Load-controlled cyclic T-bar tests: a new method to assess the combined effects of cyclic loading and consolidation

C. D. O’Loughlin, Z. Zhou, S. A. Stanier and D. J. White

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Conleth D. O’LOUGHLIN
Centre for Offshore Foundation Systems and ARC Research Hub for Offshore Floating Facilities
University of Western Australia
Perth, WA 6009, Australia
Tel: +61 8 6488 7326
Email: conleth.oloughlin@uwa.edu.au

Zefeng ZHOU (corresponding author)
Centre for Offshore Foundation Systems and ARC Research Hub for Offshore Floating Facilities
University of Western Australia
Perth, WA 6009, Australia
Tel: +61 403848151
Email: zefeng.zhou@research.uwa.edu.au

S. A. STANIER
University of Cambridge and ARC Research Hub for Offshore Floating Facilities
Cambridge CB2 1PZ, UK
Tel: +44 7856 009042
Email: sas229@cam.ac.uk

David J. WHITE
University of Southampton and ARC Research Hub for Offshore Floating Facilities
Southampton, Southampton SO17 1BJ, UK
Tel: +44 23 8059 6859
Email: david.white@soton.ac.uk

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A new method to assess the combined effects of cyclic loading and consolidation

C. D. O’LOUGHLIN¹, Z. ZHOU², S. A. STANIER³ AND D. J. WHITE⁴

Full-flow T-bar and ball penetrometer tests are often used to measure intact and remoulded soil strengths, with the latter determined after several large amplitude displacement cycles. In offshore design the remoulded soil strength is often the governing design parameter during installation of subsea infrastructure, whilst a ‘cyclic strength’ applies for the less severe operational cyclic loading. This paper utilises T-bar penetrometer tests to measure both remoulded and cyclic strengths, where the latter is determined via a new test protocol involving cycles between load rather than displacement limits. The tests use kaolin clay and a reconstituted carbonate silt and involve three cyclic phases with intervening consolidation periods. The results demonstrate the important and beneficial role of consolidation, with the loss in strength due to remoulding sometimes surpassed by the strength recovery from consolidation. The most significant gains in strength, to 2.5 times the initial value, were measured in the load-controlled cyclic tests. These data demonstrate a novel way to characterise undrained cyclic strength, taking advantage of consolidation to reduce conservatism.

Keywords: penetrometer, soil strength, cyclic loading, soil strength, consolidation and remoulding.

¹ Centre for Offshore Foundation Systems, The University of Western Australia, Crawley, WA 6009, Australia
² Centre for Offshore Foundation Systems, The University of Western Australia, Crawley, WA 6009, Australia (Corresponding author: Email: zefeng.zhou@research.uwa.edu.au)
³ University of Cambridge, Cambridge CB2 1PZ, UK
⁴ University of Southampton, Southampton SO17 1BJ, UK
INTRODUCTION

Offshore foundations are subject to cyclic loading from the ocean environment and from operational loads, such as expansion and contraction of pipelines. In conventional design, cyclic loading of fine-grained soils is treated as ‘damage’, so the cyclic undrained shear strength is less than the monotonic value at the same strain rate. The severity of the ‘damage’ is governed by the magnitude and number of cycles, and whether the loading is one-way or two-way. Procedures to estimate design cyclic strengths are based on contour diagrams of shear strain or excess pore pressure (e.g. Andersen et al. 1988, Andersen 2015). Although this methodology is well-established and robust, it neglects the potential for regains in soil strength associated with dissipation of the excess pore pressure induced by the cyclic loads.

Ignoring the regain in strength due to dissipation of excess pore pressures can result in a conservatively low estimate of soil strength, if in practice some dissipation will occur prior to the governing load being applied. This recovery is illustrated by the model scale T-bar penetrometer test in soft kaolin clay shown in Figure 1 (Hodder et al. 2013), which involved episodes of undrained cycling interspersed with consolidation periods. Although the strength degrades within each episode, the regain from consolidation is significant. This example represents onerous cyclic loading, such as that caused by an oscillating catenary riser pipe where it touches down on the seabed. Another example in which consolidation-induced strength gain is increasingly recognised, and considered in design, is the soil strength and axial friction beneath on-bottom pipelines. Experimental and numerical modelling shows that cyclic loading as a pipe is laid on the seabed causes a loss of strength due to remoulding, but the consolidation process leads to higher friction in the long term (White et al. 2017).

