

Controlled heavy haul traffic loading as a method to remediate liquefiable soft silts

Krechowiecki-Shaw, Christopher; Royal, Alexander; Jefferson, Ian

DOI:

[10.1139/cgj-2017-0223](https://doi.org/10.1139/cgj-2017-0223)

License:

None: All rights reserved

Document Version

Peer reviewed version

Citation for published version (Harvard):

Krechowiecki-Shaw, C, Royal, A & Jefferson, I 2019, 'Controlled heavy haul traffic loading as a method to remediate liquefiable soft silts', *Canadian Geotechnical Journal*, vol. 56, no. 7, pp. 911-928.
<https://doi.org/10.1139/cgj-2017-0223>

[Link to publication on Research at Birmingham portal](#)

Publisher Rights Statement:

Final version of record to appear in Canadian Geotechnical Journal - <http://www.nrcresearchpress.com/journal/cgj>

General rights

Unless a licence is specified above, all rights (including copyright and moral rights) in this document are retained by the authors and/or the copyright holders. The express permission of the copyright holder must be obtained for any use of this material other than for purposes permitted by law.

- Users may freely distribute the URL that is used to identify this publication.
- Users may download and/or print one copy of the publication from the University of Birmingham research portal for the purpose of private study or non-commercial research.
- User may use extracts from the document in line with the concept of 'fair dealing' under the Copyright, Designs and Patents Act 1988 (?)
- Users may not further distribute the material nor use it for the purposes of commercial gain.

Where a licence is displayed above, please note the terms and conditions of the licence govern your use of this document.

When citing, please reference the published version.

Take down policy

While the University of Birmingham exercises care and attention in making items available there are rare occasions when an item has been uploaded in error or has been deemed to be commercially or otherwise sensitive.

If you believe that this is the case for this document, please contact UBIRA@lists.bham.ac.uk providing details and we will remove access to the work immediately and investigate.

1 **--Controlled heavy haul traffic loading as a method to**
2 **remediate liquefiable soft silts**

3 **C. J. Krechowiecki-Shaw, A. C. D. Royal, I. Jefferson**

4

5 **C. J. Krechowiecki-Shaw. Arup, 560 Mission Street, Suite 700, San Francisco, CA 94105, USA.**

6 **Christopher.Shaw@arup.com**

7

8 **Dr. A. C. D. Royal. University of Birmingham School of Engineering, Edgbaston, Birmingham, B15**

9 **2TT. A.C.Royal@bham.ac.uk**

10

11 **Prof. I. Jefferson. University of Birmingham School of Engineering, Edgbaston, Birmingham, B15**

12 **2TT. I.Jefferson@bham.ac.uk**

13 **Abstract**

14 Transportation of extremely large indivisible loads (10,000 to 30,000 tonnes) is becoming
15 increasingly popular to allow offsite modular construction of infrastructure for oil and gas, mining
16 and renewable energy projects in remote areas. Such exceptionally large transient loads could
17 encounter unusual geohazards; there is a risk of metastable liquefaction when crossing soft
18 alluvium, causing sudden failure, potential casualties and severe production delays. Furthermore,
19 temporary roads for these payloads are a large cost to such projects; conventionally designed
20 earthworks and/or ground improvement is often unaffordable or logistically impossible.

21

22 This laboratory study indicates the fabric can be strengthened, and the hazard reduced, if the soil is
23 subject to careful repeated loading which rearranges the initially precarious fabric through gradual
24 accumulation of plastic strains. A novel remediation technique for these temporary haul roads is
25 proposed; managed deployment of increasingly heavy haul vehicles could result in staged fabric
26 rearrangement that strengthens the soil to the point where it would be safe for the heavy vehicles
27 to use it. In so doing, a more economic temporary haul road is open to operations (coupled with
28 observation methods to ensure adequate performance throughout) and production activities are not
29 overly disrupted.

