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Parameter variability of undrained shear strength and strain using a database of reconstituted soil tests

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Abstract: During construction, the mobilization of undrained shear strength must be limited to avoid soil failure. Soil strains must be controlled to avoid compromising structural serviceability. To assess foundation performance by strength mobilization, an understanding of soil strains at various levels of strength mobilization is required. In practice, ground investigation data are often limited, and assessment of the expected variation of stress–strain and undrained shear strength is improved with empirical correlations calibrated with a database. The new database RFG/TXCU-278 contains data of 278 consolidated–undrained triaxial tests on reconstituted fine-grained soil samples compiled from the literature. Analysis of the database to evaluate the variability of undrained strength ratio ($c_u/\sigma_0'$) and a reference shear strain with shear mode is undertaken in this paper. The new database provides evidence that shear strain (like undrained shear strength) is sensitive to the consolidation (isotropic or $K_0$) and shear mode (triaxial compression or extension) applied in the test. For the materials included in the database, the strength mobilization parameters obtained from a triaxial compression test can be used to predict the corresponding triaxial extension parameters to a reasonable accuracy.

Key words: databases, stress–strain, fine-grained soils.

Introduction

Prediction of soil strains at increments of stress ratio or mobilization level (cf. BSI 1994) allows engineers to better evaluate foundation performance. For this purpose, the “Mobilization Strain Framework” (MSF) is a convenient model for characterizing nonlinear stress–strain from undrained shear test data (Vardanega and Bolton 2011; Vardanega et al. 2012). Whether a simple hand calculation or a more advanced numerical model is used, checking the possible range of ground movements at any mobilization level requires the selection of representative design parameters, which are often determined by experiment. If a test is representative of the site, $\gamma_{ref}$ (a reference strain parameter) controls undrained foundation settlement at 50% mobilization. This important question of whether the soil test is representative can only be answered if the possible variation of $\gamma_{ref}$ within the displacement mechanism is known.

Understanding parameter variability by database analysis

Variability of a soil test parameter arises from an incomplete knowledge of its variation with different test conditions together with the contribution of natural geological variation. Database analysis is an essential tool for characterizing geotechnical variability (e.g., Kulhawy and Mayne 1990; Mayne 1980; Phoon and Kulhawy 1999a, 1999b; Ching and Phoon 2013, 2014a) and many empirical correlations between measured parameters are available in the literature. A decision to use general parameter trends for design depends largely upon the availability of data from the ground investigation. For example, to measure the anisotropy of undrained shear strength ($c_u$) at a ground investigation site, advanced testing apparatus such as the hollow cylinder could be used to shear soil specimens to peak failure following different complex stress paths (e.g., Brosse et al. 2017). Alternatively, a variety of soil tests can be employed to assess the effect of shear mode on $c_u$ variation (e.g., Low et al. 2011; Ratananikom et al. 2011).
In practice, projects often have limited scope for detailed ground investigation and advanced experimental work. When project test data are scarce, predicting the variation in \( c_u / \sigma_{oc}^{\prime} \) or \( \gamma \) must necessarily be estimated from any available test information. Databases such as RFG/TXCU-278 can also be used to establish prior estimates of relevant statistical parameters for subsequent Bayesian analysis as has been done for standard geotechnical parameters in the works of Cao and Wang (2014), Cao et al. (2016), and Wang et al. (2016).

**Variation of undrained strength ratio**

It is well established that \( c_u \) is sensitive to its method of measurement. For example, differences have been observed from comparisons of test specimens that were either unconsolidated or reconsolidated (Chen and Kulhawy 1993), isotropically or anisotropically consolidated (Mayne 1985), sheared in different directions (Mayne and Holtz 1985), and at various strain rates (Sheahan et al. 1996; Kulhawy and Mayne 1990). Undrained shear strength is also known to vary with stress history (Ladd et al. 1977; Mayne et al. 1996; Kulhawy and Mayne 1990). Undrained shear strength is sensitive to the consolidation type (isotropic or anisotropic) and OCR is known (Vardanega and Bolton 2011) and can be expressed as

\[
\frac{c_u}{\sigma_{oc}^{\prime}} = \text{OCR}^{\Lambda}
\]

(1)

where \( c_u / \sigma_{oc}^{\prime} \) is the normalized strength of an overconsolidated material; \( c_u / \sigma_{NC}^{\prime} \) is the normalized strength of a normally consolidated material; \( \sigma_{OC}^{\prime} \) is the present vertical effective consolidation stress; OCR is the ratio of maximum past vertical effective consolidation stress to present vertical effective consolidation stress; and \( \Lambda \) is a fitted exponent.

