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Model CPTs in Chalk

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ABSTRACT

Cemented geomaterials exist in many parts of the world. Structure and bonding largely influence their strength, stiffness, permeability and other hydromechanical properties. Despite the CPT being the most widely used geotechnical engineering soil characterization tool, most existing correlations between penetration resistance and soil parameters apply only to uncemented granular deposits. Application of existing correlations to cemented geomaterials such as soft rocks can produce misleading interpretation making CPT application more challenging. In particular, CPT databases correlating tip resistance with yield stress of the intact material show a wide scatter prompting the need for a better understanding of the mechanics of cone penetration in soft bonded materials. In this work, 1g small-scale model CPTs are performed in a soft rock, whilst in-test X-ray techniques help to reveal mechanisms behind the penetration process. Thereafter, experimental results are compared to field scale results and those modelled using the Particle Finite Element method which is geared toward large deformation analyses. The combined interpretation of the experimental and numerical data is then used to discuss some of the unique attributes of CPT behaviour in soft rock.

Keywords: Model CPT testing, Soft rock, CT and X-ray, Soil classification methods

1. Introduction

CPT testing is a standard investigation tool used to assess soil group classification (Robertson, 2016), support use of axial and lateral design approaches as well as understand permeability and other in-situ characteristics. CPT based design approaches have recently been extended to encompass more complex materials, such as soft rock, like chalk, see e.g., ICP-18 for chalk (Jardine et al., 2023). Despite this, CPT-based correlation for rock classification remains limited (Buckley et al., 2021). Robertson (2016) addressed some of the effects of cementation and material micro-structure with an update to existing Normalized Soil Behaviour Type (SBTn) charts, however stating that its application for highly cemented materials, such as soft rocks like calcarenite and chalk remain limited.

In this paper, small-scale CPT tests are performed on soft bonded rocks. Supported by the DEM numerical results by Zheng et al. (2022) it is postulated that the effects of structure, material stiffness and limited effect of overburden may lead to less significant scaling effects than in granular materials like sand, allowing for 1g small-scale tests to partially capture the behaviour occurring at field scale. Given this, the tests are then used to offer some insights into micro scale and yield behaviour of the soft rock which contribute to the quasi-constant cone end resistance response observed at the field-scale and in numerically modelled CPT tests (Rossi, 2019). Observations of the behaviour at the cone tip during and after insertion are drawn from in-test radiography and post-installation Computed Tomography (CT) through the use of a newly developed small-scale apparatus (Riccio et al., 2024). Thereafter a discussion on the application of standard SBTn approaches to field scale CPTs is given, and possible alterations to the standard charts are suggested based on observations from the small-scale tests discussed in this study and recent field-scale CPTs in the same rock mass (Buckley et al., 2017).

2. ICE-PICK testing apparatus

The multi-axis frame developed as part of the ICE-PICK project is shown in Fig. 1.

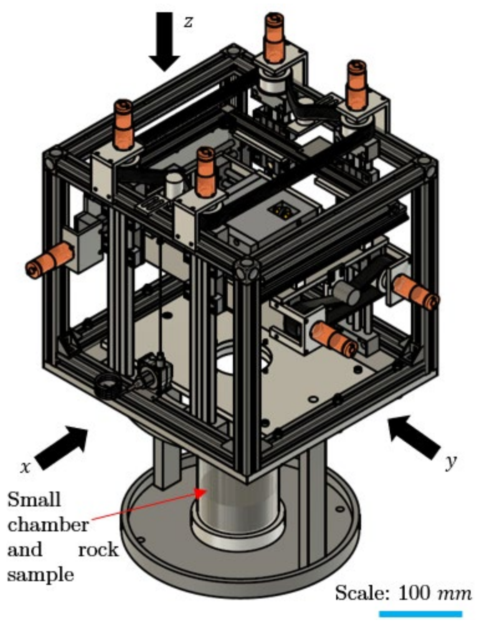

Figure 1. ICE-PICK Apparatus: Multi-axis loading frame 3D render in the small-chamber arrangement

This apparatus was designed to fulfil size, weight and operational load constraints within a CT scan bay, with the target application of studying the effects of installation and loading on displacement piles in soft rock (ICE-PICK project). Further information on the apparatus can be found in Riccio et al. (2024).