30

31 **Keywords: cyclic loading, liquefaction, temporary roads, metastable soil**

32 **List of symbols:**

- 33 A – sample representative cross-sectional area (mm^2)
- 34 A_0 – initial sample A prior to consolidation (mm^2)
- 35 A_c – sample A following anisotropic consolidation (mm^2)
- 36 e – void ratio (dimensionless)
- 37 e_0 – initial void ratio prior to consolidation (dimensionless)
- 38 e_c – void ratio after final, anisotropic consolidation stage (dimensionless)
- 39 K – coefficient of lateral earth pressure (dimensionless)
- 40 $K_{0,NC}$ – normally consolidated coefficient of lateral earth pressure at rest (dimensionless)
- 41 N – number of cycles (dimensionless)
- 42 OCR – overconsolidation ratio (dimensionless)
- 43 p' – mean normal effective stress (kPa)
- 44 PI – Plasticity Index (%) (= $LL - PL$)
- 45 PL – Plastic Limit as per BS 1377-2 (BSI, 1990a) (%)
- 46 q – deviator stress (kPa)
- 47 q_{max} – deviator stress at cycle maximum (kPa)
- 48 q_{min} – deviator stress at cycle minimum (kPa)
- 49 q_{peak} – pre-liquefaction peak (monotonic) deviator stress (kPa)
- 50 Δq – increment in deviator stress from start of shear stage (kPa)
- 51 Δq_{cyc} – peak cyclic deviator stress range (kPa) = $q_{max} - q_{min}$
- 52 Δq_{peak} – increment in deviator stress from start of (monotonic) shear to pre-liquefaction peak (kPa)
- 53 Δq_{ult} – increment to ultimate deviator stress from start of (monotonic) shear (kPa)
- 54 u – sample pore water pressure (kPa)
- 55 u_e – excess pore water pressure (kPa)
- 56 u_{max} – maximum value of u attained in a particular cycle (kPa)
- 57 u_{pl} – irrecoverable (plastic) value of u attained at the end of a particular cycle (kPa)
- 58 Δu_{pl} – increment of irrecoverable (plastic) value of u attained in a particular cycle (kPa)
- 59 ε – axial strain (dimensionless)
- 60 ε_{max} – maximum axial strain experienced in a particular cycle (dimensionless)

- 61 ϵ_{pl} – cumulative plastic axial strain in a particular cycle (dimensionless)
- 62 $\Delta\epsilon_{cyc}$ – cyclic strain range for a particular cycle (dimensionless)
- 63 $\Delta\epsilon_{el}$ – recoverable (elastic) axial strain in a particular cycle (dimensionless)
- 64 $\Delta\epsilon_{pl}$ – increment in plastic axial strain in a particular cycle (dimensionless)
- 65 σ_1 – axial total stress (kPa)
- 66 σ'_3 – total confining stress (kPa)
- 67 σ'_1 – axial effective stress (kPa)
- 68 σ'_3 – confining effective stress (kPa)

69 **1. Introduction**

70 To avoid exposure of workers to remote, potentially inhospitable sites with associated logistical and
71 quality assurance difficulties, projects in the mining, power generation, oil and gas sectors often pre-
72 fabricate modular infrastructure off-site (Mammoet, 2017). Hence, very large indivisible loads, up to
73 3000 tonnes must be transported via temporary roads on large platforms with multiple axles (e.g. 80
74 axles arranged in 40 rows; Mammoet, 2017). After transportation, the roads may have no residual
75 value; the design and construction approach must therefore focus on minimising cost. Conversely,
76 very large loads necessitate robust road foundations for overall stability, particularly where roads
77 cross soft ground. Recent silty deposits (i.e. alluvium) present the additional unusual risk of
78 metastable liquefaction at depth to this heavy haul traffic. This is a result of interaction between
79 adjacent wheel stress bulbs stressing much deeper soil than conventional traffic (Krechowiecki-Shaw
80 et al., 2017), which can be normally consolidated and prone to liquefaction under relatively small
81 disturbances. This rapid and catastrophic failure mode could suddenly topple or strand a heavy haul
82 vehicle, with risk of casualties, irreparable damage to these multimillion-dollar payloads and
83 ultimately significant production delays.

84 An observation design approach, using in-situ monitoring data to infer behavioural changes in the
85 soil and inform remedial works where necessary, would permit more economical design, reduced

86 access costs and mitigate risks. This paper demonstrates that in certain conditions, where liquefiable
87 deposits are present at relatively shallow depths (i.e. 6-10m), the action of traffic can improve the
88 resistance of soft subgrade soil; observational design could thus be used to deliver ground
89 improvement and thus substantially more reliable and affordable temporary infrastructure.

90 **2. Initiating and averting metastable liquefaction**

91 It was demonstrated by Krechowiecki-Shaw et al. (2017) that a heavy haul vehicle can apply
92 transient stresses to soil at depth, in a normally consolidated state (e.g. at 6.5m depth for the very
93 soft soil considered), with sufficient magnitude to induce localised plastic yield. Reaching yield in
94 liquefiable silty deposits is of particular concern, as yield is strain-softening. Instead of mobilising an
95 additional reserve of bearing resistance from perfect plasticity and stress redistribution (Osman and
96 Boulton, 2005; Madabhushi and Haigh, 2015), resistance can be lost. After reaching a small initiation
97 strain (in the order of 0.1% strain: Lade, 1994; Yamamuro and Lade, 1999; Wang et al., 2014) and
98 with little prior warning, unlimited or very large ground movements can occur. For example,
99 Sadrekarimi (2014) cites cases of flowslides of metastable silty sand slopes initiated by small
100 disturbances (e.g. oversteepening, construction loads) which travelled up to 2000m.