Previous evidence of this behaviour has been limited to clays of low sensitivities. Natural offshore clays are typically more sensitive, which raises the question of whether the potential for strength regain is as significant in these soils. Other offshore cyclic loading scenarios are also less severe, such as one-way cyclic loading of an anchor. In this case the cyclic loads do not exceed the monotonic capacity, in contrast to the soil flow during large amplitude cycles of a T-bar, which strains the soil beyond failure. The regain in soil strength from this lower-amplitude cycling has received less attention, despite its higher relevance for most offshore design problems.

This paper addresses these knowledge gaps through an experimental study of changes in undrained shear strength from mild and severe cyclic loading and consolidation, applied via a
T-bar penetrometer.

PENETROMETER TESTS

The tests used normally consolidated kaolin clay and carbonate silt with properties given in Table 1. Both soils were consolidated from a slurry. To vary the sensitivity of the kaolin clay, two further slurry batches were prepared with the addition of a dispersant (Sodium Hexametaphosphate) and a flocculant (Sodium Polyacrylate). These additives raise the initial voids ratio of the kaolin clay during consolidation by encouraging groups of particles to coalesce together into effectively larger particles (Bergaya et al. 2006). As shown later, the concentrations were varied in the two ‘modified’ batches, which had the effect of raising the soil sensitivity from $S_t = 2.5$ (unmodified kaolin clay) to $S_t = 4.5$ and $S_t = 6.5$ (see Table 1). The experimental programme involved both single gravity and centrifuge tests. The single gravity samples were consolidated in tubes with a specific surcharge plate to accommodate the penetrometer (Suzuki 2015, Colreavy et al. 2019) whereas the centrifuge experiments were conducted at an acceleration of 150g in rectangular sample containers (Figure 2).

All samples were normally consolidated, which was achieved by self-weight consolidation for the centrifuge samples (at 150g) and by increasing the oedometric consolidation pressure to a vertical stress of $\sigma'_{v0} = 48$ kPa for the single gravity tests.

The T-bar penetrometer uses a cylindrical bar, 5 mm in diameter and 20 mm in length, connected perpendicularly to a 5 mm diameter shaft. Strain gauges are located on a thin-walled section near the base of the shaft to measure penetration and extraction resistance. Each penetrometer test involved penetration to a target depth followed by cyclic sequences, undertaken in either displacement or load control, interspersed with consolidation periods during which the T-bar was held at a fixed displacement (see Table 2 and Figure 3 for details).

The displacement controlled cycles involved moving the T-bar vertically by ± 4 or 4.5 diameters for $N = 20$ cycles, whereas the load controlled cycles were undertaken between load limits that mobilised either 0% and 75% or 25% and 75% of the intact penetration resistance (and therefore the initial undrained shear strength, $s_{u,i}$) for either $N = 20$ or 1080 cycles.

Consolidation periods, $t_c = 1$ and 2.5 hours for the silt and kaolin respectively, were included in the centrifuge cyclic loading sequences. A longer consolidation period, $t_c = 24$ hours was used in the single gravity tests. A penetration velocity, $v_p = 3$ mm/s and a loading frequency of 1 or 5 Hz were adopted for the displacement and load-controlled cycles respectively. A
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penetration velocity, \( v_p = 3 \, \text{mm/s} \) was selected to ensure that the response was primarily undrained, noting that the dimensionless group, \( v_pD/\text{ch} = 53 \) (where \( D \) is the T-bar diameter), in excess of the \( v_pD/\text{ch} > 10 \) criteria for undrained behaviour (e.g. Lehane et al. 2009, Colreavy et al. 2016). A loading frequency of 1 or 5 Hz was adopted for the load-controlled cycles to strike a balance between achieving the targeted undrained response and ensuring accurate load limit control. Undrained shear strength was calculated as \( s_u = q/N_{\text{T-bar}} \), where \( q \) is the measured penetration resistance (i.e. the gross pressure on the T-bar projected area) and \( N_{\text{T-bar}} \) is the T-bar bearing factor, taken as 10.5 (Martin and Randolph, 2006).

