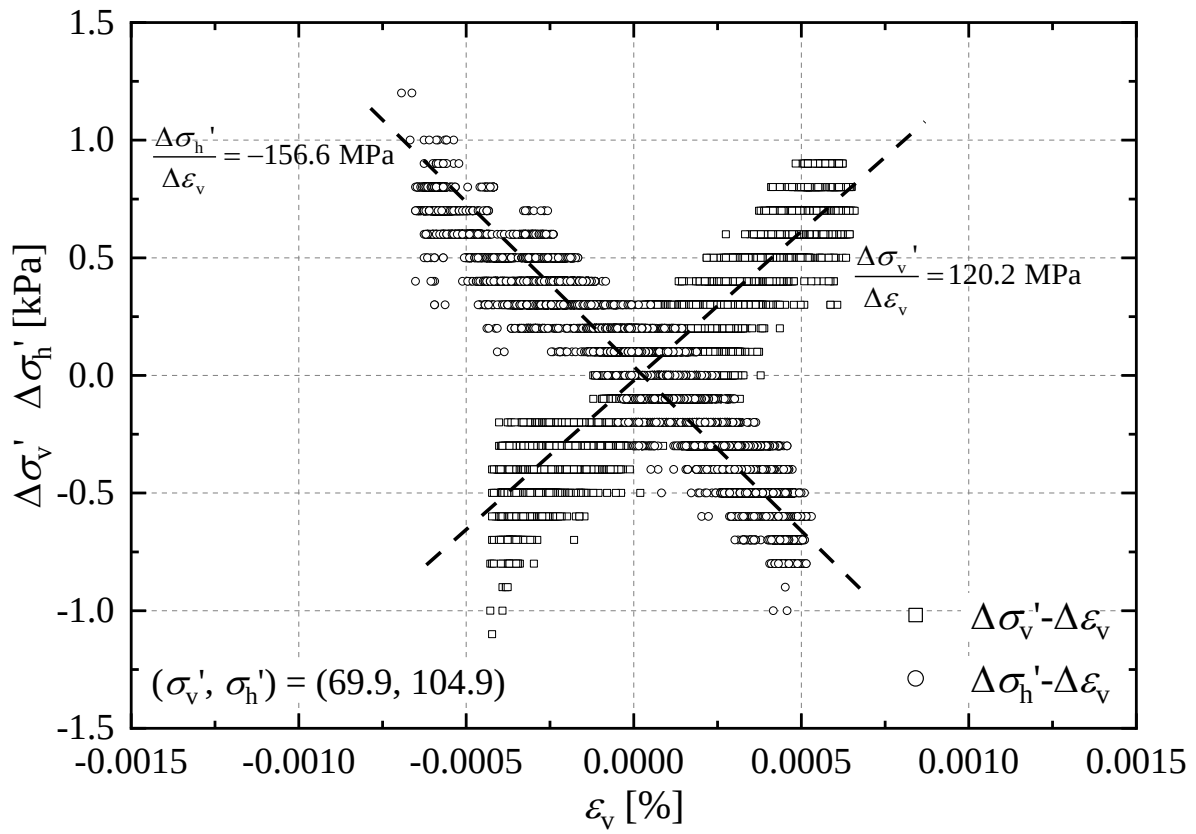


Figure 12 Stress increment-axial strain responses in drained vertical ($\pm dv'$) and horizontal ($\pm dh'$) stress probes in test PB2