101 Metastable liquefaction takes place under undrained conditions; liquefaction is impossible in
102 drained conditions (Been and Jefferies, 1985; Lade, 1999). Plastic clays are more resistant to
103 liquefaction due to their electro-chemical activity (Andrews and Martin, 2000). For slow-moving
104 traffic loads, silts may present a worst-case liquefaction risk, being both sufficiently inactive to
105 liquefy and insufficiently permeable to drain under load.

106 Metastable liquefaction is considered highly dependent upon a precarious initial fabric arrangement
107 (Lade, 1999). Strain threshold terminology, following Díaz-Rodríguez and López-Molina (2008), is
108 useful for describing the changing behaviour of a soil and progressive loss of influence of the initial
109 fabric with strain due to restructuring (Figure 1). A liquefiable soil can be considered unique in that

110 medium-strain perturbations (small irreversible fabric rearrangement) can initiate large-strain flow
111 (complete fabric re-structuring). Investigation of undrained granular assemblies using the Discrete
112 Element Method (DEM) by Kruyt (2010) and Gu et al. (2014) suggest the way in which global shear
113 resistance is mobilised changes with increasing plastic strain, from predominantly tangential
114 interparticle forces at small strains to predominantly normal interparticle forces at large strains, with
115 the soil skeleton rearranging to form a preferential arrangement in response to the load. This
116 rearrangement also causes irreversible contraction (or dilation) of the assembly and loss (or gain) of
117 contact points per particle. In very loose DEM assemblies investigated by Gong (2008), this loss of
118 contact points can be such that static equilibrium is no longer maintained and is hypothesised to be
119 the mechanism for liquefaction.

120 Edwards et al. (2004) theorised that, due to frictional restraint, particles in a granular assembly are
121 only free to seek the lowest energy state (i.e. a denser packing) if sufficient agitation overcomes
122 energy barriers. The medium-strain threshold represents this minimum agitation, whereby minimal
123 numbers of particles are liberated. Under a larger mechanical energy input liquefaction is initiated; a
124 far larger number of particles are simultaneously liberated, triggering a chain reaction of contact
125 breakage which destabilises the soil skeleton. Metastable soils in this context may be characterised
126 as having unusually large differential between the initial and lowest possible energy state. Clearly a
127 loose initial packing is important to achieve this.

128 Contraction in undrained shear, characteristic of normally consolidated or lightly overconsolidated
129 soil, is required to trigger this type of liquefaction (Been and Jefferies, 1985; Lade, 1999). Increasing
130 overconsolidation ratio (OCR) from 1 to 2 is found to change static behaviour of Mississippi River
131 Valley silt from liquefying to strain-hardening (Wang and Luna, 2012) and significantly increase cyclic
132 resistance (Wang et al., 2016). Similar behaviour is seen in sand (Been and Jefferies, 1985) and low-
133 plasticity clay (Santagata and Germaine, 2005) when a normally consolidated and overconsolidated
134 response are compared. The reduced contraction in undrained shear and retention of lower void

135 ratio than an equivalent normally consolidated soil (after Schofield and Wroth, 1968) are expected
136 to be responsible for this behavioural change.

137 Initiation of metastable liquefaction is governed by the soil's effective stress state, occurring when a
138 certain ratio of shear to normal effective stress (i.e. q/p') is mobilised, termed the Instability Line
139 (Lade, 1994). Whilst this Instability Line remains constant for soil prepared, consolidated and
140 sheared without complex stress history (Lade, 1994; Doanh et al., 2012), applying a load-unload
141 stress history can change the angle of the Instability Line to be preferentially stronger in one
142 direction (Doanh et al., 2012). The Instability Line can therefore be considered a function of fabric
143 and a property that can be changed through changes to fabric.

144 Post-cyclic monotonic tests (Ward, 1983; Togrol and Güler, 1984; Wang et al., 2015) indicate that
145 raising pore water pressures through cyclic pre-loading results in similar monotonic shear response
146 to samples overconsolidated by removal of the same external stress. Cyclic loading can reduce the
147 post-cyclic monotonic strength if strains are high (Brown et al., 1977; Díaz-Rodríguez and
148 Santamarina, 2001; Santagata and Germaine, 2005) but can also improve strength if plastic cyclic
149 strains are sufficiently low (e.g. less than 3% in Brown et al., 1977; 0.5% in Santagata and Germaine,
150 2005). Whether this is primarily as a result of changes to the stress state or beneficial fabric
151 rearrangement is unclear.

152 If a soil's in-situ fabric arrangement can be altered to make the arrangement less precarious or to
153 change volumetric behaviour from contraction to dilation (i.e. behave as if overconsolidation has
154 been induced), then it may be possible to treat a liquefiable soil through mechanical application of
155 cyclic load. Developing a certain amount of irreversible fabric rearrangement is expected to be
156 necessary to effect the necessary behavioural change in the soil skeleton. Silt cyclically loaded and
157 then consolidated before monotonic shearing by Wang et al. (2014) demonstrated similar
158 behaviour; unless the cyclic yield strain (determined after Erken and Ulker, 2007) was exceeded, no
159 changes to strength from consolidation was observed. This paper presents results from triaxial

160 testing on a silt soil prepared in a metastable condition, which demonstrate medium-strain cyclic
161 pre-loading can disrupt the metastable energy state and stabilise liquefiable soil without actually
162 triggering liquefaction. Cyclic pre-loading, effected by carefully controlled traffic passages, could
163 therefore use this mechanism to stabilise deep liquefiable soils beneath heavy haul roads and thus
164 improve both economy and reliability of this temporary infrastructure.

