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Cyclic characterisation of low-to-medium density chalk for offshore driven pile design

T. Liu  
*University of Bristol, Bristol, UK*

R. Ahmadi-Naghadeh  
*Jönköping University, Jönköping, Sweden*

R.M. Buckley  
*University of Glasgow, Glasgow, UK*

R.A. McAdam  
*Ørsted Power Ltd, London, UK*

R.J. Jardine, K. Vinck  
*Imperial College London, London, UK*

S. Kontoe  
*University of Patras, Patras, Greece*

B.W. Byrne  
*University of Oxford, Oxford, UK*

**ABSTRACT:** Project-specific advanced laboratory testing is employed increasingly frequently in site investigations for major offshore projects. Such testing needs to focus on characterising properties under in-situ conditions, while also catering for the effects of foundation installation and subsequent service conditions, including cyclic loading. Low-to-medium density Chalk, a variable soft biomicrite, can be de-structured to soft paste under dynamic percussion or large-strain repetitive shearing, posing significant challenges and uncertainties for driven pile design. This paper draws on key outcomes from undrained cyclic triaxial test programmes on both intact chalk and dynamically de-structured (putty) chalk. The cyclic response of intact chalk resembles the fatigue behaviour of hard rocks and develops little sign of damage before sharp pore pressure reductions and brittle collapse occurs. In contrast, fully de-structured chalk develops both contractive and dilative phases, as seen with silts. The associated effective stress reductions vary systematically with the number of cycles and cyclic stress ratio. A laboratory-based global axial cyclic predictive method is proposed from the experiments and employed to predict the outcomes of field axial cyclic loading pile tests. The research provides the basis for robust cyclic design guidance for piles driven in low-to-medium density Chalk.

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1 **Introduction**

Project- and site-specific laboratory testing play key roles in modern geotechnical design, including studies for large-diameter open-ended driven piles that are employed extensively to support offshore and onshore infrastructure (Andersen, 2009). Jardine (2014, 2020) emphasised the value of advanced laboratory element tests in characterising the mechanical properties of geomaterials under both *in-situ* conditions and *in-service* conditions which could differ significantly due to the effects of foundation installation and subsequent operational and extreme loading. The outcomes from such experiments can affect the project economics profoundly, especially when ground conditions are particularly difficult to characterise, as with chalk or hard rock sites.

Chalk is a high-porosity, calcareous, variable soft rock that is found in large areas of North-West Europe and other regions worldwide (Mortimore 2012) and encountered at foundation depths of many onshore and offshore projects. Routine driven pile design is subject to great uncertainty in chalk. The major joint industry ALPACA and ALPACA Plus research projects (Jardine et al., 2023a), which were completed in 2022, aimed to address these challenges. High-quality monotonic and cyclic, axial and lateral loading field testing was conducted on 41 driven piles, with diameters ranging from 139mm to 1.8m, at the low-to-medium density St Nicholas at Wade (SNW) chalk test site in Kent (UK). The team combined these with intensive parallel laboratory testing and analysis to develop new paradigms for design.

Laterally loaded piles mobilise large volumes of chalk in their active and passive zones. Field and numerical studies reported by McAdam et al. (2023) and Pedone et al. (2023) reveal that the pile’s monotonic response is dominated by the brittle and fractured nature of the surrounding chalk mass at *in-situ* states and chalk’s high-pressure behaviour that may be developed during passive loading to failure. Piles’ response to repetitive lateral loading is governed by the cyclic response of the fractured ‘intact’ chalk and any gapping that may be formed at the chalk-pile interface. Laboratory monotonic and cyclic triaxial and/or direct simple shear tests (DSS) on high-quality intact specimens, including high pressure and constant rate of strain tests are crucial to enabling representative predictive design analyses.

In contrast, the driven piles’ axial responses are dominated by the properties of the heavily de-structured soft chalk putty annulus that forms around their shafts during installation and reconsolidates under higher stresses as the piles age in-situ. Wen et
al. (2023) and Jardine et al. (2023b) show that predicting the piles’ local load transfer mechanisms and global monotonic and cyclic responses accurately relies on monotonic and cyclic tests on samples de-structured at their natural water contents, as well as putty-steal ring shear interface tests.