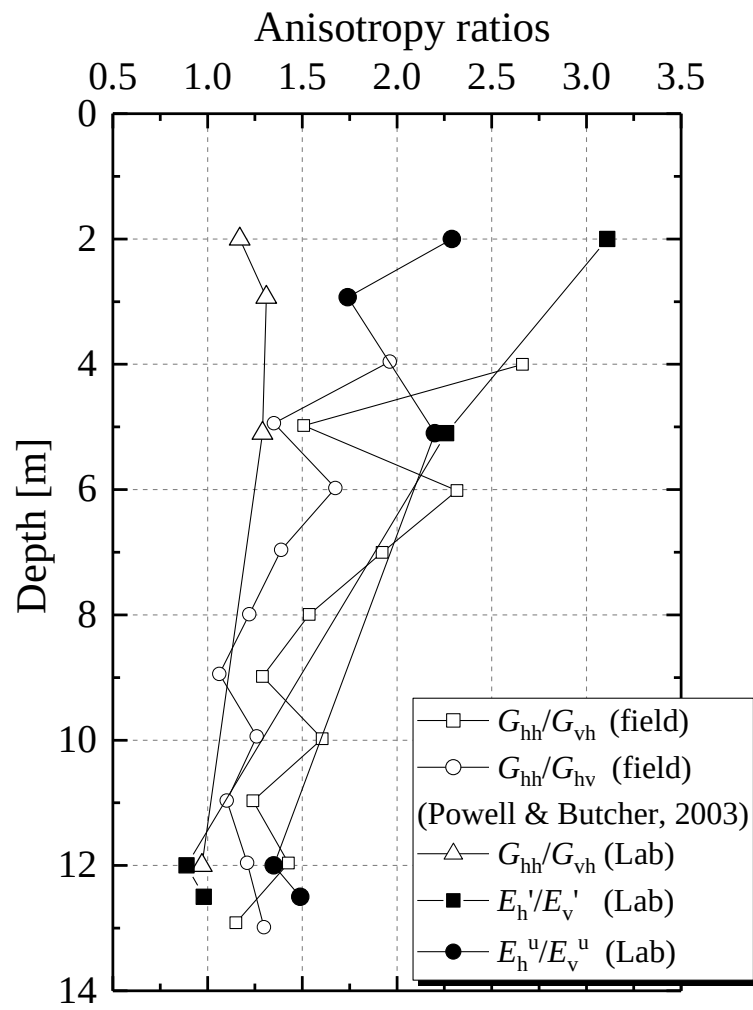


Figure 13 Anisotropic stiffness profile of the Cowden till site

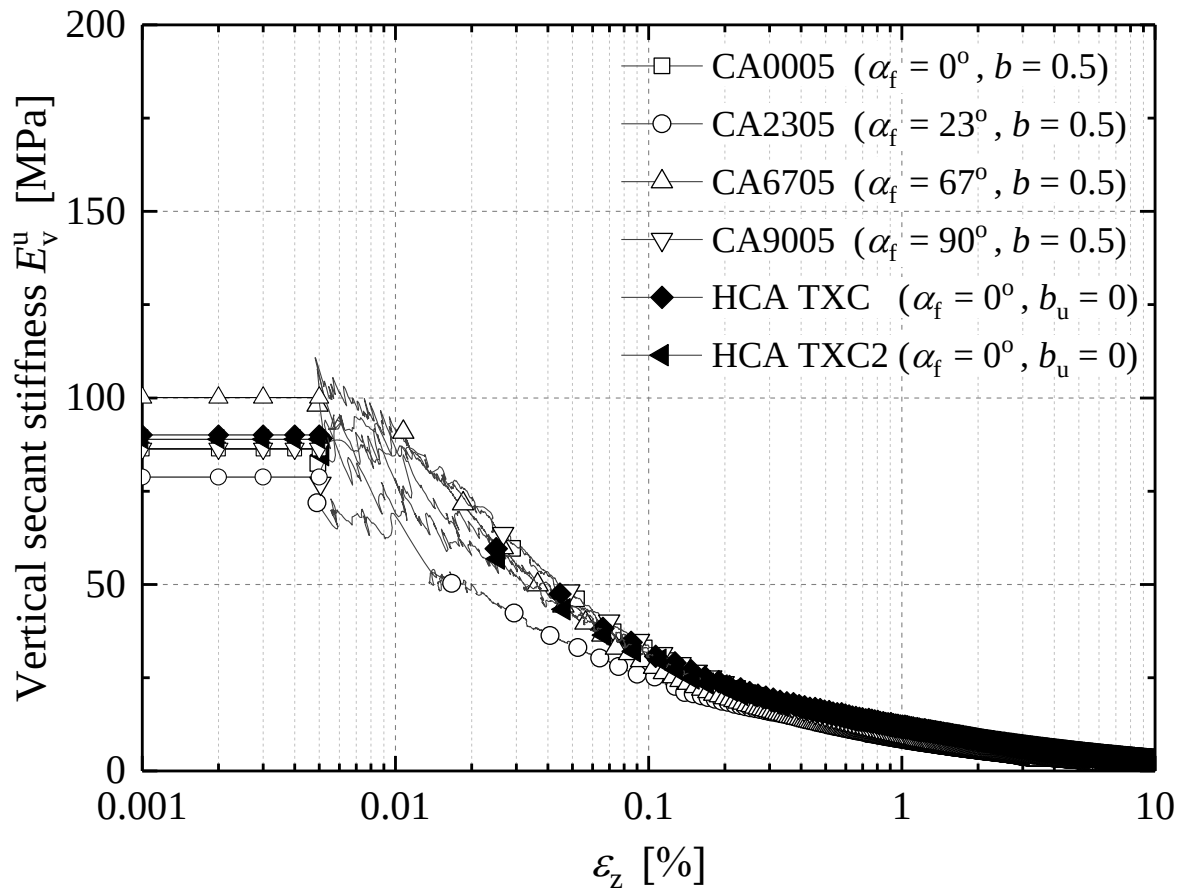


Figure 14 Degradation of undrained vertical Young's Moduli E_v^u against vertical (axial) strain