**EFFECT OF SOIL SENSITIVITY ON CONSOLIDATION-INDUCED STRENGTH REGAIN**

Figure 4a shows profiles of undrained shear strength, \( s_u \), with depth, \( z \), for the single gravity Type II tests in kaolin clay with different sensitivities (Tests 1-3). Adding flocculants to the kaolin clay increased the intact strength, while the remoulded strength remaining approximately constant. Sahdi et al. (2010) saw a similar response for kaolin clay with coloured dye.

Sensitivity, \( S_t \), defined as the ratio of intact to fully remoulded strength in the first cyclic episode, was \( S_t = 2.5 \) for pure kaolin but up to 6.5 for samples with additives. The corresponding undrained shear strength ratios, \( (s_{u,i}/\sigma'_{v0})_{\text{NC}} \) are 0.15 and 0.4.

After the \( t_c = 24 \) hour consolidation period between each episode, \( s_u \) increases due to dissipation of the excess pore pressure from the preceding cycles. During the first consolidation period strength increases by a factor of 2.6 - 4.6, and during the second period by a factor of 1.9 - 2.5 (Figure 4b). The greater gains are for the soils with higher sensitivity, indicating that the potential for consolidation-induced strength gain is actually higher in soils with increased sensitivity.

The changes in strength, expressed relative to the initial strength, \( s_{u,j} \), for pure kaolin and the carbonate silt show similar trends (Figure 5). The carbonate silt has a higher sensitivity, so the cyclic sequences soften more. However, the proportional gain in strength during each consolidation period is also higher for the carbonate silt.

These responses can be illustrated conceptually via the stress path of a soil element during and after cyclic remoulding for soils of low and high sensitivity (Figure 6). The elements for a low and a high sensitivity soil are assumed to start at the same state on a normal compression line (NCL) (Point O in Figure 6). Initial penetration of the T-bar induces excess pore pressures that reduce the vertical effective stress from point O to point A. Cycling the T-bar generates
additional excess pore pressure until the stress reaches the fully remoulded strength line (RSL) at point B (White and Hodder 2010; Hodder et al. 2013; Zhou et al. 2019).

The distance between the NCL and the RSL is controlled by the sensitivity, such that the fully remoulded state for a low sensitivity soil is represented by point B₁, which is at a higher vertical effective stress than for a high sensitivity soil, which is represented by point B₂. The reconsolidation phase follows a stress path shown by the κ line. After consolidation the reduction in specific volume, \( v \), and hence the increase in \( s_u \), is higher for the higher sensitivity soil (point C₂) than for the low sensitivity soil (point C₁). This analysis matches the test results, and is intended to indicate only the relative positions of the NCL and RSL. In practice, both may move due to the level of structure in the soil, as evident from the higher intact strengths in the higher sensitivity kaolin in Figure 4. However, the relative changes in strength are controlled by the relative spacing of the (normal and remoulded compression) lines, not their absolute position.

**EFFECT OF ONE-WAY CYCLIC LOADING ON SOIL STRENGTH**

The Type III tests involved penetration to 43 mm depth followed by 20 load-controlled cycles between 0.25\( q_i \) and 0.75\( q_i \), where \( q_i \) was the initial resistance at that depth. After the cycles, penetration either resumed immediately (Test 6a, Figure 7a) or after consolidation for \( t_c = 2.5 \) hours (Test 6b, Figure 7b). A reference test without cycling is also shown on Figure 7. The cycles alone have negligible effect on \( s_u \) (Figure 7a) but after consolidation there is a localised increase in \( s_u \) to ~2.2\( s_{u,i} \).