Vinck et al. (2022) and Liu et al. (2023a) reported monotonic experiments on intact, low-to-medium density, Margate and Seaford chalk sampled at SNW as part of the ALPACA programme. They considered chalk behaviour under in-situ low stresses \( p' < 150\text{kPa} \) and a range of higher stresses up to \( p' = 12.8\text{MPa} \). Extensive undrained cyclic triaxial testing was also undertaken by Ahmadi-Naghadeh et al. (2022) and Liu et al. (2022) covering chalk states from intact to de-structured conditions. The key aim was to investigate the cyclic conditions relevant to lateral and axial cyclic driven pile analysis and so enable the development of ‘ALPACA’ laboratory-based predictive methods, including the axial cyclic pile design tools set out by Buckley et al. (2023).

This paper summarises the key laboratory cyclic element test findings and focuses on the interpretive frameworks applied to characterise the chalk’s cyclic behaviour and develop robust empirical correlations between cyclic pore-pressures and strengths, loading parameters and numbers of applied cycles.

2 Chalk’s response to driven pile installation and ageing

Dynamic percussion applied during impact driving of open steel piles creates de-structured, chalk ‘putty’ beneath their advancing tips which spreads and further softens around their shafts (Lord et al., 2002). Buckley et al. (2018) and Vinck (2021) observed chalk conditions immediately after driving through windows cut into open-ended tubular steel pile shafts. Site measurements indicated undrained shear strengths \( S_0 < 20\text{kPa} \) in thin annuli formed around the pile shafts of thickness close to wall thickness. Observations made after long-term ageing with exhumed piles, and in pits excavated around axially tested piles, revealed how the putty reconsolidates over time to achieve notably lower water contents and significantly greater shear strengths. Following Buckley et al. (2018), Wen et al. (2023) and Pedone et al. (2023) introduced into their t-z and 3D FE numerical analyses of piles driven in chalk three distinct zones of reconsolidated putty chalk (Zone A), intact naturally fractured chalk (Zone C) and the intermediate zone where additional fracturing is caused by pile driving (Zone B).

The key aspects associated with driven pile installation and ageing are illustrated in Figure 1, after Buckley et al. (2020), noting the development of significantly higher than in-situ driving stresses as the driven piles’ tips approach the relevant depth, followed by rapid stress degradation and relaxation as the tips pass by. Pore-water pressures rise to several MPa near the pile tips during driving but dissipate rapidly through the chalk’s fissures. Reconsolidation and in-situ ageing processes lead to the annuli formed around pile shafts becoming highly compacted, stronger, and showing low water contents.

![Figure 1. Illustration of stress states during installation, equalisation, and ageing of driven piles (Buckley et al., 2020)](image)

3 Laboratory testing procedures and programmes

3.1 Sampling and testing procedures

Buckley et al. (2018), Vinck (2021) and Vinck et al. (2022) described the SNW test site’s ground conditions and profiles. The ALPACA laboratory programme focused on CIRIA grade B2 (Lord et al. 2002) structured, very weak to weak, low-to-medium density white chalk. High quality block samples of 350×350×250mm dimension were carefully hand-trimmed on site. All visibly fissured and weathered materials were avoided. Intact specimens of 76mm height and 38mm diameter were cored for testing with a high-stability radial arm drill employing water-flushing. Vinck (2021) reported that sample preparation is particularly challenging with chalk and detailed the steps implemented to minimise sampling disturbance and ensure end-parallelism and flatness. He also noted the samples contained vertically oriented micro-fissures with 10 to 25mm spacing that cannot be identified easily by eye. Vinck et al. (2022) and Ahmadi-Naghadeh et al. (2022) provided full details of the triaxial test procedures.