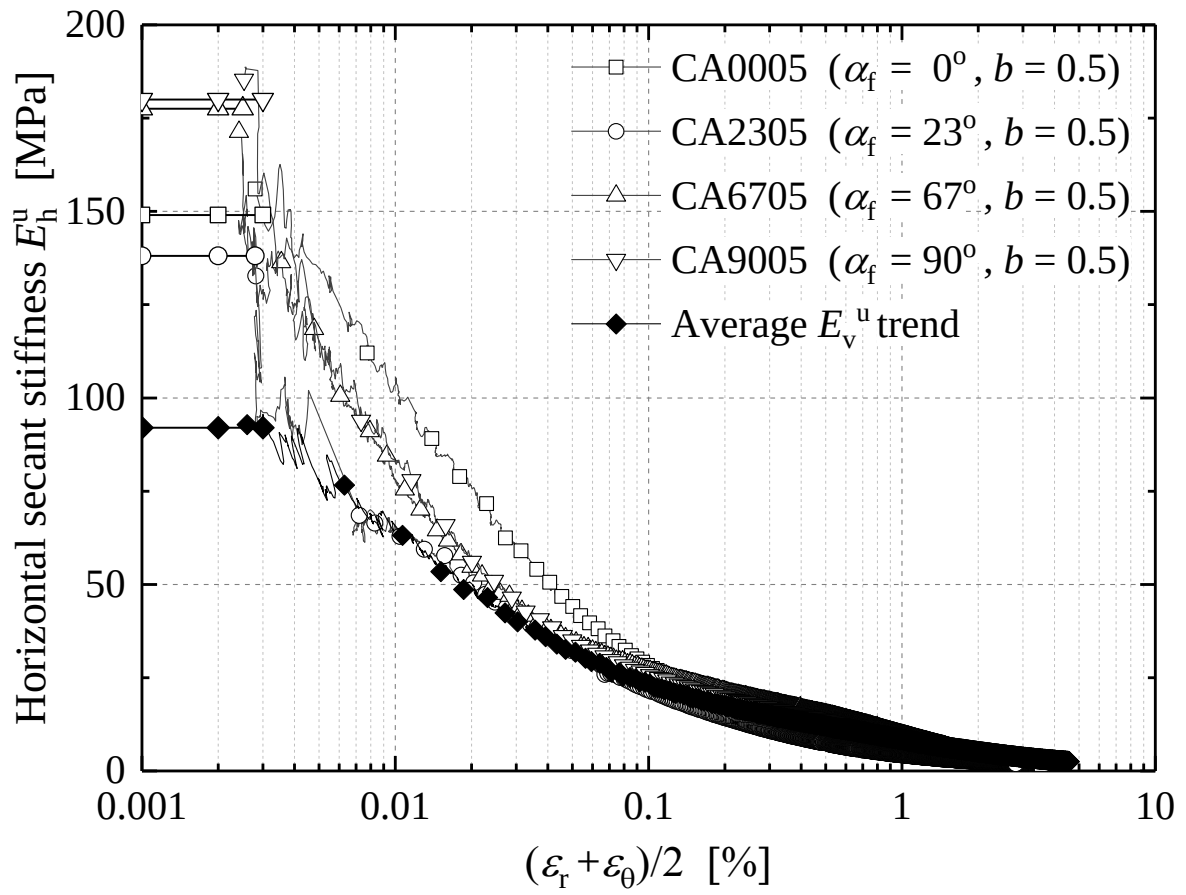


Figure 15 Degradation of undrained horizontal Young's Moduli E_h^u against horizontal strain

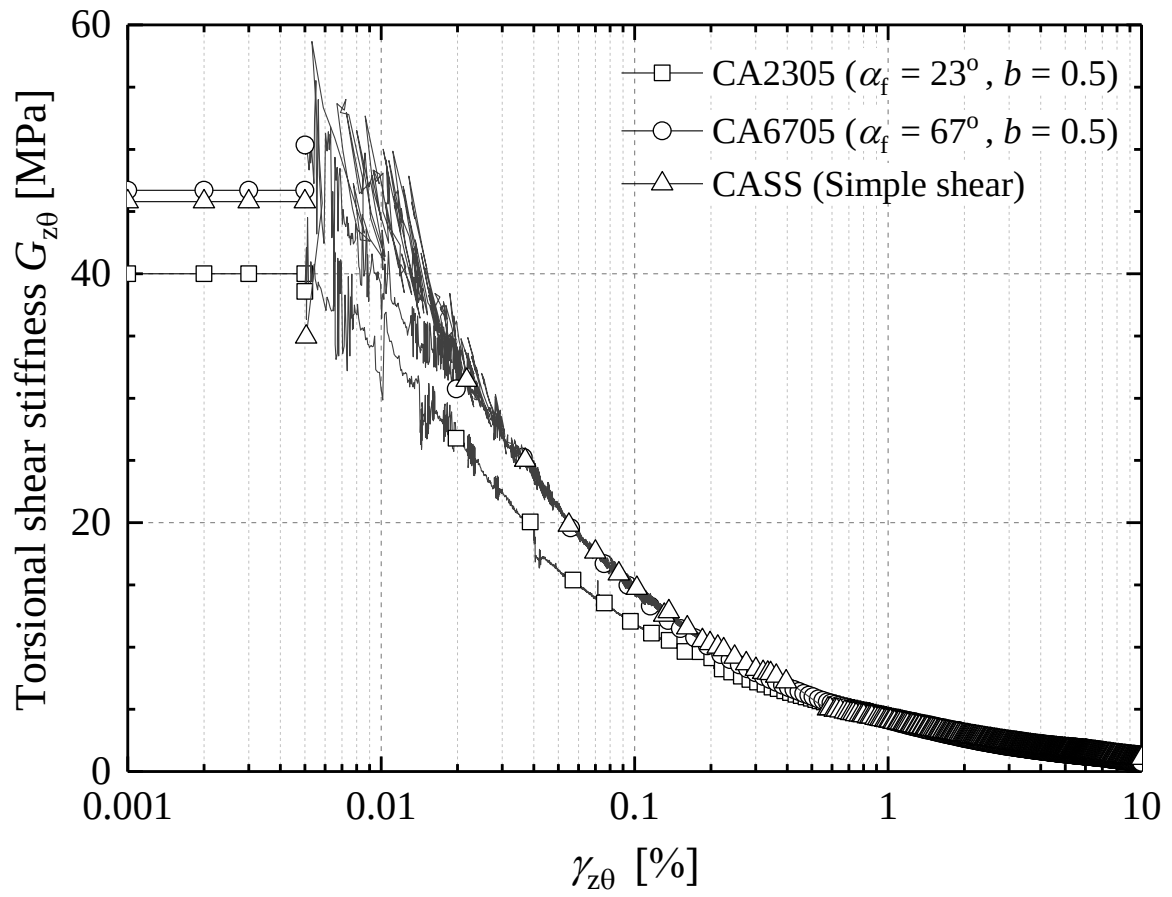


Figure 16 Degradation of torsional shear stiffness $G_{z\theta}$ against torsional shear strain