Using large databases of soil tests, Mayne (1988) and Ching and Phoon (2014b) showed that the fitted regression coefficient \( \Lambda \) was sensitive to the consolidation type (isotropic or anisotropic) and mode of shear (triaxial compression or extension). Following the framework proposed by Kulhawy and Mayne (1990), Ching and Phoon (2013) developed a data-driven method to standardize \( c_u / \sigma_{OC}^{\prime} \) using modification factors to capture the effects of different test mode, OCR, strain rate, and plasticity index.

**Variation of mobilized strain**

While much research effort has focused on understanding \( c_u \) variability, less information is available to quantify the variability of shear strains. The MSF has been developed to incorporate undrained strength mobilization parameters in a framework suitable for reliability-based design style approaches (Vardanega and Bolton 2016a) by employing a simple power-law model. Equation 2 can be fitted to shear stress–strain data if the peak failure stress is known (Vardanega and Bolton 2011) and can be expressed as

\[
\frac{1}{M} = \frac{\tau_{mob}}{c_u} = 0.5 \left( \frac{\gamma}{\gamma_{50 \text{ CKU}}} \right)^{b_{CKU}} 0.2 \leq \frac{\tau_{mob}}{c_u} \leq 0.8
\]

where \( M \) is the mobilization factor (which is akin to a reduction factor on undrained shear strength); \( \tau_{mob} \) is the mobilized shear strength; \( \gamma \) is the shear strain; \( \gamma_{50 \text{ CKU}} \) is the shear strain to mobilize 0.5\( c_u \) under isotropically consolidated undrained conditions (denoted in previous works as \( \gamma_{ref} \) and for compression and extension tests the notations \( \gamma_{50 \text{ CRUC}} \) and \( \gamma_{50 \text{ CKUE} \) are respectively used in this paper); and \( b_{CKU} \) is an exponent to describe nonlinear-
Table 1. Sources of experimental data in RFG/TXCU-278.