Laboratory dynamic compaction was employed to de-structure intact chalk specimens in an analogous way to pile driving and produce uniform batches of chalk putty for laboratory testing. An automatic Proctor compactor applied blows at \( \approx 2s \) intervals with a 4.5kg ram and a 300mm drop height to 350-450g chalk lumps contained in a 100mm diameter annular split mould. The resulting, mainly silt-sized, putty gave 9±3kPa fall-cone undrained shear
strengths. Liu et al. (2022) employed an ‘in-mould’ consolidation procedure to form instrumented triaxial specimens from soft putty and reproduce the water contents noted in the putttified, consolidated and aged zone immediately adjacent to driven pile shafts by Buckley et al. (2018). While similar procedures can be applied to form DSS test specimens, the programme concentrated on triaxial tests, following the approach set out by Aghakouchak et al. (2015).

3.2 Testing programmes

Undrained conditions were applied in the triaxial shearing experiments on intact and fully de-structured chalk specimens. Control experiments were performed to establish the representative average monotonic undrained shear strengths ($S_u$) available for the effective stress levels considered. Table I provides an overview of the cyclic programme, tabulating the initial mean effective ($p_0'$), the ranges of mean ($q_{\text{mean}}$) and cyclic ($q_{\text{cyc}}$) deviatoric stresses. The mean and cyclic stresses are normalised by the maximum allowable deviatoric stresses ($2S_u$) and expressed in a cyclic interaction diagram, as demonstrated later. The $q_{\text{mean}}$ and $q_{\text{cyc}}$ values were chosen to populate the interaction diagram defined in the normalised $q_{\text{cyc}}/(2S_u) = q_{\text{mean}}/(2S_u)$ space.

Ahmadi-Naghadeh et al. (2022) explored the scope for natural deviations in $S_u$ of the nominally identical intact chalk specimens sub-sampled from one chalk block. Monotonic CIU tests on ‘identical’ specimens indicated representative mean $S_u$ values of $\approx 1200$ kPa under in-situ ($p_0' = 42$ kPa) conditions and $\approx 1300$ kPa under the elevated $p_0' = 342$ kPa conditions and revealed variations of $\pm 12\%$ relative to the means. Such deviations were ascribed to the distributions of micro-fissures and other natural imperfections within the specimens. Liu et al. (2022) found the chalk putty’s resistance to cyclic loading is dominated by its pre-phase transformation (PT) behaviour. The peak pre-PT $q_{\text{PT}}$ points were taken as indicating the operational monotonic shear strengths ($2S_u$) of the de-structured chalk, giving rounded $S_u$ values of 50 kPa and 100 kPa for the $p_0' = 200$ and 400 kPa tests respectively, with $S_d/p_0' = 0.25$.

Table 1. Overview of cyclic triaxial testing programme (Liu et al. 2023b)

<table>
<thead>
<tr>
<th>Material</th>
<th>$p_0'$ (kPa)</th>
<th>$S_u$ (kPa)</th>
<th>$q_{\text{mean}}$ (kPa)</th>
<th>$q_{\text{cyc}}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>42</td>
<td>1200</td>
<td>750–1575 (0.31–0.66)</td>
<td>250–1087 (0.10–0.45)</td>
</tr>
<tr>
<td>D</td>
<td>342</td>
<td>1300</td>
<td>975–1220 (0.38–0.47)</td>
<td>975–1220 (0.38–0.47)</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>50</td>
<td>-15–79 (-0.15–0.79)</td>
<td>17–75 (0.17–0.75)</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>100</td>
<td>0 (0)</td>
<td>60–150 (0.30–0.75)</td>
</tr>
</tbody>
</table>

Note: I - Intact chalk, D - De-structured chalk.

4 Interpretive framework for intact chalk subjected to cyclic loading

4.1 Cyclic failure characteristics

The intact chalk’s response differed from that of saturated sands or clays tested under comparable monotonic or cyclic loading conditions. Figure 2 presents the stress-strain trends identified from the ‘pre-shearing’, ‘creep’ and ‘first cycle’ stages of typical ‘in-situ pressure’ (ICy series) and ‘elevated pressure’ (ECy series) tests ICy-D3 ($p_0' = 42$ kPa, $q_{\text{mean}} = 1225$ kPa, $q_{\text{cyc}} = 950$ kPa) and ECy-F1 ($p_0' = 342$ kPa, $q_{\text{mean}} = 1220$ kPa, $q_{\text{cyc}} = 1220$ kPa). ICy-D3 displayed stiffness increasing with increasing deviatoric stress and strain during its initial loading stage, with an initial undrained Young’s modulus $E_u \approx 0.8$ GPa that rose markedly once $q$ exceeded 500 kPa. In contrast, ECy-F1, which was consolidated to a 300 kPa higher $p_0'$, gave $E_{u,\text{max}} = 9.6$ GPa, after which stiffness reduced with strain and $q_{\text{mean}}$. Significant strains developed over the 48-hour, constant $q_{\text{mean}}$, imposed creep pause periods. The initial stiffnesses mobilised on cycling from $q_{\text{mean}}$ were significantly higher than the monotonic test values and showed only modest changes up to the onset of failure. These stiffness variations were interpreted as reflecting micro-fissures closing over the pre-shear and creep stages.