3. Experimental set up

As part of the ICE-PICK testing campaign, a total of 11 CPT tests were performed on various intact soft rocks. Cored samples (100 mm diameter, 200 mm height) of soft white chalk (UCS<3 MPa) were fully saturated and placed within a cylindrical acrylic chamber (120 mm diameter). The sealed chalk sample to chamber gap was infilled with a stiff epoxy resin to prevent radial displacement (Alvarez-Borges, 2019). The small chamber set up was fitted to the multi-axis loading frame as shown in Fig. 1. The geometry of the CPT cone consisted of a closed-ended steel rod of 8 mm diameter (D_{cpt}), radius ($r_{cpt} = 4$ mm), and standard cone angle of 60^o which was achieved through milling. This resulted in a projected cone basal area $A_{cone} = 50.26\ \text{mm}^2$. During the insertion, live radiography was used to track the tip position, mitigating the requirement for inclinometers along the pile and reducing the test-set up complexity. After the test was completed and the cone extracted, post-test CT scans were conducted to observe the extent of damage on the rock samples, allowing for comparisons and mapping to pre-existing numerical simulations (Oliynyk et al., 2023; Rossi, 2019).

3.1. Parameter derivation

The model CPT had no local instrumentation (i.e., no local measurements for pore water pressure, sleeve or end bearing resistance) and relied upon force and displacement measurements from the loading frame. The slow install rate (0.03 mm/s) and material permeability ($k_v \approx 10^{-8}$ m/s, see Riccio et al. (2023)) enabled drained conditions when following the normalized velocity approach outlined by Finnie & Randolph (1994). Given this, it was assumed that the total tip resistance ($q_c = F/A_{cone}$) was equivalent to the corrected drained cone resistance (q_t) since excess pore pressure was assumed to be negligible.

The CPT was installed to 100 mm penetration depth (z) before being fully extracted to measure the uplift force ($F_{z,up}$) from which sleeve friction ($f_s = F_{z,up}/A_{shaft}$) was initially derived where ($A_{shaft} = \pi \cdot D_{cpt} \cdot z$). The sleeve friction obtained in this way was characterised by very low values, which further reduced during pullout (Fig. 2b). This was consistent with the evidence of rock damage and radial stress decay observed for piles installed in soft rocks, e.g. Ciantia et al. (2021); Jardine et al., (2023); Previtali et al. (2023). If this profile was considered representative of the shaft resistance during installation, the cone resistance became a function of penetration depth as shown in Fig. 2a. As an alternative, the installation curve could be approximated by a bilinear function, in which the first portion is given by the transition of the rock resistance on the tip from UCS to oedometric (OED) conditions; and the second by a constant cone resistance and linearly increasing shaft/side resistance. In this instance this was simplified, and it was assumed that the function for shaft resistance linearly increased from ground level, as shown in Fig. 2b, leading to a quasi-constant cone resistance as shown in Fig. 2a.

3.2. In-test observations

Tracking of the cone movements via in-test radiography, showed that the CPT maintained reasonable verticality throughout penetration. Although some minor deviation from the centreline coincided with an increased force component in the direction of verticality loss, the cone remained within the recommended 2^o limit set out by Lunne et al., (1997).

4. Small, numerical and field scale comparisons

In Fig. 2 the derived cone (a) and sleeve resistance profiles (b) are shown alongside a field scale CPT push to 6 m below ground level in the same rock mass (Buckley et al., 2017) and numerical simulations to 0.3 m penetration using a structured modified cam-clay model calibrated based on similar intact material properties (Riccio et al., 2023; Rossi, 2019).