| Reference                  | Test material                        | $w_l$ (%) | $I_P$ (%) | Test modes      | OCR range | Excluded test data                                      |
|---------------------------|--------------------------------------|-----------|-----------|-----------------|-----------|--------------------------------------------------------|
| **Isotropically consolidated undrained triaxial shear tests** |                                      |           |           |                 |           |                                                        |
| Parry (1956, 1960)        | Weald clay ($n = 8$)                 | 43        | 25        | CIUC ($n = 6$), CIUE ($n = 2$) | 1–12      | 8 undrained tests available for digitization (drained tests excluded) |
| Gasparre (2005) - 6 of 7 tests from Abdulhadi (2004) | London clay ($n = 7$)               | 63–67     | 35–41     | CIUC ($n = 7$)   | 1–12      | 6 tests excluded from digitization due to poor resolution |
| Gens (1982)               | Lower Cromer till ($n = 10$)        | 25        | 12        | CIUC ($n = 5$), CIUE ($n = 5$) | 1–10      | All undrained tests included (drained tests excluded)   |
| Loudon (1967)             | Kaolin ($n = 8$)                     | 74        | 32        | CIUC ($n = 8$)   | 1–8.1     | —                                                      |
| Liu (2004)                | Kaolin ($n = 22$)                    | 56        | 24        | CIUC ($n = 11$), CIUE ($n = 11$) | 1–8       | 3 tests excluded from digitization due to poor resolution |
| Sachan and Penumadu (2007)| Kaolin ($n = 12$)                    | 62        | 30        | CIUC ($n = 6$), CIUE ($n = 6$) | 1–10      | 6 tests on “flocculated” samples included; 6 tests on “dispersed” samples excluded |
| Conn (1988)               | Keuper marl ($n = 20$)               | 36        | 17        | CIUC ($n = 9$), CIUE ($n = 11$) | 1–10      | —                                                      |
| Valls-Marquez (2009)      | Kaolin ($n = 11$)                    | 65        | 32        | CIUC ($n = 7$), CIUE ($n = 4$) | 1–5.1     | —                                                      |
| Braathen (1966)           | Boston Blue clay ($n = 3$)           | 45.5      | 22.3      | CIUC ($n = 3$)   | 1–8.1     | 4 cyclic tests and 7 anisotropically consolidated tests excluded from digitization as per selection criteria; 1 CIUC test (OCR = 2) excluded due to possible seating-bedding error |
| Gens (1980)               | Lower Cromer till ($n = 10$)        | 26        | 12        | CIUC ($n = 5$), CIUE ($n = 5$) | 1–10      | —                                                      |
| Atkinson and Little (1988)| Ware Lodgement till ($n = 7$)       | 40        | 22        | CIUC ($n = 7$)   | 1–32      | 10 “tubed” (intact) samples excluded                    |
| Kamal (2012)              | Oxford clay ($n = 5$)                | 66        | 32        | CIUC ($n = 5$)   | 1–10      | —                                                      |
| Gault clay ($n = 3$)      | 74        | 46        | CIUC ($n = 3$)   | 1–5       | —                                                      |
| Kimmeridge clay ($n = 3$) | 49        | 26        | CIUC ($n = 3$)   | 1–5      | —                                                      |
| Vardanega et al. (2012)b  | Kaolin ($n = 18$)                    | 62.6      | 33        | CIUC ($n = 18$)  | 1–20      | —                                                      |
| Parry and Nadarajah (1974)| Kaolin ($n = 8$)                     | 72        | 32        | CIUC ($n = 4$), CIUE ($n = 4$) | 1–2.3     | —                                                      |
| **$K_0$-consolidated undrained triaxial shear tests** |                                      |           |           |                 |           |                                                        |
| Fayad (1986)              | Boston Blue clay ($n = 7$)           | 42        | 21        | CIUC ($n = 1$)   | 7.5       | —                                                      |
| Zhu and Yin (2000)        | Hong Kong Marine clay ($n = 24$)     | 60        | 32        | CIUC ($n = 12$), CIUE ($n = 12$) | 1–8       | —                                                      |
| Atkinson and Little (1988)| Ware Lodgement till ($n = 7$)       | 40        | 22        | CIUC ($n = 7$)   | 1–32      | 10 “tubed” (intact) samples excluded                    |
| Kamal (2012)              | Oxford clay ($n = 5$)                | 66        | 32        | CIUC ($n = 5$)   | 1–10      | —                                                      |
| Gault clay ($n = 3$)      | 74        | 46        | CIUC ($n = 3$)   | 1–5       | —                                                      |
| Kimmeridge clay ($n = 3$) | 49        | 26        | CIUC ($n = 3$)   | 1–5      | —                                                      |
| Vardanega et al. (2012)b  | Kaolin ($n = 18$)                    | 62.6      | 33        | CIUC ($n = 18$)  | 1–20      | —                                                      |
| Parry and Nadarajah (1974)| Kaolin ($n = 8$)                     | 72        | 32        | CIUC ($n = 4$), CIUE ($n = 4$) | 1–2.3     | —                                                      |

Note: Digitized peak deviator stress and shear strain $\gamma = 1.5$ times axial strain have been used to develop all the correlations in this paper. $n$, number of tests.

*Intwostudies (Gasparre 2005; Sheahan 1991), the authors identify $w_l$ and $w_P$ values for the block sample associated with each reconstituted specimen, while the other studies indicate a single “best estimate” value for the set of specimens.

*bExperimental data of the triaxial tests published by Vardanega et al. (2012) were reanalysed and re-filtered from the original data source for this paper.
23 fine-grained soils from 21 publications. Shear-stress–strain data from 278 consolidated undrained triaxial tests were digitized or acquired from the authors’ tabulated data where available. The selection criteria for the database were (i) multiple experiments using reconstituted samples of natural fine-grained soil that were (ii) consolidated at different OCRs, under isotropic or $K_0$ conditions, and (iii) subsequently sheared in triaxial compression or extension up to peak failure to examine the effect of applied shear mode. (Several datasets included samples sheared in compression only, to increase the range of soil types studied — see Table 1.)