The changes in \( s_u \) from all Type III tests (Tests 6a-e, Tests 9a-e) with small-amplitude load-controlled cycles are summarised in Figure 8, alongside the large-amplitude cyclic tests (Type IV), with a sub-figure for each soil type. The following observations are made:

- The displacement-controlled cycles fully remould the soil causing a significant reduction in strength. In contrast, the load-controlled cycles cause minimal reduction in soil strength (<5%), even though each cycle mobilises 0.75\( s_{u,i} \).

- The increase in \( s_u \) after the consolidation period following each load-controlled cyclic episode is significant for both soils. Strengths of 2.1\( s_{u,i} \) and 2.5\( s_{u,i} \) are reached after the first and second consolidation periods in kaolin, compared to 2.0\( s_{u,i} \) and 2.3\( s_{u,i} \) in the carbonate silt. These post-consolidation strengths are typically 2-3 times greater than observed after the displacement-controlled cycles.
Consolidation immediately following the initial penetration (i.e. without load cycles, Test Type I) also led to a significant increase in $s_u$, to $\sim 1.9s_{u,i}$ for kaolin (Test 4) and $\sim 1.7s_{u,i}$ for carbonate silt (Test 7). However, a greater strength is measured when the consolidation period is preceded by a cyclic episode, which generates additional excess pore pressure.

Tests 10 and 11 provide further evidence of the gain in strength from combined cyclic loading and consolidation. These Type IV tests (Figure 3d) involved $N = 1,080$ cycles in a single episode (reflecting the number of cycles that might occur in a typical three-hour storm). There was no subsequent consolidation period but consolidation would have occurred concurrent with the cycles. This process caused $s_u$ to increase by 2.35 times in Test 10 (which cycled from $0.25q_i - 0.75q_i$), and by 2.9 times in Test 11 (which cycled from $0 - 0.75q_i$). These increases are slightly higher than the strength gain from three $N = 20$ cyclic episodes with intervening $t_c = 1$ hour consolidation periods.

Figure 8 highlights the range of changes in soil strength that can result from cyclic loading; the variation from fully remoulded conditions to the consolidation-induced hardening after one-way cyclic loading is a factor of 6 for kaolin clay and 11.5 for the carbonate silt.

These varying changes in strength can also be explained using conceptual stress-paths for each Test Type. For example, the first episode of one-way cyclic loading followed by consolidation (Type III) is represented in Figure 9a by the stress path O-A$_2$-B$_2$, which generated more excess pore pressure than Type I (O-A$_1$-B$_1$) as Type III involved 20 cycles of one-way loading after the initial penetration. During the subsequent consolidation, the reduction in specific volume for Type III ($\Delta v_{III}$) is higher than that for Type I ($\Delta v_{I}$), so the potential for further excess pore pressure generation (e.g. in the next T-bar pass, stress paths A$_1$-B$_1$ and A$_2$-B$_2$) is lower for Type III than Type I. Consequently, the next pass of the T-bar involves a higher vertical effective stress and soil strength for Type III (point B$_2$) than Type I (point B$_1$).

The same logic applies to Type IV; the additional cycles ($N = 1080$) generate additional pore pressure, although concurrent consolidation leads to a curved effective stress path. The pore pressure generation increases with cyclic amplitude so the reduction in specific volume is higher for Test IV-b ($0 - 0.75q_i$, $\Delta v_{IV-b}$) than Test IV-a ($0.25q_i - 0.75q_i$, $\Delta v_{IV-a}$), leading to a higher strength gain.

Figure 9b compares stress paths for a soil element subjected to 20 displacement- and load-
controlled cycles followed by the same consolidation period. The displacement-controlled cycles in Type II led to fully remoulded conditions and hence a low vertical effective stress on the RSL, whereas the load-controlled cycles generated much less excess pore pressure so the post-cyclic effective stress was higher. Hence, after consolidation the reduction in specific volume is greater for Type II ($\Delta v_{II}$) than Type III ($\Delta v_{III}$), so the remobilised soil strength is higher for Type II than Type III, whereas Figure 8b shows the opposite. However, if the very high accumulated shear strain causes the intact strength line (ISL) to migrate to the left (e.g. as per the models of Cocjin et al. 2017; Hodder et al. 2013; Zhou et al. 2019), then the effective stress and soil strength in Type II (point B$_5$) is lower than in Type III (point B$_2$).