165 **3. Testing method**

166 A series of cyclic and static triaxial tests were performed on a synthetic silt mix soil in order to
167 investigate the trigger conditions for liquefaction, threshold stresses and strains (see Figure 1) and
168 changes in behaviour induced by cyclic loading and plastic strain accumulation and so demonstrate
169 how gradual strain accumulation through cyclic load can strengthen liquefiable soil.

170 **3.1. Soil classification**

171 The soil used in these experiments (referred to as 'Silt Mix' soil) is a blend of commercially available
172 geomaterials, used to ensure consistency throughout the test programme, as follows:

- 173 • 25% (by mass) Blooma playpit sand (medium to fine, subangular to angular-grained silica
174 sand, supplied by B&Q PLC)
- 175 • 60% Silverbond M10 Silica Flour (silt-sized ground silica, supplied by Minerals Marketing)
- 176 • 15% Puraflo 50 English China Clay (low activity Kaolinite clay, supplied by Potteryworks Ltd.)

177 When mixed to form a soil of soft consistency, it is liquefiable under repeated hand pressure and
178 displays dilatancy when sheared by hand following procedures in BSI (2015). The Silt Mix had Liquid
179 Limit (*LL*) of 21-22% (determined using the Drop-Cone method of BSI, 1990), a Plastic Limit (*PL*) of
180 14-15% and a Plasticity Index (*PI*) of 7-9%, corresponding to low-plasticity Clay just above the 'A-line'
181 on a Casagrande Chart. Hydrometer tests indicate 8-12% of the Silt Mix is finer than 2 μ m. Particle
182 density tests (BSI, 1990) indicate a relative density of 2.69 \pm 0.03.

183 The low LL and fraction finer than $2\mu\text{m}$, combined with the low activity expected from the silt
184 minerals (recently crushed silica with no alteration), suggests the Silt Mix is likely to behave in a
185 'granular' manner (Andrews and Martin, 2000).

186 **3.2. Specimen preparation**

187 It is particularly important when preparing silty/sandy soils to note that the fabric generated during
188 sample preparation, e.g. whether a sample is created by compaction of moist soil or consolidation
189 from slurry, has an impact on the mechanical response (Bradshaw and Baxter, 2007). It is also
190 important to reproduce a similar stress state to recent alluvial deposits, i.e. normally consolidated at
191 reasonably low stresses. A slurry consolidation preparation technique was thus chosen, based on
192 work by Wang et al. (2011), because:

- 193 • It is possible to produce a very soft sample by changing the consolidation load;
194 reconsolidation to a normally consolidated state then requires only relatively low stresses
- 195 • Soil fabric is expected to be more representative of recent alluvial silt; a clear difference is
196 apparent when compared to a soft sample produced by compaction (Figure 2)

197 Commonly, clay samples are consolidated to the desired water content from slurries mixed at a
198 water content of 2 to 3 LL (e.g. Brown et al., 1975; O'Reilly et al., 1988; Lin and Penumadu, 2005).
199 However for predominantly silty soils this carries a risk of fines segregation. Following Wang et al.
200 (2011), the Silt Mix slurry was mixed at 1.5 LL ; this was found to be sufficiently fluid to not hold open
201 air bubbles but thick enough to resist segregation.

202 Friction between the slurry mould and slurry can produce samples of non-uniform water content.
203 Valls-Marquez et al. (2008) found lenses of wet, soft soil after consolidating clay from slurry: water
204 contents at mid-height could be 8% higher than the ends. Intra-sample variations of this magnitude
205 introduce difficulty in analysis; selecting a uniform water content to represent sample behaviour is
206 unrealistic. Furthermore, due to the Silt Mix's low PI , water content differences of over 3% between

207 ends and mid-height resulted in the sample slumping in the middle under its own weight, even
208 though the top of the sample was stiff.

209 Changes were made based upon the silt slurry consolidation method developed by Wang et al.
210 (2011), which achieved a maximum water content difference of 1.2%. A latex membrane containing
211 the soil slurry reduced friction and intra-sample water content differences. The membrane also
212 restrained the sample upon mould extraction and reduces slumping tendencies. For maximum
213 effect, the latex membrane needed to also enclose the top cap, otherwise a small quantity of slurry
214 can become trapped against the mould and provide increasing frictional resistance as the cap
215 settles. With these changes to mould design, consistency was improved to typically maintain water
216 content differences of below 1.5%, sufficient to resist necessary handling to set up triaxial tests
217 without slumping. The impact of various sleeving arrangements is shown in Figure 3. Moisture
218 contents at this stage were estimated (from consolidation volume change measurement) to be 20-
219 21%, i.e. void ratio, $e = 0.54$ to 0.56 .