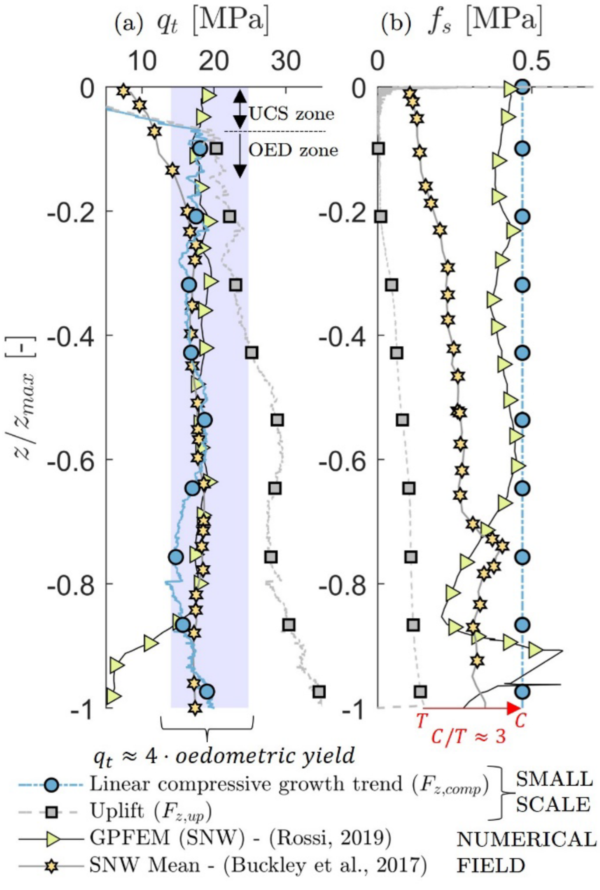

Figure 2. CPT derived properties (a) Small-scale experimental corrected cone resistance and (b) Compressive and tensile sleeve friction alongside both numerical and field CPT results for cone resistance (a) and sleeve friction (b).

Penetration depth (z) was normalized by the maximum penetration depths (z_{max}) for each representative test. Cone resistance at all scales offered a quasi-constant response following an initial peak induced early in the stroke as the intact yield of the material was overcome. The fact that similar values were obtained when compared to both field data and numerical simulations suggested that q_t was relatively unaffected by scale effects and over-burden, as alluded to by Vinck et al. (2022). This behaviour was different from that observed in sands and clays' (Idriss & Boulanger, 2006; Robertson, 2016) which at low-stress levels usually require aggressive scaling for any meaningful comparison (LeBlanc et al., 2010).

The small-scale compressive quasi-linear sleeve friction, shown in Fig. 2b, was approximately 3 times larger than that derived from the uplift, which was indicative of compressive-to-tensile (C/T) features, such as stress rotations (Jardine et al., 2023) concurrent with excessive damage around the cone tip as shown in the post-test CT (see Fig. 3a). The shaft resistance from small-scale tests was consistent with the numerical result as both considered isotropic, intact rock (Alvarez-Borges, 2019); while the field structure is characterised by weathering and discontinuities, which decrease with depth (Vinck et al., 2022). This is also suggested by the tendency of the field scale results to approach the small-scale results toward z_{max}.

4.1. Intact yield and cone resistance

Fig. 3a shows the failure mechanism observed in the post-test CT scan at the tip of the CPT cone, also observed as the volumetric strain extent in numerical simulation, see Fig. 3b Oliynyk et al., 2023). The area of destructured material extended approximately 1-2 times the cone diameter outward, in a bulbous shape.

Such failure pattern and shape are indicative of a punching-type failure, previously observed in small-scale tests on both shallow footings (Castellanza et al., 2009; Ciantia et al., 2018; Nova et al., 2008) and close-ended piles (Alvarez-Borges, 2019). For flat ended piles, this type of failure pattern, as suggested by Nova et al., (2008) and seen in numerical CPT simulations (Oliynyk et al., 2023) is associated with a quasi-oedometric type stress state, as the structured material acts as non-deformable radial confinement rather than the triaxial compression typical of soils. This corresponds to an initial quasi-elastic behaviour, followed by rapid pore-collapse (Riccio et al., 2023). The de-structuration to a densified granular material is marked by the highly non-linear stress-dependent stiffness under ideal oedometric conditions, which are constrained by the material yield ahead of the cone. This means that radial stress and destructured materials' stiffness can only increase up to that of the intact rock. Moreover, in the CPT, because of the cone shaped tip, the basal area is affected by the failure mechanism that also extends radially.