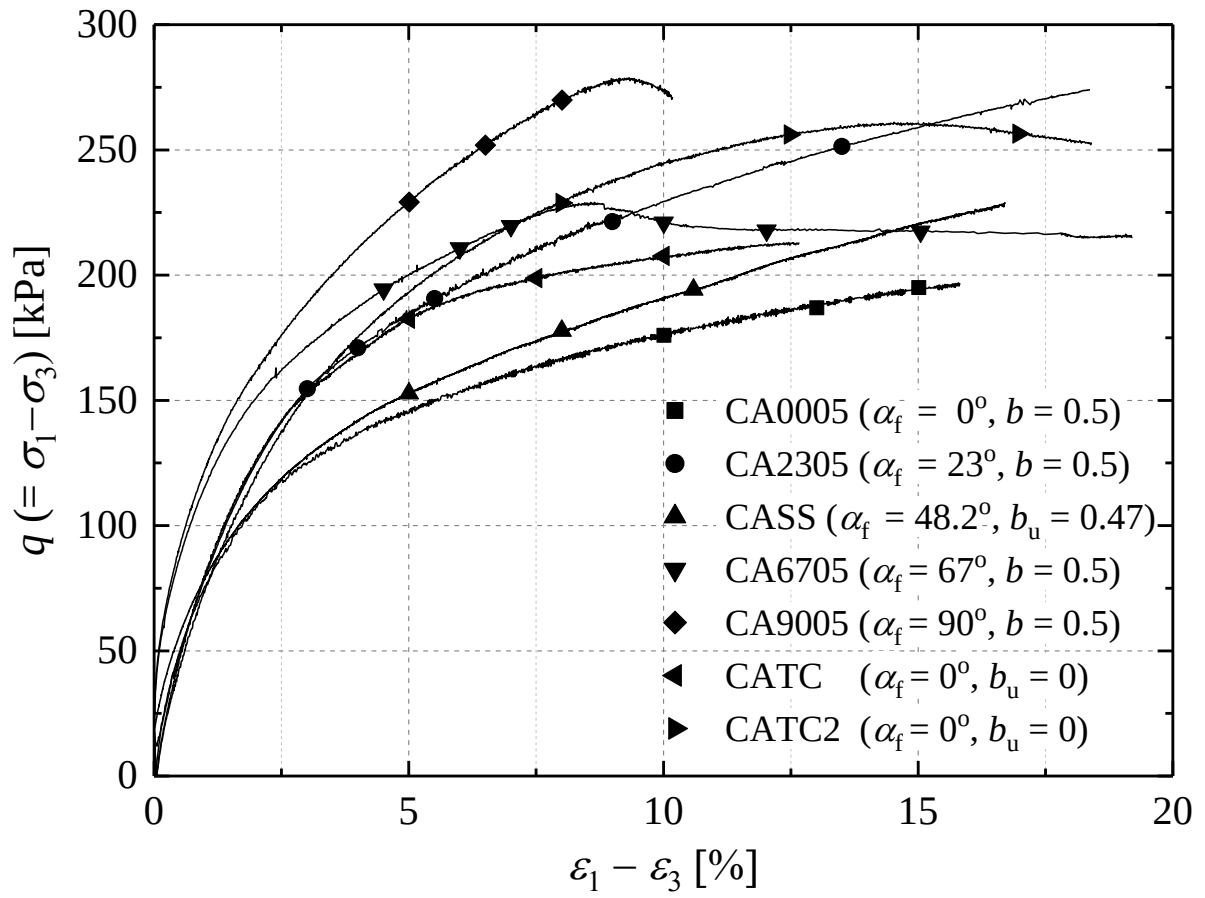


Figure 17 Deviatoric shear stress-strain trends in the undrained HCA shearing tests