The intact chalk’s response to high-level cyclic loading is illustrated in Figure 3 by tracking the evolution of axial strain and pore pressure with $N$ for a typical unstable test (ICy-D3). Brittle failure occurred after 181 regular cycles. The strains and pore pressure records showed little or no sign of impending instability, until $N \approx 150$. Little change was seen in either the damping ratio (which remained at around 4%) or the cyclic stiffness, until degradation set in over the last 30 cycles. The effective stress path showed no leftward drift, as is usual in tests on saturated soils. Instead, the paths remained within a tight band close to the non-tension limit until cyclic loading degraded sufficiently the cemented chalk’s internal structure for abrupt failure to occur.

Figure 2. Stress-strain response during monotonic pre-shearing, creep and the first cycle for tests ICy-D3 ($p_0' = 42$ kPa, $q_{\text{mean}} = 1225$ kPa, $q_{\text{cyc}} = 950$ kPa) and ECy-F1 ($p_0' = 342$ kPa, $q_{\text{mean}} = 1220$ kPa, $q_{\text{cyc}} = 1220$ kPa).
propagate within the matrix before coalescing into macrocracks (Cerfontaine & Collin, 2018).

4.2 Cyclic interaction diagram

The experimental outcomes allow cyclic interaction diagrams to be established. As mentioned earlier, the monotonic and cyclic test $S_u$ values of nominally identical specimens were subject to scatter of around ±12% due to individual variations in micro-structure and geometrical imperfections. Such scatter precluded applying any elaborate contouring scheme for the numbers of cycles required to cause failure under given combinations of mean ($q_{\text{mean}}$) and cyclic ($q_{\text{cyc}}$) stresses. Figure 4 includes a tentative fan of linear $N_f$ contours to indicate the main trends applying between 1 and 3000 cycles. Note that the nominal $N_f = 1$ contour line, which is plotted from $q_{\text{cyc}} = 0$, $q_{\text{mean}} = 2S_u$ to $q_{\text{cyc}} = 2S_u$, $q_{\text{mean}} = 0$, neglects any potential strain rate effects. Note that several individual failure test points deviate from the interpreted contours by up to 0.05, or 10% of the maximum $q$ applied in the $q_{\text{cyc}}(2S_u) - q_{\text{mean}}(2S_u)$ diagram. Similar variations are observed with concretes and cemented soils.

The region below the 3000-cycle contour in Figure 4 identifies a stable region within which cycling has no deleterious effect and chalk manifests nearly visco-elastic behaviour. Applying thousands of cycles within this region improves stiffness without any loss of undrained shear strength.

4.3 Interpretive framework

Rather than resembling the cyclic response of saturated granular media, the intact chalk’s behaviour appears closer to that of hard rocks, and solids such as metals, glass or concrete – where load cycling above certain threshold levels initiates microshearing or cracking around inherent micro-flaws that generate stress concentrations. The latter initiators include micro-voids, discontinuities, sharp edges or imperfections in specimen geometry. In rocks, repeated loading prompts progressive wear and shearing between grains, forming microcracks that