Across all of the test types, the gain in strength relative to the initial strength is converging towards $s_u/s_{u,i} \sim 3-4$. Similar evidence is provided by other studies using variable rate and episodic penetrometer tests (Chow et al. 2019). This value is similar to the spacing ratio between the intact and remoulded or critical state lines, which controls the gain in strength predicted from these critical state-type frameworks. Parallel work for the axial friction on pipelines and shallow penetrometers shows that the undrained strength of normally-consolidated soil can rise by this ratio under episodes of sliding failure and reconsolidation (White et al. 2015, Schneider et al. 2019). The present study suggests that the same ratio may be generally applicable for bearing-type loading.

CONCLUSIONS

The changing strength of soft soil when subjected to varying episodes of cyclic loading is a topical challenge in offshore engineering.

Data from T-bar penetrometer tests involving episodes of large amplitude cyclic displacements and also novel small-amplitude load cycling highlights the effect of consolidation on strength. Large amplitude cyclic loading remoulds the soil to a minimum value, although the regain in strength due to consolidation is significant, and can surpass the strength loss from remoulding. The regain is higher in soils with higher sensitivities. Low amplitude one-way cyclic loading, mobilising a peak resistance equivalent to 75% of the initial monotonic strength, did not cause a reduction in strength, but led to a very significant increase in soil strength, to almost 2.5 times the initial monotonic strength, due to the consolidation either during or after cycling.

Consolidation around a T-bar penetrometer is relatively rapid due to the small device, which allows these new test protocols to explore changes in strength that would occur over the life of
a larger structure, due to both small and large amplitude cyclic loads.

The experimental evidence in this paper provides impetus to challenge the conventional design paradigm of discounting undrained shear strength to allow for cyclic loading. Although a consolidation period is necessary for the observed strength gains to accumulate, they can be created by relatively low-level cyclic loading and offer potentially significant benefits in available bearing capacity.

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c_h \quad \text{coefficient of consolidation}

D \quad \text{diameter of T-bar}

N \quad \text{cycle number}

N_{\text{T-bar}} \quad \text{T-bar bearing factor}

q \quad \text{measured penetration resistance}

q_i \quad \text{initial penetration resistance}

s_u \quad \text{undrained shear strength}

s_{u,i} \quad \text{initial undrained shear strength}

S_t \quad \text{soil sensitivity}

\left(\frac{s_u}{\sigma'_{v0}}\right)_{\text{NC}} \quad \text{normally consolidated undrained strength ratio}

\tau_c \quad \text{consolidation time}

v_p \quad \text{penetration velocity}

z \quad \text{soil depth}

\sigma'_{v0} \quad \text{in situ geostatic effective stress}

\gamma' \quad \text{soil effective unit weight}
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Figure 2 Experimental arrangement: (a) single gravity tests, (b) centrifuge tests.
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(a)

Time, $t$

Penetration depth, $z$

$\tau_0 = 1$ hr or $2.5$ hrs

Consolidation, $\tau_c$

Penetration

(b)

Time, $\tau$

Penetration depth, $z$

Amplitude of each cycle: $8D$ or $9D$

Consolidation, $\tau_c$

Consolidation, $\tau_c$

$\tau_0 = 1$ hr or $2.5$ hrs

Episode 1

20 cycles

Episode 2

20 cycles

Episode 3

20 cycles

(c)

Amplitude of each cycle: $0.2 - 0.75q_a$

($q_a = 38kN$)