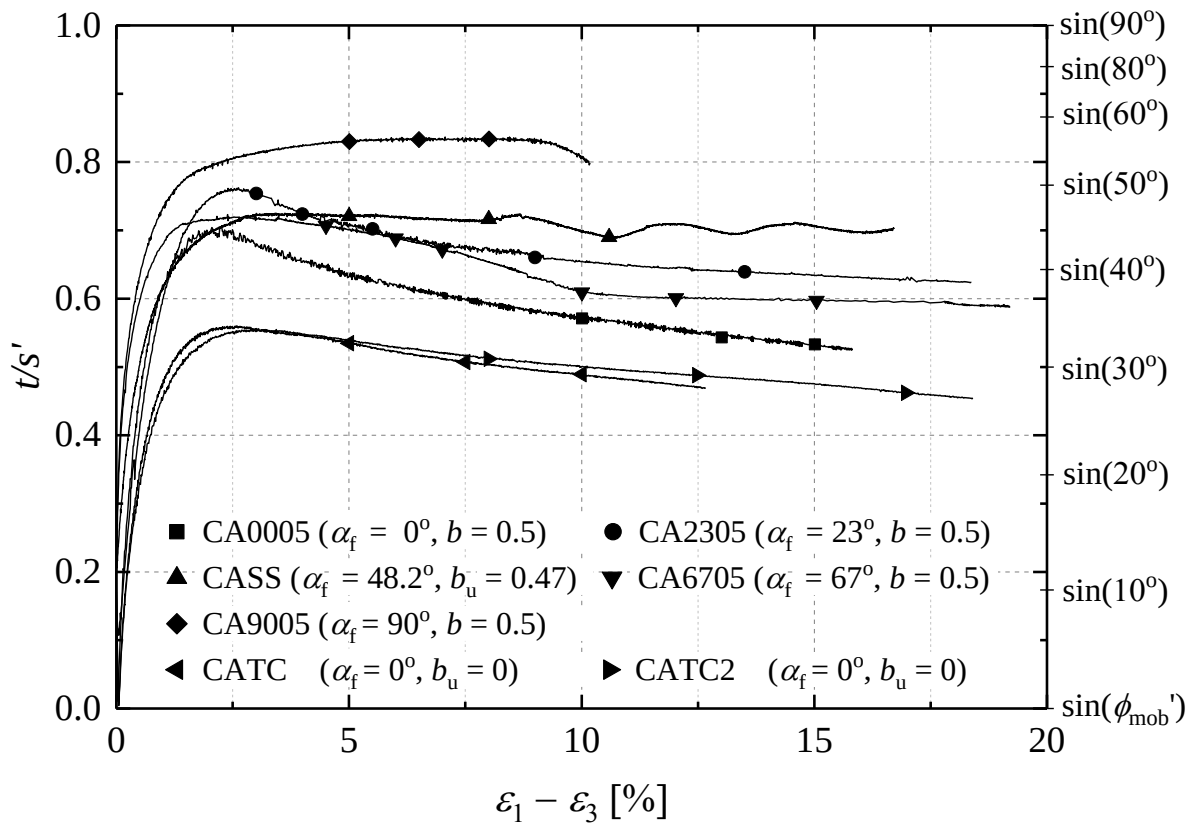


Figure 18 Stress ratio-strain trends in the undrained HCA shearing tests

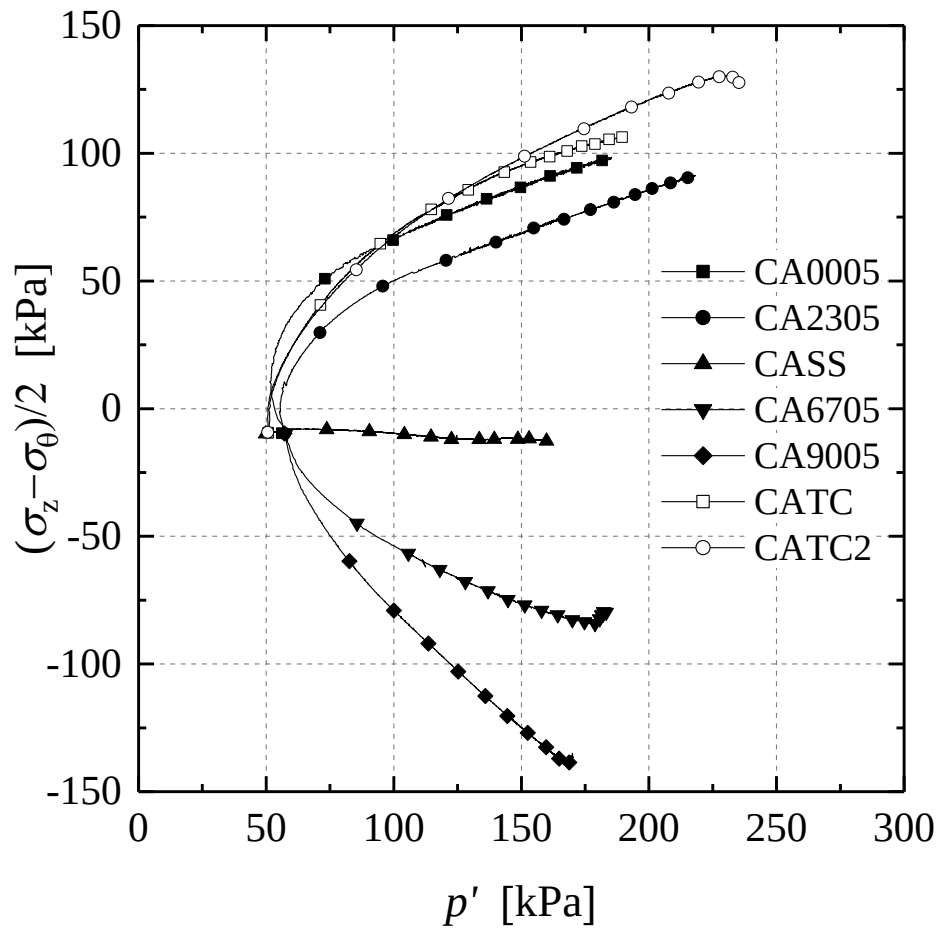


Figure 19 Projection of effective stress paths on the $[(\sigma_z - \sigma_\theta)/2] - p'$ plane