The triaxial test outcomes for chalk can be interpreted as ‘$S$-$N$’ curves that plot the maximum cyclic load ($q_{\text{max}} = q_{\text{mean}} + q_{\text{cyc}}$) against $\log N$ in a similar way to fatigue tests on hard rocks, metals or glass, with contours interpolated to show how the $q_{\text{max}}(2S_u)$ curves fall as $q_{\text{cyc}}/q_{\text{mean}}$ rises. Figure 5 shows contours for limiting $q_{\text{max}}(2S_u)$ values at given $q_{\text{cyc}}/q_{\text{mean}}$ ratios. The relationships plotted are compatible with the fan of $N_f$ contours proposed tentatively in Figure 4, which follow Equation 1, whose function relating to $N_f$ is independent of $q_{\text{cyc}}/q_{\text{mean}}$.

$$q_{\text{cyc}} = f(N) \cdot (2S_u - q_{\text{mean}})$$

(1)

The $f(N)$ represents for each $N_f$ line its projected intercept on the vertical axis of the interactive $q_{\text{cyc}}/(2S_u)-q_{\text{mean}}/(2S_u)$ diagram. The maximum cyclic stress can be expressed as Equation 2.

$$\frac{q_{\text{max}}}{2S_u} = f(N) \cdot \frac{1 + q_{\text{cyc}}/q_{\text{mean}}}{f(N) + q_{\text{cyc}}/q_{\text{mean}}}$$

(2)

The following Equation 3 is proposed as representing the trends identified in Figure 5. It incorporates a lower limit of 0.35 for $f(N)$ under one-way loading. The lower limit corresponds to a fatigue limit (or fatigue strength) of $q_{\text{max}}/(2S_u) = 0.52$ below which specimens could sustain cycles indefinitely under the critical $q_{\text{cyc}}/q_{\text{mean}} = 1$ one-way loading condition.

$$f(N) = 0.35 + \frac{1}{1.54 + 0.37 \times [\log_{10}(N_f)]^{2.75}}$$

(3)

Equation 3 is applicable to regular cycling with $q_{\text{cyc}}/q_{\text{mean}}$ no greater than unity. It offers a basis for empirical modelling of how the shear strength or
yielding properties of chalk diminish under such conditions. Figure 6 provides a three-dimensional representation of Equation 2 combined with Equation 3. Potential inhomogeneity and local variations are recognised as major challenges when constructing S-N curves for rocks, adding to any impact of experimental factors such as specimen size, saturation degree, anisotropy, cycling frequency and waveform (Cerfontaine & Collin, 2018).

Figure 5. Experimental data against interpretive trends for S-N plot and $q_{cy}=q_{mean}$ contour lines

![Figure 5](image)

Figure 6. Three-dimensional representation of the $q_{max}/(2S_u)$-$q_{cy}=q_{mean}$-log10(N) correlation

![Figure 6](image)

5 Cyclic characterisation of de-structured chalk to aid axial cyclic pile design

5.1 Experimental observation

The fully de-structured re-consolidated putty’s response to undrained monotonic and cyclic loading differs markedly from that of the parent intact chalk. Figure 7 demonstrates the effective stress paths followed by reconsolidated de-structured chalk under high-level cyclic loading, taking ‘putty’ tests DCy-C3 ($q_{mean} = 0\text{kPa}$, $q_{cy}=60\text{kPa}$) and DCy-D4 ($q_{mean} = 28\text{kPa}$, $q_{cy}=60\text{kPa}$) as examples. The specimens were consolidated to $p_0' = 200\text{kPa}$, pre-sheared to different $q_{mean}$ but cycled at identical $q_{cy}$. Also plotted are the zoomed-in effective stress paths of six illustrative cycles prior to, and shortly after, their failure at $N \approx N_\infty$.

The de-structured chalk samples developed significant axial straining that accelerated markedly as cyclic failure approached. The stress-strain curves fell initially in tight bands but fanned out as failure approached. The effective stress paths drifted leftward invariably as pore water pressures grew. The paths traversed the phase transformation points and slopes defined by monotonic loading, as well as the critical state slopes M. Both contractive and dilative behaviour occurred during individual cycles, as shown in Figure 7. Following the approach by Mao & Fahey (2003), best-fit lines drawn through the cyclic phase transformation points identified from the effective stress path loops of the two-way cyclic tests with $q_{mean} = 0$ indicated $(q/p)'_{PT}$ gradients of $\approx 0.54$ and 0.38 in compression and extension respectively. These gradients fell well below the monotonic phase transformation stress ratios. Systematic variations in the damping ratio and cyclic stiffness trends were also observed. Damping ratios showed maxima near failure, followed by marked post-failure reductions as cyclic strains increased.