Strain rate corrections were not applied to the digitized test data as a universal modification factor for strain measurements was not available. Previous studies have shown that $c_u$ increases by 10%–20% per log cycle of increased strain rate (e.g., Kulhawy and Mayne 1990). The number of digitized data points for each triaxial test ranged from 3 to around 200 with a mean of 24. Therefore, for consistency the model parameters were derived by applying either eq. 2 or 3 as appropriate to the digitized test data and then using the fitted equation to calculate them (e.g., $\frac{c_u}{\sigma'_{vo}}$ and $\sigma'_{vo}$).

### Classification of database samples

Classification of the 23 experimental soils indicate a wide range of plasticity (see Supplementary Material, Fig. S1), with about 70% of materials classified as inorganic and medium-high plasticity. Materials classified outside of this range include the processed kaolin clays, which cluster close to the A-line, and a low-plasticity glacial till investigated by Gens (1982). With the exception of the kaolin materials, all soils included in RFG/TXCU-278 were sampled from natural deposits.

### Analysis

The power-law model (eq. 2 or 3) was fitted to the data points of 271 tests with a range of $0.779 \leq R^2 \leq 0.9999$ and $0.0017 \leq \text{S.E.} \leq 0.0925$. Seven tests provided only peak stress data. The collected test database is presented in sub-databases of specimen consolidation type (isotropic or $K_0$) and shear mode (triaxial compression or triaxial extension), which are identified by test mode i.e., CIUC, CIUE, CKUC, and CKUE. Undrained strength data from a triaxial test database of natural clays, digitized from Mayne and Holtz (1985), are also presented for comparison: about 75% of each sub-database consists of normally consolidated specimens, with OCR ranging from 1 to 25 for CIU tests and 1 to 20 for CKU tests.

Empirical correlations (or transformation models) of the test parameters were investigated using linear regression analysis and standard errors were calculated to describe scatter in the data (cf. Phoon and Kulhawy 1999a, 1999b). An alternative description of parameter variability using predicted vs. measured plots and bandwidths of prediction error is valuable (Koutsofias et al. 2017; Kootahi and Mayne 2017), particularly when evaluating the variability of different parameters (or the uncertainty of different transformation models). All factor errors quoted in this paper refer to a region that encompasses 80% of the data points and may be viewed in graphical form in the Supplementary Material.

### Correlation between triaxial extension and compression parameters

The undrained strength (Fig. 1) and strain parameters (Fig. 2) obtained for each digitized test are presented by comparing extension and compression modes. Pairs of tests (i.e., extension and compression) on the same material with identical OCR (±0.1) and strain rate were selected from each published series of experiments. Linear regression analysis indicates that a significant relationship exists between $c_u/\sigma'_{vo}$ measured in compression and in extension for samples consolidated under either isotropic or $K_0$ stresses, with a high coefficient of determination and $p < 0.001$.

[Fig. 1. Comparison of normalized undrained shear strength from triaxial extension and compression tests on two similarly reconstituted specimens for (a) CIU tests and (b) CKU tests. (Colour online.)]
In some extent, greater strength anisotropy and more variability than reconstituted soils. A comparison of predicted vs. measured reconstituted soil data shows the factor error of the regression to be 1.3–1.4 depending on consolidation type (see Supplementary Material, Fig. S21).

### Table 2

| Sample type | Database reference | Test mode | Mean | Standard deviation | n | Reference |
|-------------|-------------------|-----------|------|--------------------|---|-----------|
| Reconstituted | This study | CIUC | 0.459 | 0.143 | 114 | This study |
| Reconstituted | This study | CIUE | 0.399 | 0.082 | 55 | This study |
| Intact | Vardanega and Bolton (2011) | CIU | 0.608 | 0.158 | 92 | Vardanega and Bolton (2011) |