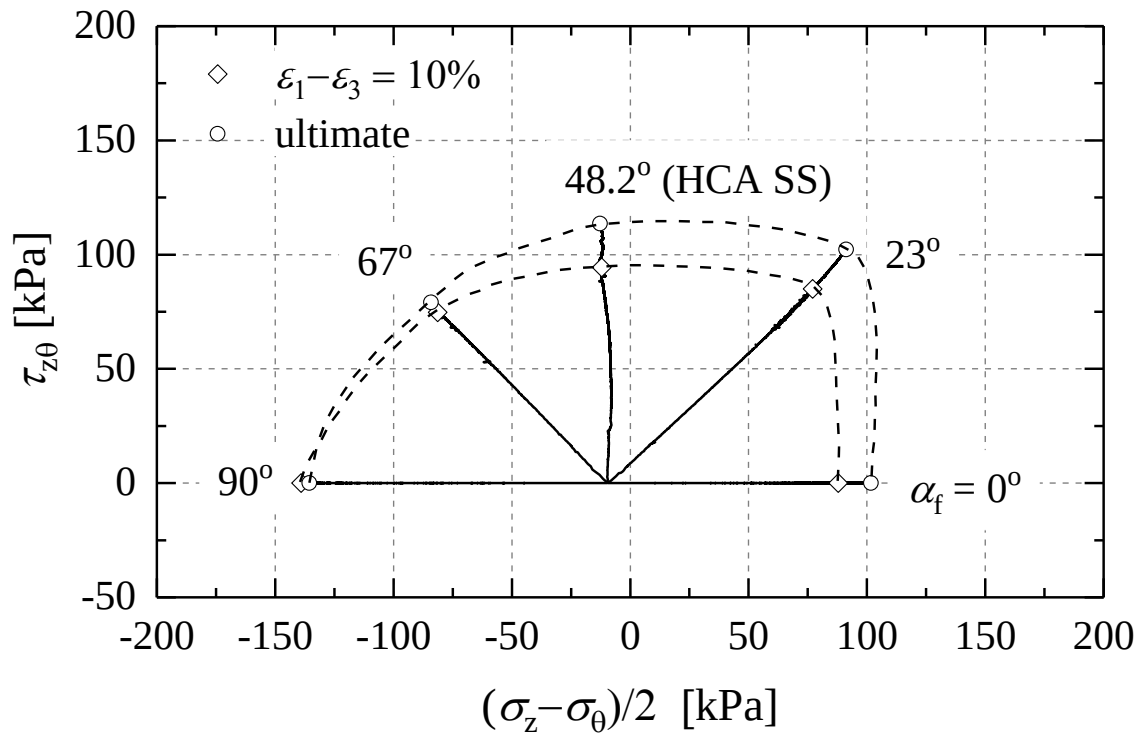


Figure 20 Projection of the stress paths on the $\tau_{z\theta}$ - $[(\sigma_z - \sigma_\theta)/2]$ plane

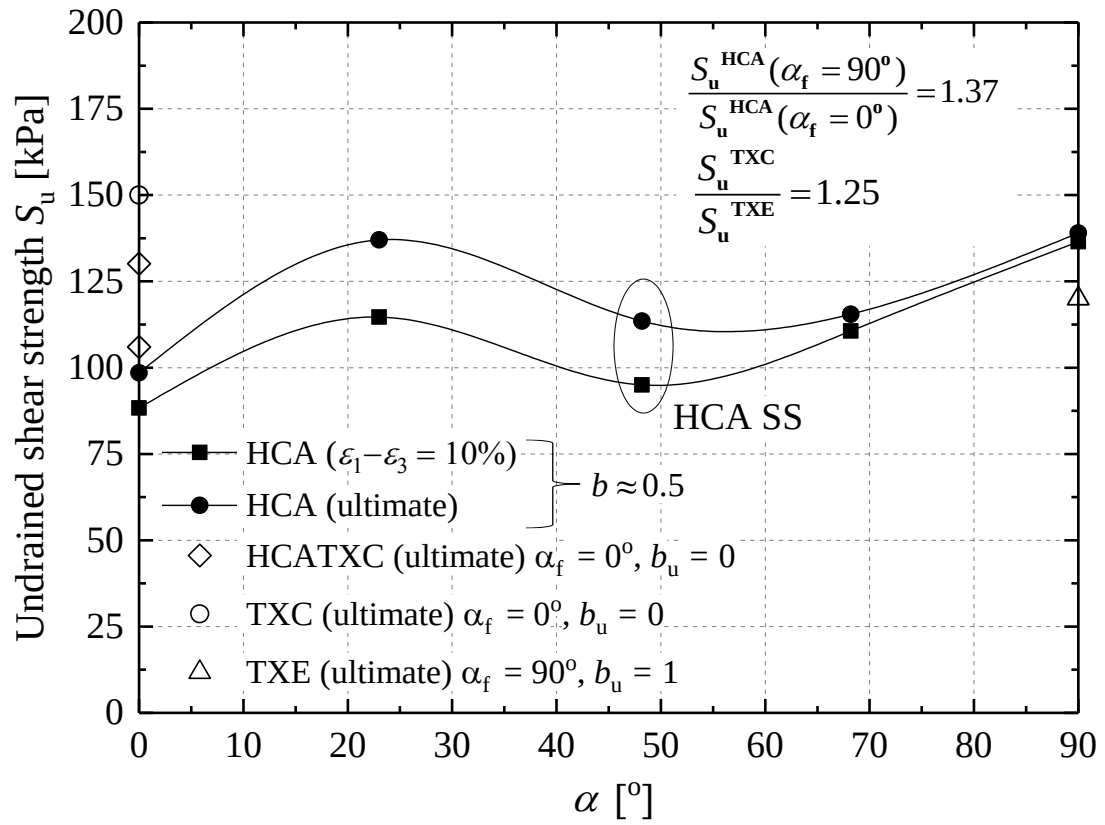


Figure 21 Variations of undrained shear strength S_u with the orientation of major principal stress