Assuming ideal conditions as above, traditional oedometric tests may be used to tentatively assess cone-resistance, as similarly done using unconfined compression tests, see e.g., Brunning & Ishak (2012). Differences in direct comparisons of oedometric yield and q_t values are postulated to be a result of the evolving basal area conditions and shape effects during the CPT insertion. In the small-scale test, the effective basal area appeared to increase as the material yielded and densified beneath the tip, as it was observed in both the CT image shown in Fig. 3a and the numerical simulation in Fig. 3b (Oliynyk et al., 2023). As mentioned above, the extent of

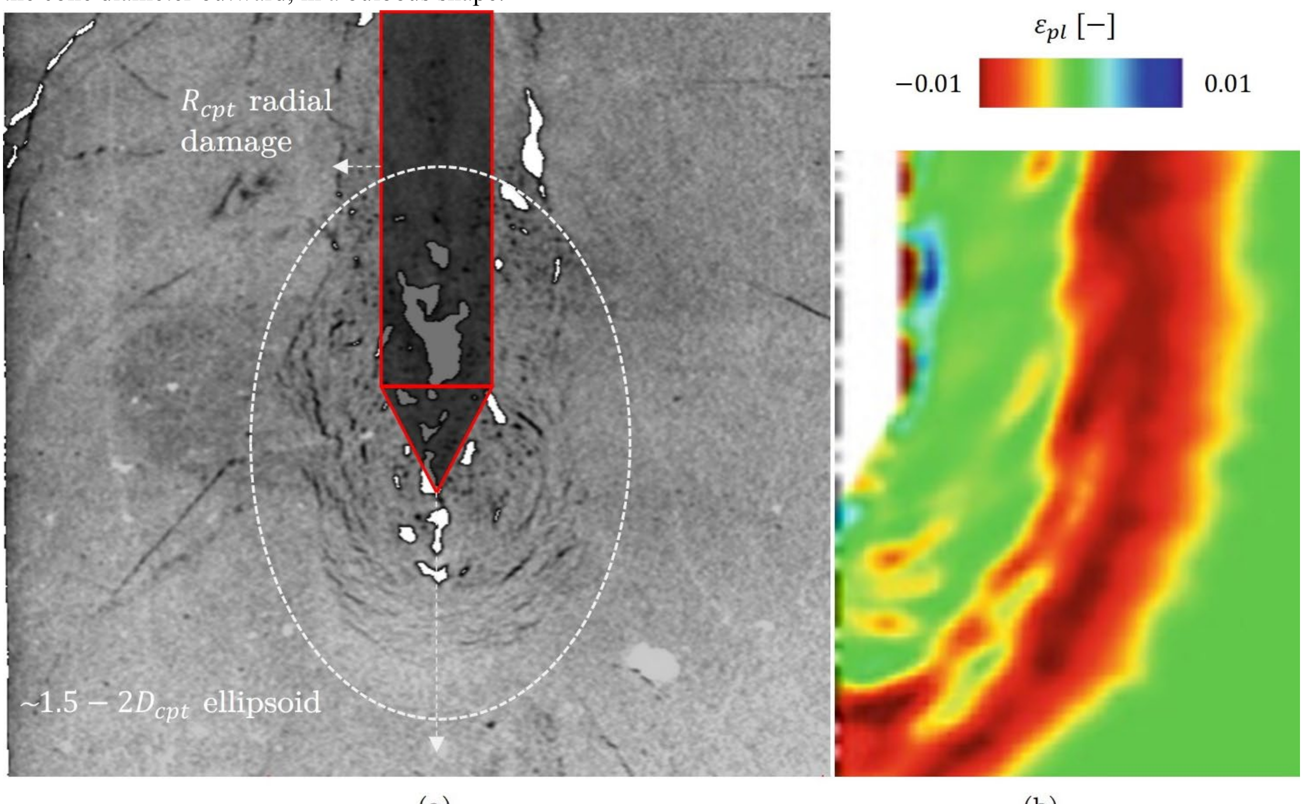

Figure 3. (a) CT centre-line slice observing cone tip state after at $z/z_{max} = 1$ (b) Numerically simulated cone insertion test in a structured comparative material (obtained from Oliynyk et al., (2023) observing volumetric strain of similar geometric extent to the small-scale test

the damage zone ahead of the tip was approximately 1.5–2D ahead of the tip and ~0.5D along the cone edges.