220 Following Wang et al. (2011), the procedure minimised handling of the soft samples, thus minimising
221 disturbance. To avoid transporting, samples were consolidated on the triaxial cell platen using the
222 ram. A split mould, cut from a 100mm diameter PVC pipe, was used so the top cap could be fitted
223 and sealed before the mould was removed. Even with these precautions, some samples were
224 disturbed during test set-up and slumped, resulting in significant changes in to observed behaviour
225 (see Section 4.5).

226 Once extracted from the mould, samples were consolidated anisotropically to reflect at-rest
227 conditions at depth in a young deposit. The deviator stress applied during consolidation is also
228 expected to increase risk of liquefaction, as the stress increment required to reach the Instability
229 Line is reduced (Doanh et al., 2012). No backpressure saturation stages were needed as all samples
230 achieved a B value of 0.95 or greater. During consolidation, a backpressure of 100kPa was
231 maintained such that negative excess pressures could be tolerated without cavitation (Head, 1986).

232 The normally consolidated lateral earth pressure coefficient at rest ($K_{0,NC}$) was determined by
233 consolidating samples by matching the axial strain to the volume change to maintain a constant
234 area, as described in Head (1986), and a value of $K = 0.45$ found. For expediency of the testing
235 program, most tests were consolidated anisotropically to this stress state through two stages; an
236 initial isotropic stage to achieve the desired effective confining pressure, followed by a gradual
237 increase in deviator stress. Undrained static tests consolidated in this manner were found to be
238 acceptably similar to those consolidated under explicit zero lateral strain conditions, i.e. both
239 produced liquefaction at 0.1% to 0.3% strain following a deviator stress increment of 23-29kPa and
240 recovery after 1.5% to 3% strain.

241 To achieve a comparable stress state between the overconsolidated samples sheared monotonically
242 and post-cyclic monotonic shear tests, two samples were overconsolidated from their anisotropic
243 normally consolidated state by lowering the cell pressure, to simulate loss of effective stress through
244 rising pore pressure.

245 Static and cyclic testing was performed using the VJTech Dynamic Triaxial Testing System (DTTS).
246 Pore water pressure was measured by a base transducer, axial load by a submersible load cell and
247 cell and back pressures were controlled by pneumatic and hydraulic Automatic Pressure Controllers
248 respectively, the latter also allowing measurement of volume change during consolidation.

249 A unique identifier was assigned to each sample tested in order to catalogue the varying test
250 conditions. They denote the test series (M for monotonic tests, CA/CB/CC for cyclic test series A, B,
251 C). For cyclic tests, additional information is provided in a second number denoting the cyclic loading
252 relative the liquefaction threshold (i.e. 154 indicates $\Delta q_{cyc}/\Delta q_{peak} = 1.54$) and a third number relating
253 either to repeated test conditions (e.g. CA-154-2 is the second repeat test) or in the case of series C
254 tests with variable cycle counts, the number of cycles (i.e. CC-154-7 has 7no. cycles of $\Delta q_{cyc}/\Delta q_{peak} =$
255 1.54).

256 Frictional restraint of the triaxial sample ends prevents radial expansion and results in non-uniform
257 stress distribution through the sample (Bishop and Green, 1965; Sheng et al., 1997) which has been
258 found to reduce the failure strain (Schofield and Wroth, 1968, Lee, 1978) and increase strain under
259 cyclic load (Lee and Vernese, 1978). Placing a lubricated membrane and disc assembly beneath the
260 sample (following Head, 1986) was not possible without lifting and disturbing the soft sample.
261 However it was possible to place a lubricated end assembly on top of the sample, thus doubling its
262 effective length with respect to end restraint (similarly to Kirkpatrick and Belshaw, 1968). A few tests
263 were conducted in this manner to investigate end restraint effects, but as the inclusion of
264 compressible grease tends to obfuscate the true stiffness behaviour at low strains (i.e. close to those
265 at which liquefaction occurs) the majority were tested without.