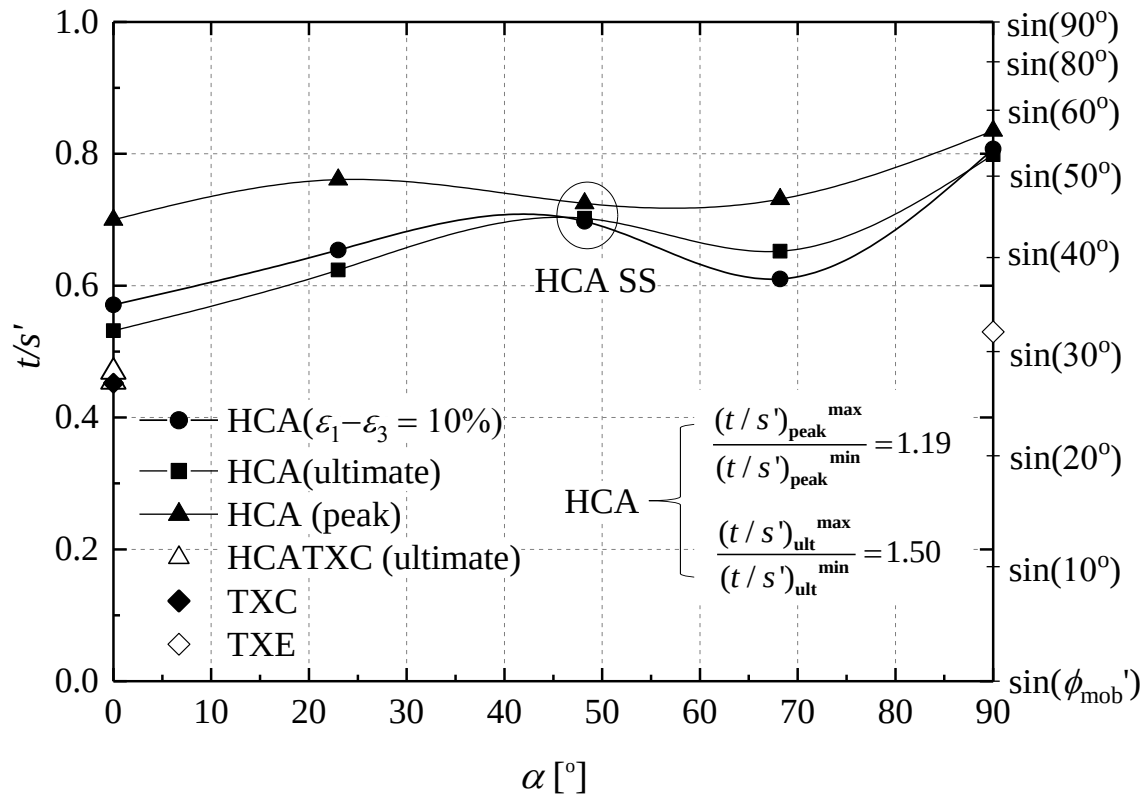


Figure 22 Variations in stress ratio t/s' with the orientation of major principal stress

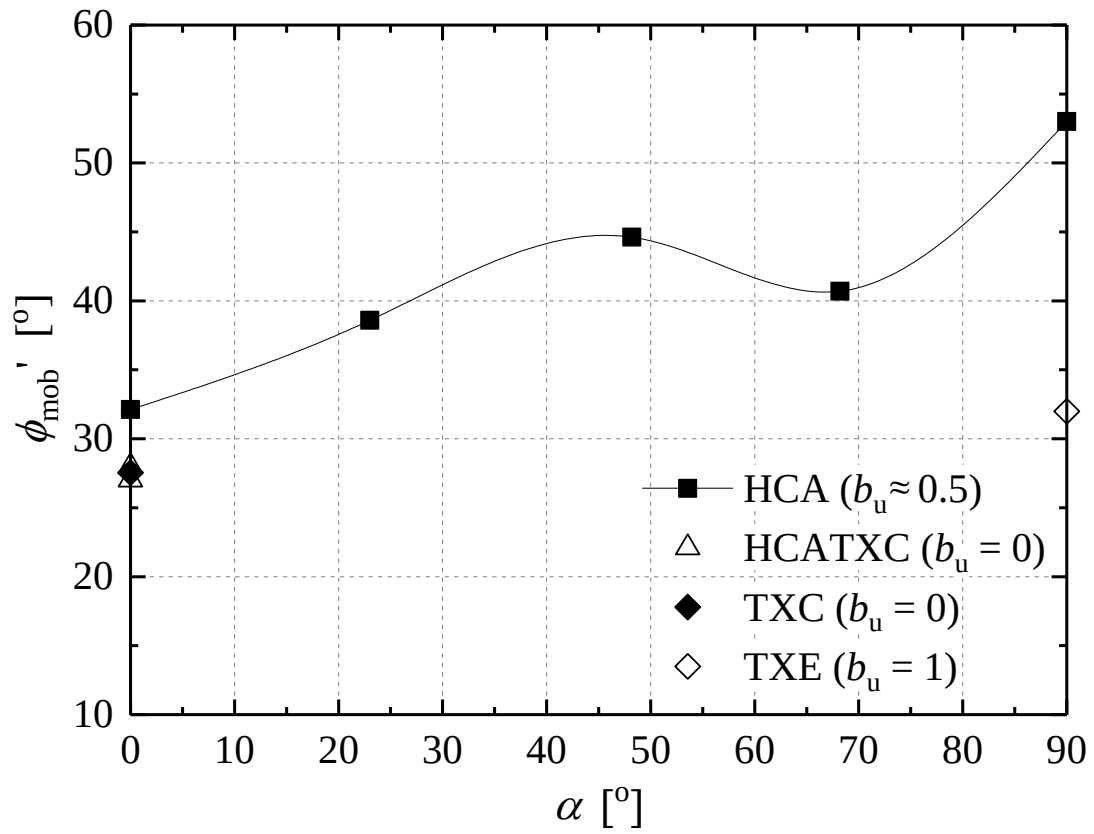


Figure 23 Variations in ultimate shear resistance angle with the orientation of major principal stress