Using a radially instrumented deformable oedometer (Riccio et al., 2023), the elastic yield in the case of a soft rock can be identified where $k_0 = \sigma_r/\sigma_{oed}$, begins to change or where $\sigma_{oed} - \varepsilon_a$ deviates from linearity and undergoes a transition to a destructured material. This initiated at roughly $\sigma_{oed} \approx 3.5 - 5.5\ (6.2 -$ high case) MPa for the chalk used in this study and reached a transition at $\approx 1.3 - 1.6 \cdot \sigma_{oed}$. The strength of soft rocks, including σ_{oed} are known to be strongly affected by saturation degree associated with water-induced weakening effects (Ciantia et al., 2015). Variability in the limit strengths reported here and from the literature was likely influenced by this since submerged conditions were not necessarily maintained during the tests. The oedometer yield values were a little higher than the unconfined strength, consistent with the intact materials' low Poisson's ratio. Fig. 2a shows that scaling the oedometric yield for the basal area observed in the CT scan ($\approx 4 \cdot A_{cpt}$) offered a reasonable estimate of the cone resistance. It is noted that the upper bound for σ_{oed} over-estimated the cone resistance, consistent with the hypothesis of partial saturation in the oedometer test versus the fully saturated CPT test setup. This simple empirical scaling approach also provided good estimate of the field scale tests q_t which fell within the range of the scaled oedometric response (Fig. 2a).

While it is generally cheaper to carry out a field CPT than obtaining and testing rock samples, the oedometer properties can be inferred from well-established density and rock state index correlations (Matthews & Clayton, 1993) site suitability may be assessed for driven pile design, using e.g., the latest ICP design method for soft rock (Jardine et al., 2023), providing a ballpark figure at the early-stage of the project (FEED).

4.2. In-situ soil classification

There are few attempts in the literature to correlate material grade through cone resistance and friction ratio for soft rock, e.g., Buckley et al. (2021); Searle (1979), however a consistent method still has not been developed. The current SBTn approach offered by Robertson (2016) does consider the effects of cementation which may be used to evaluate the presence of micro-structure in cemented materials. Additional information to classify the behaviour may come in the form of pore pressure measurement.

In chalk and highly bonded saturated materials, pore pressure measurements at the cone shoulder (u_2) tend to be extremely high, as seen regularly in CPTu tests in various soft carbonate rocks, e.g., (Buckley, 2018; Buckley et al., 2017; Carotenuto et al., 2018). This characteristic appears to correlate with the bonded materials' propensity to contract under yield leading to extremely high pore pressures and which tend to increase with higher levels of cementation (i.e., greater sudden contraction/brittle failure). Fig. 4a to d, shows examples of various cemented materials (Bastani et al., 2018; Buckley et al., 2017; De Mio et al., 2010; Ku et al., 2013; Lunne et al., 1997) and their relation between normalized pore pressure (B_q) and cone resistance (Q_{tn}), see Eq (2) and (3), respectively.

$$B_q = \Delta u_2/\sigma'_{v0} \quad (2)$$

and

$$Q_{tn} = (q_t - \sigma_v/p_a) \cdot (p_a/\sigma'_{v0})^n. \quad (3)$$

Where u_2 is pore pressure measured at the cone shoulder (see Fig. 4) and σ'_{v0}, σ_v are effective and total stresses respectively, whilst p_a is atmospheric pressure and n is an exponent for stress changes with depth (assumed 0 in this case).