266 **3.3. Strain thresholds**

267 Understanding the medium-strain threshold (i.e. initiation of plasticity) is important to determine
268 the extent to which cyclic load rearranges the soil fabric, and to inform selection of appropriate
269 cyclic stresses. The test method of Hsu and Vucetic (2006) uses Direct Simple Shear apparatus to
270 determine this threshold with servo-controlled vertical stress to maintain constant volume
271 conditions (and thus records a very accurate pore pressure response). Strain-controlled cycles of
272 increasing magnitude are applied until external indications of plasticity (i.e. accumulation of pore
273 pressure) are observed; this marks the medium-strain threshold. Triaxial apparatus with
274 measurement of pore pressures in the base cannot perform this test with the same accuracy; lower
275 strains at the radially-restrained base reduce the tendency to contract (Bishop and Green, 1965) and
276 the medium-strain threshold may thus be overestimated. However measurement of deviator stress
277 (for anisotropically consolidated samples) under strain-controlled cycles shows an initiation of
278 cyclically-induced relaxation (Figure 4). This implies that initiation of plastic behaviour is not properly
279 detected by the base pore pressure transducer: global sample axial stress appears to be a more
280 useful indicator of medium-strain behaviour. This apparent medium-strain threshold between 0.01%

281 and 0.025% is in agreement with results for low plasticity silts and clays presented by Díaz-Rodríguez
282 and López-Molina (2008).

283 **3.4. Area corrections**

284 During loading, correction of the sample area with strain for estimation of a representative global
285 stress is required. The conventional volume equivalent right cylinder method is used in this study:

286 Equation 1:

$$287 \quad [1] \quad A_c = A_0 \frac{1 - \frac{\Delta V}{V_0}}{1 - \frac{\varepsilon_a}{100\%}}$$

288 Results from Sheng et al. (1997) show this correction to be acceptably similar to more sophisticated
289 methods (which use local measurements) up to around 10-15% strain for a ‘barreling’ sample with
290 frictional end restraint and hence is considered acceptable for use in this study.

291 In order to respond to feedback in cyclic tests sufficiently quickly, the DTTS’s cyclic control
292 algorithms are based on either ram displacement or axial load measurement (i.e. not stress directly).
293 During undrained, load-controlled cyclic tests, the cyclic and permanent (from anisotropic
294 consolidation) components of the deviator stress reduce with increasing strain (due to increasing
295 sample area; Equation 1). Relaxation of deviator stress is particularly significant (Figure 5); when the
296 maximum and minimum cyclic stress is maintained constant by manually adjusting the cyclic load
297 range (in line with Equation 1) strain accumulation is significantly increased, as can be observed
298 when comparing cyclic series A (CA, uncorrected) and B (CB, corrected) tests. In fact, a previously
299 stable cyclic test (25kPa, CA-96) liquefies if this correction is included (as in test CB-96-1); neglecting
300 this correction overestimates the threshold stress.

301 **3.5. Interaction with creep**

302 Creep strains are strains that continue after excess pore water pressures have dissipated and during
303 the subsequent shear stages. For undrained monotonic tests, Santagata and Germaine (2005)

304 suggested the strain rate should be 30 to 50 times faster than the final consolidation creep rate to
305 avoid creep influencing the test. Typically, final consolidation creep rates were found to be
306 0.005%/hr to 0.01%/hr; the strain rate used in monotonic tests, 1.3%/hr, is sufficiently fast to avoid
307 such interaction and still develop representative base pore water pressure readings before failure
308 (Head, 1986; BSI, 1990).

309 Tests CB-31-1, CB-31-2 and CB-31/77 experienced the smallest cyclic strains (i.e. 0.008%/cycle),
310 remaining within the elastic strain region but at sufficiently fast rates to avoid interaction (equivalent
311 to 0.43%/hr). However plastic strains still accumulated in these tests, at rates slightly faster than the
312 recorded creep rates at the end of consolidation (i.e. 2.7 to 3.5 times faster). This is reduced but not
313 eliminated by extending consolidation times (Figure 6). Pore pressure accumulation is also highly
314 dependent upon creep rates. Given the elastic nature of these strains and strong dependence upon
315 creep rates, it is likely they accumulate strain and pore pressure entirely as a result of creep
316 interaction. Laboratory estimates of strain accumulation under such low loads are therefore likely to
317 be significantly overestimated.

318 Tests CB-77 and the second part of CB-31/77 experienced an initial cyclic strain of 0.022% to 0.026%
319 (i.e. close to the volume change threshold strain), but become similarly influenced by creep rates
320 later in the test as the plastic axial strain increment per cycle, $\Delta\varepsilon_{pl}$, reduces (Figure 7). Test CB-77,
321 with the higher creep rate, reaches a near-constant plastic strain rate suggesting creep interaction.
322 Whilst the limit from Santagata and Germaine (2005) appears effective in describing the range over
323 which creep interaction is negligible, the similarity of the two tests up to cyclic plastic strains of
324 0.004% (i.e. 13 times the faster creep rate) suggest it may be conservative in this case.