### Table 3

| Sample type | Database reference | Test mode | Slope regression coefficient | Equation | n | R² | S.E. | p-Value | Error bounds (%) |
|-------------|-------------------|-----------|-------------------------------|----------|---|-----|-------|----------|------------------|
| Reconstituted | This study | CIUC | 0.288 | (9) log 10(OCR) = 0.653 log10(OCR) – 0.541 | 115 | 0.86 | 0.114 | <0.001 | 80 |
| Reconstituted | This study | CIUE | 0.267 | (10) log 10(OCR) = 0.729 log10(OCR) – 0.574 | 55 | 0.92 | 0.083 | <0.001 | 80 |
| Intact | Mayne (1988) | CIUC | 0.20 | - | 1–20 | 0.58 | 0.058 | <0.001 | 80 |
| Intact | Mayne (1988) | CIUE | 0.20 | - | 1–20 | 0.60 | 0.060 | <0.001 | 80 |
| Reconstituted | This study | CKUC | 0.300 | (11) log 10(OCR) = 0.790 log10(OCR) – 0.522 | 74 | 0.96 | 0.015 | <0.001 | 80 |
| Intact | Mayne (1988) | CAUC | 0.20 | - | 1–20 | 0.78 | 0.078 | <0.001 | 80 |
| Reconstituted | This study | CKUE | 0.165 | (12) log 10(OCR) = 0.952 log10(OCR) – 0.782 | 55 | 0.94 | 0.067 | <0.001 | 79 |
| Intact | Mayne (1988) | CAUE | 0.12 | - | 1–20 | 0.85 | 0.085 | <0.001 | 79 |

### Figure 2

- (a): Comparison of CIU and CKU from triaxial extension tests on two similarly reconstituted specimens. 
- (b): Comparison of CIU and CKU from triaxial compression tests on two similarly reconstituted specimens.

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Significant correlations also exist between the reference strains measured in triaxial extension and compression, although only reconstituted soil data are available. Figure 2 shows that the reference strains are less sensitive to shear mode if tested from an isotropic stress state: the slope regression coefficient for CKU tests is five times the slope for CIU tests. Reference strains mobilized in CKUE, in some cases, are one order of magnitude greater than the strains mobilized in CKUC; considerable scatter of the strain anisotropy (see Supplementary Material, Fig. S3) warrants further investigation. No correlation to describe the shear mode effect was found for $b_{\text{CKU}}$ or $b_{\text{CIU}}$ (see Table 2 for average means and standard deviations) although the CKU tests analysed here show more disparity between compression and extension (see Supplementary Material, Fig. S4). Using the framework presented in this paper, a designer could justify the likely variation of reference strain from a single triaxial compression test with no prior information about the material or in situ conditions. From this database, the prediction of triaxial extension reference strain includes a factor error of 1.7–2.2 (dependent on CIU or CKU test conditions), which can be incorporated into sensitivity analyses.

Fig. 3. (a) Variation of $\gamma_{50,\text{CIU}}$ with OCR for all CIUC and CIUE tests in the database. (b) Variation of $\gamma_{50,\text{CKU}}$ with OCR for all CKUC and CKUE tests in the database. Previously reported trend $\gamma_{50,\text{CKUC}} = 0.0004OCR^{1.57}$ (Casey 2016), eq. 4, is shown for comparison. [Colour online.]
Estimation of OCR

Using only two measurements of c_u/c'_{or} at different depths, the SHANSEP framework (Ladd et al. 1977; Mayne 1988) can be adopted to assess OCR of the soil using eq. 1. Table 3 shows the values of (c_u/c'_{or}) and λ by shear mode for the sub-databases presented here and in other studies (see also Mayne et al. 2009) for values of (c_u/c'_{or}) by test mode. The reference strain data in Fig. 3 suggest that a similar approach can be used with measurements of γ. The new transformation models given by eqs. 5–8 identify positive correlations between the reference strain and OCR in all four test modes. Hence, with knowledge of a reference strain from a triaxial test, OCR may be estimated (using an analogous approach to that shown in Mayne 1988 with c_u).

Using eqs. 5–8 (Fig. 3), OCR can be approximated with a factor error of 1.5–2.7 for the selected consolidation-shear mode. Adopting the SHANSEP framework (eqs. 9–12, given in Table 3) to estimate OCR produces a factor error of 1.3–1.8. Perhaps as expected, soil mobilization strains are a poorer predictor of OCR than undrained shear strength (i.e., the correlations have lower R^2 values). Figures S7 and S8 (see Supplementary Materials) show that for eqs. 5–8, around 80% of the data plots within a bandwidth of 1.7–2.1 factor error (about the predicted = measured line). Equation 6 is of similar form to eq. 4 and this may be partly explained by some shared data from Abdulhadi (2009).