Penetration

Consolidation, $\tau_c$

$\tau_0 = 1$ hr or $2.5$ hrs

Episode 1

20 cycles

Episode 2

20 cycles

Episode 3

20 cycles

Time, $t$

Displacement control
Load control
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Figure 3 Test procedures: (a) Type I, (b) Type II, (c) Type III, (d) Type IV
Figure 4  Single gravity test results: displacement controlled cycles (Test Type II): (a) undrained shear strength profiles, (b) Change in undrained shear strength during cycles and after consolidation periods.
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Figure 6  Effective stress path for Test type II (single gravity tests)
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Figure 7  Example results from load-controlled T-bar tests (centrifuge) in kaolin clay: (a) cyclic loading, (b) cyclic loading followed by a consolidation period
Figure 8  Comparison of changing soil strength due to load and displacement controlled loading cycles in the centrifuge tests: (a) kaolin clay, (b) carbonate silt
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Figure 9 Effective stress paths for: (a) load controlled cyclic T-bar tests, (b) displacement- and load-controlled T-bar tests.
Table 1  Soil parameters

<table>
<thead>
<tr>
<th>Soil properties</th>
<th>Kaolin clay</th>
<th>Carbonate silt</th>
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</thead>
<tbody>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.6</td>
<td>2.71</td>
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<tr>
<td>Liquid limit, LL (%)</td>
<td>61</td>
<td>67</td>
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<tr>
<td>Plastic limit, PL (%)</td>
<td>27</td>
<td>39</td>
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<tr>
<td>Compression index, $\lambda$</td>
<td>0.205</td>
<td>0.287</td>
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<tr>
<td>Swelling index, $\kappa$</td>
<td>0.044</td>
<td>0.036</td>
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<tr>
<td>Soil sensitivity, $S_t$</td>
<td>2.5, 4.5, 6.5</td>
<td>5</td>
</tr>
<tr>
<td>Normally consolidated undrained strength ratio, $(s_u/\sigma_{v0})_{NC}$</td>
<td>0.15 ($S_t = 2.5$)</td>
<td>0.385</td>
</tr>
<tr>
<td></td>
<td>0.25 ($S_t = 4.5$)*</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.40 ($S_t = 6.5$)**</td>
<td></td>
</tr>
<tr>
<td>Coefficient of horizontal consolidation, $c_h$ (m$^2$/year)</td>
<td>2.6 ($\sigma'_v = 40$ kPa)</td>
<td>8.9 ($\sigma'_v = 40$ kPa)</td>
</tr>
</tbody>
</table>

*: Batch 1 ($S_t = 4.5$): 5 kg of kaolin powder mixed with (a) flocculant: 0.1 kg Sodium Hexametaphosphate dissolved in 2.5 kg water; and (b) dispersant: 0.0005 kg Sodium Polyacrylate dissolved in 2.5 kg water.

**: Batch 2 ($S_t = 6.5$): 5 kg of kaolin powder mixed with (a) flocculant: 0.1 kg Sodium Hexametaphosphate dissolved in 2.5 kg water; and (b) dispersant: 0.00075 kg Sodium Polyacrylate dissolved in 2.5 kg water.
### Table 2 Test parameters

<table>
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<tr>
<th>Test environment</th>
<th>Soil type</th>
<th>Test no.</th>
<th>Soil sensitivity</th>
<th>Test type</th>
<th>Test parameters</th>
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<td>Test 8</td>
<td></td>
<td>Type II</td>
<td>$t_c = 1$ hr</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test 9a</td>
<td>5</td>
<td>Type III</td>
<td>$t_c = 1$ hr</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test 9b</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test 9c</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test 9d</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test 9e</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test 10</td>
<td></td>
<td>Type IV</td>
<td>Cyclic loading: $0.25q_i - 0.75q_i$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Test 11</td>
<td></td>
<td>Type IV</td>
<td>Cyclic loading: $0 - 0.75q_i$</td>
</tr>
</tbody>
</table>
Figure 1 Changing soil strength due to cyclic remoulding and reconsolidation (after Hodder et al. 2013).
Consolidation, $t_c$

Penetration

Extraction

$z$

$t_c = 1$ hr or $2.5$ hrs

Episode 1
20 cycles

Episode 2
20 cycles

Episode 3
20 cycles

Amplitude of each cycle: $8D$ or $9D$

Figure 3
Click here to access/download:Figure;OLoughlinfig03.pptx
Amplitude of each cycle: $0.25q_i - 0.75q_i$ ($f = 5\text{Hz}$)