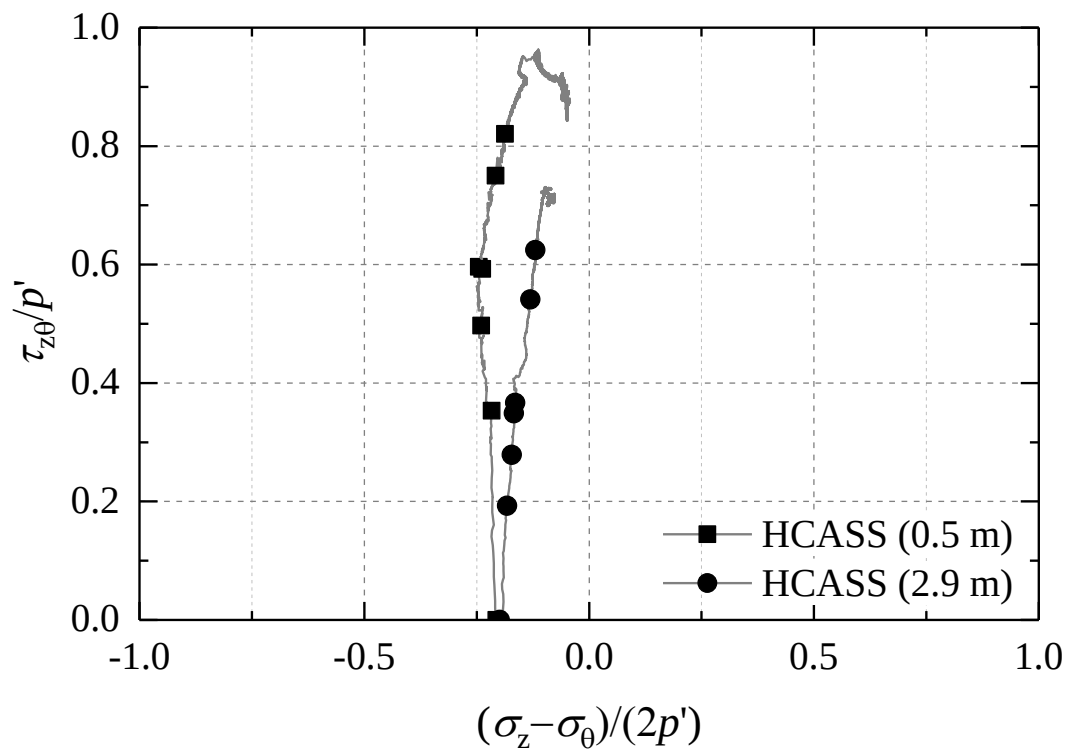


Figure 24 Projection of the undrained HCA simple shear stress paths

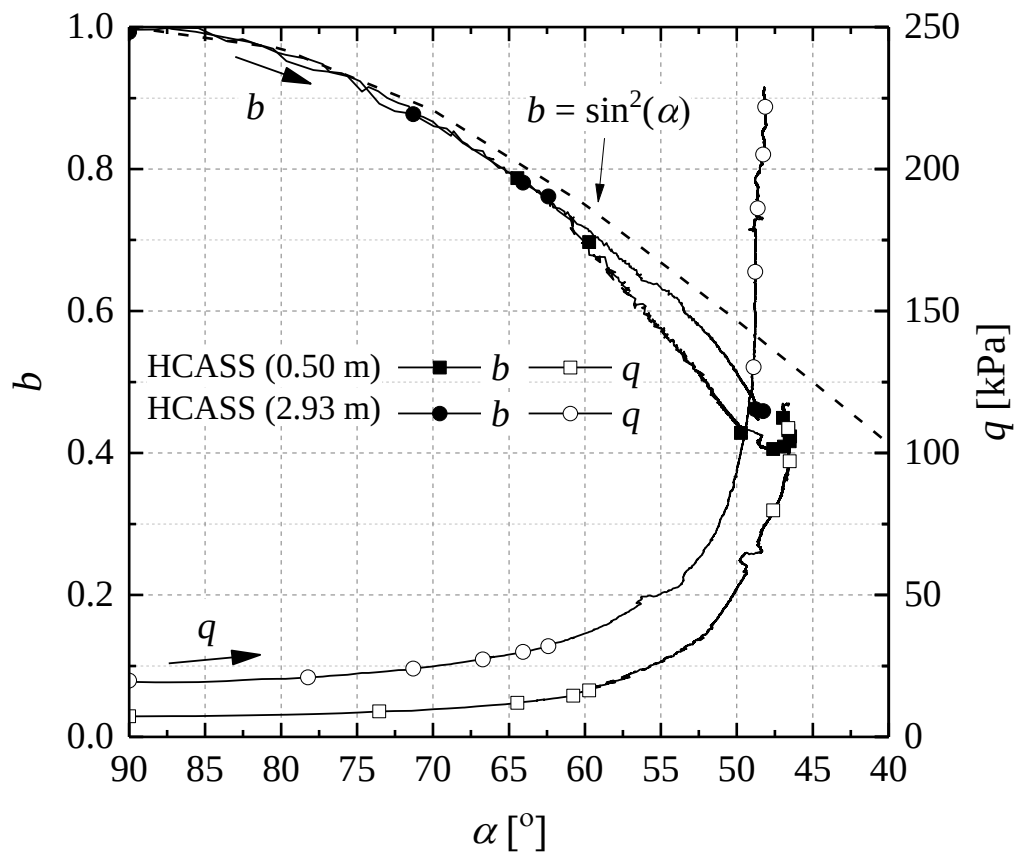


Figure 25 Variation trends of the major principal stress direction α and intermediate principal stress factor b during undrained HCA simple shear