\[ \gamma_{50\text{CRUC}} = 0.0010(OCR) + 0.0074 \]
\[ (n = 114, R^2 = 0.51, S.E. = 0.0051, p < 0.001) \]

\[ \gamma_{50\text{CRUC}} = 0.0013(OCR) + 0.0042 \]
\[ (n = 55, R^2 = 0.65, S.E. = 0.0033, p < 0.001) \]

\[ \log_{10}(\gamma_{50\text{CRUC}}) = 3.15 \log_{10}(OCR) \]
\[ (n = 67, R^2 = 0.79, S.E. = 0.234, p < 0.001) \]

which can be rearranged as

\[ \gamma_{50\text{CRUC}} = 0.00049(OCR)^{1.35} \]

\[ \gamma_{50\text{CRUC}} = 0.0038(OCR) \]
\[ (n = 30, R^2 = 0.45, S.E. = 0.0086, p < 0.001) \]

Summary

RFC/TCU-278 is a large database of triaxial tests on reconstituted soil samples that has been analysed by consolidation mode (isotropic or K0) and shear mode (compression or extension) to quantify the variability of shear strength and strain. Undrained strength ratio (c_u/c'_{or}) and MSF parameters \( \gamma_{50\text{CRU}} \gamma_{50\text{CRU}} b_{\text{CRU}} b_{\text{CRU}} \) and \( P_{\text{CRU}} \) were chosen to study the influences of shear mode (isotropic or K0) and OCR on parameter variability. The correlations presented for quantifying the variability of the undrained strength and strain parameters in this study may not be representative of intact materials, but the general trends are useful for those wishing to assess the effects of OCR (less reported for studies on intact soils) and shear mode. Factor errors of the new transformation models provide a useful indication of parameter variability related to the uncertain effects of different experimental procedures and the material variability of reconstituted soils.

Data availability statement

This research has not generated new experimental data.

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List of symbols

B stress ratio for tests sheared from anisotropic conditions with initial shear stress

fitted exponent eq. 2 in power-law regression of normalized shear strain $G_{0}/G_{CR}$ and stress ratio $(\sigma_{moh}/k_{0})$

exponent that describes nonlinearity for isotropically consolidated undrained compression tests

exponent that describes nonlinearity for isotropically consolidated undrained extension tests

exponent that describes soil nonlinearity for $K_{cr}$ consolidated undrained compression tests

exponent that describes soil nonlinearity for $K_{cr}$ consolidated undrained extension tests

anisotropically consolidated undrained triaxial compression

anisotropically consolidated undrained triaxial extension

isotropically consolidated undrained test of any shear mode

isotropically consolidated undrained triaxial compression

isotropically consolidated undrained triaxial extension

$K_{cr}$-consolidated undrained test of any shear mode

$K_{cr}$-consolidated undrained triaxial compression

$K_{cr}$-consolidated undrained triaxial extension

undrained shear strength

normalized undrained shear strength

normalized undrained shear strength of a normally consolidated material

null hypothesis

plasticity index

ratio of horizontal to vertical stress with zero lateral strain

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$M$ mobilization factor (which is akin to a reduction factor on undrained shear strength)

MSF mobilization strain framework

$n$ number of data points

OCR overconsolidation ratio (maximum past consolidation stress to present consolidation stress)

$p$ calculated probability of finding the observed value to be at least as extreme as the test statistic when the null hypothesis $H_0$ is true

$R^2$ coefficient of determination

S.E. standard error

SHANSEP stress history and normalized soil engineering properties

$W_L$ liquid limit

$W_P$ plastic limit

$\gamma$ shear strain

$\gamma_{so\ CIU}$ reference shear strain to mobilize $0.5c_u$ in an isotropically consolidated undrained test

$\gamma_{so\ CIUC}$ reference shear strain to mobilize $0.5(c_u - \tau_0)$ in a $K_0$ consolidated undrained test

$\gamma_{so\ CKUC}$ reference shear strain to mobilize $0.5(c_u - \tau_0)$ in a $K_0$ consolidated undrained compression test

$\gamma_{so\ CKUE}$ reference shear strain to mobilize $0.5(c_u - \tau_0)$ in a $K_0$ consolidated undrained extension test

$\gamma_{ref}$ a reference shear strain

$A$ fitted exponent in power-law regression of normalized undrained strength and OCR

$\sigma_{c0}'$ current vertical consolidation stress

$\tau_0$ initial shear stress

$\tau_{mob}$ mobilized shear strength