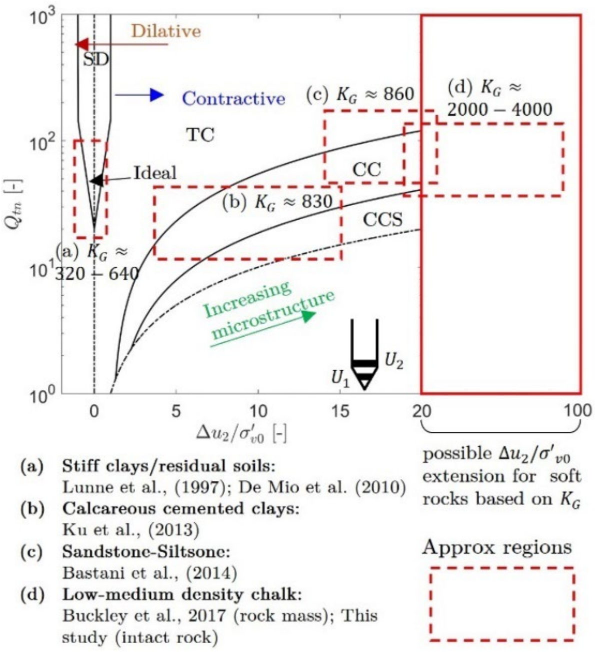

Figure 4. Normalized cone resistance (Q_{tn}) versus Normalized u_2 position pore pressure (B_q). Where CC=Clay like contractive, CCS = Clay like contractive (sensitive), TC=Transitional Contractive, SD=Sand-like dilative

Materials which were more significantly cemented appeared to exhibit more marked contractive characteristics which were supported by the extremely high pore pressures at the cone shoulder (u_2). CPT tests performed in clean structured soft chalk also exhibited extremely high pore pressures ~ 10MPa, falling far out with the standard modified SBTn chart which is indicative of highly contractive behaviour (Robertson, 2016). This is as expected for initial yield of mostly intact materials' which has been observed in isotropic and $1D - k_0$ triaxial tests, owing to structure-permitted space and sudden volumetric changes upon yield (see e.g., (Lagioia & Nova, 1995; Leroueil & Vaughan, 1990). The observations drawn from K_G (cementation level based on the ratio of stiffness and cone resistance, see Eq (4) and Q_{tn} versus B_q contrasted that of the Q_{tn} versus F_r plot.

$$K_G^* = (G_0/q_t) \cdot (Q_{tn})^{0.75} \quad (4)$$

Where G_0 is the rock mass shear modulus, typically derived from shear wave velocity or bender element tests, although the latter can over-estimate this value in weathered rock (Robertson, 2016; Vinck et al., 2022).

In that case the method indicated highly dilative behaviour, reflecting the unique material response for structured materials which may not always be correctly identified in standard soil classification approaches as also highlighted by Robertson (2016).

These observations drawn from various cemented materials suggest that further discussion is needed to address the challenges of in-situ classification of soft rock materials. It is thought that the Q_{tn} verus B_q SBTn chart may require extension as shown in Fig. 4 to encapsulate soft rocks which typically exhibit highly contractive behaviour. Whilst the authors believe that increasing levels of cementation play a role in an increasingly contractive response, additional factors such as material fabric, initial porosity and evolving permeability during yield also influence this response again confirming the highly unique characteristics of soft rock.

5. Conclusion

In this short paper, small-scale CPT tests performed in soft chalk rock have been discussed. The results obtained closely resemble field and numerical results, irrespective of scaling and low-stress states. The tests have also been used to assess existing classification techniques in the literature, and new observations have been drawn on their applicability in soft rock materials. It is postulated that with full CPT measurements one can assess broadly the state of the soft rock material and it is thought that u_2 measurements may be a key identifier of the marked contractive behaviour exhibited at the cone tip during insertion. Without such knowledge, classic soil classification techniques may indicate a dilative state, leading to subjectivity in the assessment of the soft rock condition.

A new multi-axis loading frame was used to facilitate small-scale CPT tests, and its compactness allowed for in-test radiography to be used throughout the test to assess the CPT verticality and the general rock behaviour. Furthermore, post-test CT scans enabled observations on the damage induced around the cone tip, as also observed in equivalent numerical simulations. The observations obtained from the CT imaging allowed for the development of a simple linear scaling rule to predict cone resistance from oedometric tests which offered good comparison to both the field and numerical tests. This highlights the importance of small-scale testing and its possible application in improving the knowledge and engineering application of soft rock materials which are present in many offshore locations.

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