325 **3.6. Summary of triaxial tests**

326 In order to simulate the 'deep' stress state considered of critical importance in Krechowiecki-Shaw et
327 al. (2017), anisotropically consolidated triaxial samples were subject to undrained cyclic deviator
328 stress pulses of varying magnitude (except when close to failure, Krechowiecki-Shaw et al., (2017),

329 found that these large vehicles did not induce significant principal stress rotation at depth).
330 Monotonic undrained tests on normally consolidated samples were used to inform selection of cyclic
331 stress levels. Cyclic loads close to the increment in deviator stress required to initiate static
332 liquefaction (Δq_{peak}) were then applied and the effects of increasing or decreasing cyclic stresses on
333 cyclic strain accumulation and instability were observed. After cyclic tests (unless cyclic strains were
334 too large), a post-cyclic monotonic undrained shear stage was included to determine whether the
335 cyclic load had improved or weakened the soil.

336 Testing conditions and outcomes are summarised in Tables 1 to 5. 'Intact' monotonic tests (i.e.
337 sheared from anisotropic normal consolidation) are summarised in Table 1, overconsolidated tests in
338 Table 2, cyclic series A, B and C tests are summarised in Tables 3, 4 and 5 respectively. 'L' in the test
339 name indicates a lubricated top assembly was used (other samples have 'standard ends'), whilst 'd'
340 indicates this sample is suspected to have been disturbed in preparation.

341 Note that the cyclic stress in tests is also normalised by the mean Δq_{peak} (26kPa), as further discussed
342 in Section 4.1. All cyclic tests use haversine waveforms of frequency 0.01Hz unless stated otherwise.
343 Figure 8 demonstrates the definitions of various cyclic stress, strain and pore pressure terms used
344 frequently in analysis.

345 **4. Experimental results**

346 ***4.1. Cyclic and static liquefaction***

347 Undrained shearing behaviour has strong connections to strain levels. Between strains of 0.1% and
348 0.3%, strain-softening liquefaction (loss of static stress or increasing cyclic plastic strain rates) is
349 initiated in both monotonic and cyclic tests (Figure 9). Liquefaction is initiated at a consistent q/p' ,
350 i.e. the Instability Line (Figure 10). Liquefaction is temporary; strength recovers after sufficient strain
351 (1.5% to 3.0%) and in monotonic tests a clear dilation during this recovery phase (over 4% strain) is
352 apparent (Figure 9, Figure 10). Increasing post-liquefaction cyclic resistance also appears to mobilise

353 dilation, illustrating a change from pore water pressure response ‘lagging’ the stress pulse (as
354 expected when finite sample permeability is considered) to ‘leading’ the stress pulse, i.e. the
355 maximum cycle stress appears to induce a reduction in pore water pressure (Figure 11). A similar
356 phenomenon of liquefaction at around 0.5% strain, followed by dilatant recovery, was previously
357 observed in monotonic undrained tests by Wang et al. (2011, 2014) on natural Mississippi River
358 Valley Silt.

359 Conventionally, the critical state strength of a soil is a sensible reference point for analysis (e.g.
360 Andersen, 1988; Frost, 2000). However, as a critical state was not observed in this study (discussed
361 further in Section 4.2), the mean pre-liquefaction peak in monotonic undrained shear is instead
362 chosen to normalise cyclic stresses (i.e. $\Delta q_{cyc}/\Delta q_{peak}$); this has the additional advantage that it focuses
363 analysis on the phenomenon of interest, i.e. metastable liquefaction.

364 Figure 9 indicates a cyclic ‘threshold stress’ between 0.77 and 0.96 Δq_{peak} (i.e. $\Delta \varepsilon_{pl}$ continually
365 reduces in the former but not the latter): cyclic and monotonic liquefaction thus occur under similar
366 stress increments. Under cyclic load, stability can be maintained for a limited number of cycles
367 above the threshold stress (e.g. tests CB-96-1 and CB-154 in Figure 9). Liquefaction is triggered only
368 when the initiation strain (strongly linked to pore water pressure and thus the Instability Line
369 condition) and threshold stress are exceeded simultaneously. This is corroborated by test CA-96;
370 gradual stress relaxation from above to below the threshold stress results in stability being
371 maintained whilst the liquefaction initiation strain is exceeded (a final strain of 1.74%; Figure 5).

372 In agreement with results from Wang et al. (2014) and Santagata and Germaine (2005),
373 overconsolidated samples no longer liquefy (Figure 12), although their ultimate strength is similar
374 (due to the low swell potential of silt). At the lower OCR tested (1.1; test M-10), behaviour is
375 continuously strain-hardening but the influence of the formerly precarious fabric is still apparent
376 through loss of stiffness between 0.1% and 2.0% strain. Figure 13 shows the effective stress state of

377 both overconsolidated samples to have crossed the Instability Line during drained swell-back (i.e.
378 stable conditions) and this may be responsible for subsequent stable behaviour.

379 Once unloaded from their insitu state, it has been observed that normally consolidated soil samples
380 will retain some 'memory' of overconsolidation when reloaded to the previous stress state and
381 require additional load (i.e. 1.5 to 4 times the previous maximum: Ladd and Foott, 1977; Overy,
382 1982; Santagata and Germaine, 2005) to regain normally consolidated behaviour. The cyclic
383 response of a sample with such a stress history (i.e. taken to anisotropic normal consolidation,
384 deviator stress then removed and reapplied) accumulates significantly less strain, as shown in Figure
385 14. The small apparent level of overconsolidation induced is sufficient to stabilise the fabric and
386 avoid liquefaction even though final stress conditions the same.