Displacement control

Load control

Episode 1
20 cycles

Test a

Penetration

Consolidation, $t_c$

Episode 2
20 cycles

Test b

Penetration

Consolidation, $t_c$

($t_c = 1\text{ hr or 2.5 hrs}$)

Episode 3
20 cycles

Test c

Penetration

Consolidation, $t_c$

Test d

Penetration

Consolidation, $t_c$

Test e

Penetration
Amplitude of each cycle: $0.25q_i - 0.75q_i$ or $0 - 0.75q_i$ (18 mins; $f = 1$ Hz)

Penetration resistance, $q$

Displacement control
Load control

Penetration depth, $z$

Penetration
Extraction

$0.75q_i$

$0.25q_i$

$q = 0, z = 0$

1080 cycles
Figure 5

- Strength variation ratio, \( \frac{s}{s_{ui}} \)
- Cycle number, \( N \)

Episode 1: \( \times 3.4 \)  
- Kaolin (Test 5)
- Carbonate silt (Test 8)

Episode 2: \( \times 2.8 \)

Episode 3: \( \times 2.0 \)
Specific volume, $v$

Vertical effective stress, $\sigma'_v$ (log scale)

- Reconsolidation
- Remoulding

- RSL (high sensitivity)
- RSL (low sensitivity)

- URL
- NCL

Figure 6

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Penetration depth, $z$ (mm)

Undrained shear strength, $s_u$ (kPa)

Figure 7

(a)
Penetration depth, \( z \) (mm)

Undrained shear strength, \( s_u \) (kPa)

Test 5 (Type II, initial penetration)

Test 6b (Type III; one cyclic episode; one consolidation period)
Figure 8

- **Test 4:** Consolidation
  \( t_c = 2.5 \) hrs (no cycles)

- **Load-controlled cycles**
  \( t_c = 2.5 \) hrs
  - Episode 1:
    - Test 6a
  - Episode 2:
    - Test 6b
  - Episode 3:
    - Test 6c
    - Test 6d
    - Test 6e

- **Episode**
  - Episode 1: Test 6a
  - Episode 2: Test 6b
  - Episode 3: Test 6c, Test 6d, Test 6e

- **Strength variation ratio, \( s_u/s_{u,i} \)**

- **Cycle number, \( N \)**
Load-controlled cycles

N = 20

Test 7: consolidation
tc = 1 hr (no cycles)

Displacement-controlled cycles, N = 20

Test 8

Episode 1

Episode 2

Episode 3

Test 9a

Test 9b

Test 9c

Test 9d

Test 9e

N = 1080 (0 – 0.75qi), 18 mins

N = 1080 (0.25qi – 0.75qi), 18 mins

Test 10

Test 11

Test 8

N = 1080 (0 – 0.75qi), 18 mins

N = 1080 (0.25qi – 0.75qi), 18 mins

Test 10

Test 11
Specific volume, $v$

Vertical effective stress, $\sigma'_v$ (log scale)

NCL

$\lambda$

$\kappa$

URL

ISL

A1

A2

A3

A4

RSL

B1

B2

B3

B4

Test 6b or 9b (Type III, first episode): $N = 1080$ (18 mins)

$0 - 0.75q_i$

Test 11 (Type IV-b): $N = 1080$ (18 mins)

$0.25q_i - 0.75q_i$

Test 10 (Type IV-a):

$N = 1080$ (18 mins)

$0.25q_i, 0.75q_i$

Test 4 or 7 (Type I)

Test 5 or 8 (Type II):

First episode, fully remoulded

Test 4 or 7 (Type I)

Test 5 or 8 (Type II):

First episode, fully remoulded

Delta $\Delta v_i$, $\Delta v_{III}$, $\Delta v_{IV-b}$

$a$

$b$

Test 4 or 7 (Type I)

Test 5 or 8 (Type II):

First episode, fully remoulded

A1

A2

A3

A4

RSL

B1

B2

B3

B4

ISL

NCL

Click here to access/download: Figure: OLoughlinfig09.pptx