![Figure 7](image)

5.2 Cyclic interaction diagram

The interactive effects of the $q_{mean}$ and $q_{cy}$ components are illustrated in Figure 8 in $q_{cy}(2S_u)$-$q_{mean}(2S_u)$ and $q_{cy}(p_0')$-$q_{mean}(p_0')$ space. A tentative family of $N_\infty (= 10, 30, 100, 300, 1000, 3000$ and $10000)$ contours that extends to $q_{mean}(2S_u) = -0.15$ is included to illustrate stress interaction patterns in the one- and two-way cycling regions. Nominal contours are shown as dashed straight lines over the unpopulated extension region that link the contours towards a putative lower $q_{cy}(2S_u)$ limit of 0.15.
The lower-levels, high \( N_f \), contours show less curvature and tighter spacings than those representing high-level cycling. The interactive stress diagram region below the \( N_f = 10,000 \) contour represents the stable area within which, although strains could accumulate slowly and effective stresses reduce, cyclic failure did not occur. A power-law form Equation 4 was found suitable to correlate mean effective stress drift with the number of cycles (\( \Delta p'/p_0' \)-\( N \)). The relationship can be further applied to generate shaft capacity degradation trends that match the trends identified from field cyclic pile tests, as discussed later.

\[
\Delta p' = A \times (B + \frac{q_{\text{cyc}}}{p_0'}) \times N^C
\]

where parameters \( A, B \) and \( C \) define the rate of \( p' \) degradation and the maximum cyclic stress ratio that could lead to beneficial, null, or deleterious cycling effects. The \( p_0' = 200 \) kPa tests with \( (q_{\text{cyc}} + q_{\text{mean}})/p_0' < 0.4 \) indicate \( A = -0.05 \) and \( B = -0.12 \). Best fitting for parameter \( C \) indicated \( C = 3.48xq_{\text{cyc}}/p_0' \). Figure 9 demonstrates how the above correlations and parameters provide generally good matches with the test results from this study.

5.3 Interpretive framework

The cyclic triaxial tests’ detailed, cycle-by-cycle measurements enable further interpretation and application in the laboratory test-based predictive framework for axial cyclic pile loading assessment described by Jardine (2020). Detailed observations of the mean effective stress drifts indicated that all tests showed \( \Delta p'/p_0' \) decreasing continuously against \( N_f \), although it is possible that cycling at lower levels would identify conditions under which no reduction occurred. The rates of \( \Delta p'/p_0' \) degradation depended principally on the cyclic stress ratio (\( q_{\text{cyc}}/p_0' \)). The influence of \( q_{\text{mean}}/p_0' \) was modest over the central portion of the interactive stress diagram and became more significant as \( (q_{\text{cyc}} + q_{\text{mean}})/p_0' \) exceeded 0.4 in the \( p_0' = 200 \) kPa tests.
5.4 Laboratory-based ‘global’ prediction procedure for axial cyclic pile design

Outcomes from the above cyclic triaxial tests on fully de-structured chalk enabled Buckley et al. (2023) to develop a laboratory-based simplified global approach that employs the cyclic element testing to offer predictions for ALPACA axially cyclic field pile tests. Figure 10 presents a schematic illustration of the local stress paths expected on the pile shaft under monotonic and cyclic loading to failure. The Coulomb failure criterion applies as shaft shear stress reaches static peak \( \tau_{\text{max}} \). Under cyclic loading conditions, local shaft cyclic failure initiates when the peak of the cyclic effective stress paths engages the interface failure \( \delta' \) envelope, and can be expressed with the following Equation 5.

\[
\frac{\tau_{\text{mean}}}{(\tau_{\text{ref}})_{\text{static}}} = \frac{1}{S} \left( 1 - \frac{(\Delta \sigma')}{(\sigma_{\text{rc}}')_{\text{cyclic}}} \right) - \frac{\tau_{\text{cyc}}}{(\tau_{\text{ref}})_{\text{static}}} \tag{5}
\]

in which \( S \) is defined as \( (\sigma_{\text{rc}}')_{\text{static}}/(\sigma_{\text{rc}}') \). The pile shaft shear stress can be related to the triaxial deviatoric stress changes \( (\Delta q) \) and the variations in triaxial mean effective stress \( (\Delta p') \) taken as indicators as to how \( \sigma_{\text{rc}}' \) may change close to the pile under cyclic loading. A correction factor \( T \) is introduced in Equation 6 to correlate changes in mean effective stress under triaxial conditions and vertical effective stress under simple shear conditions, which offer a better match for the kinematic boundary conditions applying at the pile shaft.

\[
\Delta p' = \frac{\Delta \sigma'}{\sigma_{\text{rc}}'} \times T \tag{6}
\]

\( T \) was found to fall between 0.50-0.64 by comparing pairs of normalised effective stress paths from (i) monotonic triaxial tests by Liu et al. (2022) and (ii) direct simple shear (DSS) tests on normally consolidated fully de-structured chalk prepared with dynamic compaction to similar void ratios. An average \( T = 0.57 \) was adopted. Equation 5 is re-arranged as:

\[
\frac{q_{\text{mean}}}{(\tau_{\text{ref}})_{\text{static}}} = \frac{1}{S} \left( 1 + \frac{(\Delta p')}{(p_0')} \times \frac{1}{T} \right) - \frac{q_{\text{cyc}}}{(\tau_{\text{ref}})_{\text{static}}} \tag{7}
\]

in which \( \Delta p'/p_0' \) can be expressed with the above Equation 4. Equation 8 can then offer preliminary ‘global’ predictions from single element tests of the failure conditions of the field cyclic pile tests.

\[
\frac{Q_{\text{mean}}}{Q_{\text{ref}}} = \frac{1}{S} \left( 1 + \frac{(\Delta p')}{(p_0')} \times \frac{1}{T} \right) - \frac{Q_{\text{cyc}}}{Q_{\text{ref}}} \tag{8}
\]

The above procedures were applied to predict the ‘global’ contours for failures with \( N_f = 10, 100 \) and 1,000 with the contours established for the ALPACA field tests. Broadly similar trends were found between the predicted and interpreted pile test contours for the LD piles \( (D = 0.508m, L_p = 10.16m) \), as shown in Figure 11, adopting parameters from the \( p_0' = 200kPa \) cyclic triaxial tests. The largest difference \( (0.1) \) was found for the \( N = 10 \) contour covering extreme two-way cycling, while the \( Q_{\text{cyc}}/Q_{\text{ref}} \) contours for \( N = 1,000 \) agree within 0.03. Comparisons for other cases are given by Buckley et al. (2023).

![Figure 10. Schematic diagram of pile shaft failure under monotonic and cyclic loading](image)

**Figure 10. Schematic diagram of pile shaft failure under monotonic and cyclic loading**

6 Summary and Conclusions

Project-specific advanced laboratory testing is employed increasingly frequently to characterise the mechanical properties of geomaterials under both in-situ conditions and in-service conditions. The significant effects of driven pile installation in high-porosity, low-to-medium density chalk, called for highly specific laboratory testing protocols to enable cyclic axial and lateral driven pile design. This paper presents the interpretive frameworks for chalk subjected to cyclic loading and sets out how the experimental outcomes may be applied to aid industrial design. The following conclusions are drawn:

1. Intact chalk fails abruptly under high-level cyclic loading and provides little indication of damage prior to failure. Its behaviour resembles the fatigue response of metals, concretes and rocks, as interpreted within the \( S-N \) response framework.
(2) Chalk putty’s response to undrained cyclic loading differs significantly from that of its parent intact chalk, developing systematic patterns of strain accumulation, effective stress path drift, variations in damping ratio and cyclic stiffness, as well as cyclic phase transformation.

(3) Chalk putty’s mean effective stress degradation can be expressed with power-law functions linked to the cyclic loading parameters.

(4) A simple ‘global’ prediction procedure proposed from the cyclic triaxial tests offered satisfactory predictions for the cyclic axial failure contours interpreted from the ALPACA field pile tests.

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