387 **4.2. Ultimate failure states**

388 Whilst potentially catastrophic for heavy haul road traffic, meta-stable liquefaction is not an ultimate
389 failure mode (Muhunthan and Worthen, 2011) and recovery of strength can follow. After liquefying,
390 undrained effective stress paths of silt samples can show pore pressure dilation (e.g. Figure 10;
391 Yamamuro and Lade, 1999; Wang and Luna, 2012). Silts able to withstand large strains and
392 significant dilation can reach a plastic critical state with constant pore pressure and deviator stress
393 (at strains over 20% in Yamamuro and Lade, 1999, and Wang and Luna, 2012). However this is not
394 the only ultimate failure mode available; dilatant soil may not experience uniform dilation but may
395 fail before the critical state through strain localisation, which is conversely brittle and strain-
396 softening (Schofield and Wroth, 1968; Nova, 2010) and thus of greater concern to the haul road
397 designer.

398 The tests presented herein all failed prematurely during dilation (at 7-11% with standard ends, 15%
399 with lubricated top) through shear banding, after which behaviour is consistently strain-softening
400 (once corrected for membrane restraint, following Head, 1986 and La Rochelle et al., 1988). This is
401 relatively common (e.g. Ward, 1983) and is expected to be due to non-uniformity, either in

402 composition or stress field (from end platen roughness), which will accelerate a strain localisation
403 (Nova, 2010). During the dilatant phase, lubricated samples were able to withstand greater strains
404 (Figure 12) and hence achieve higher ultimate strengths, particularly following cyclic loading. Cyclic
405 tests with lubricated ends also accumulated strain more slowly, particularly at large strains (Figure
406 15). Observed changes in the failure mode suggest this is due to more uniform stress conditions;
407 instead of failing either in a single diagonal shear band or a double 'X' shaped mechanism, they
408 exhibited a multi-planar, general shear failure (similarly observed by Lunne et al., 2006) implying
409 increased ability to mobilise dilatant strength.

410 As the insitu stress field is likely to be highly non-uniform, designers should exercise caution when
411 relying upon dilatant strengths. However, the transient nature of traffic loads does imply a lower risk
412 of shear banding when compared to static load; micro-drainage across the discontinuity, which
413 weakens the shear band locally, has less time to take place (Atkinson, 2000). This is supported by the
414 failure modes of tests presented herein: all standard-end monotonic tests reached shear band
415 failure at 7-11% strain but under similar cumulative strains, standard-end cyclic tests (excepting the
416 most heavily loaded; $\Delta q_{cyc}/\Delta q_{peak} = 1.92$) remained stable and only failed in subsequent monotonic
417 shear (Figure 16). Similarly, the ability of lubricated-end tests to withstand greater strains before
418 shear banding is also apparent in these post-cyclic monotonic tests, but may not reflect the risk of
419 premature insitu strain localisation failure.

420 Zhang & Garga (1997), testing uniform sands under undrained conditions, found the apparent
421 recovery of strength after liquefaction predominantly arises from large-strain triaxial end restraint:
422 lubricating sample ends reduced post-liquefaction recovery to near-negligible levels. Tests presented
423 herein differ from Zhang and Garga (1997) in this respect; Silt Mix samples start to recover strength
424 at much lower strains (1.5 to 3.0%), instead of large (10 to 20%) strains, when end restraint is
425 expected to be significant (Bishop and Green, 1965; Sheng et al., 1997). Stress-strain response and
426 cyclic strain accumulation only appears influenced by end restraint (i.e. lubricated and standard-

427 ended tests diverge) at very large strains (Figure 12; Figure 16). This greater propensity for dilatant
428 recovery at lower strains may be a function of the soil used, viz. angular, more multi-graded silt.

429 **4.3. Behaviour changes from cyclic pre-loading**

430 If cyclic loading is sufficient to trigger liquefaction, the response to subsequent monotonic loading is
431 much more dilatant and brittle than the monotonic response of intact (i.e. no cyclic stage) samples
432 (Figure 16), implying induced overconsolidation similar to that observed by Ward (1983), Togrol and
433 Güler (1984) and Li et al. (2011). It is apparent that the sum of cyclic plastic and monotonic strains
434 governs shear band initiation; dilating soil requires large localised grain movements (i.e. large
435 strains) to 'open up' a shear band (Atkinson, 2000, Zhao and Guo, 2015). Cyclically liquefied samples
436 also have a tendency to fail at lower cumulative strains in shear banding during monotonic loading
437 than intact tests (Figure 16). Large cumulative strains and pore water pressures can thus weaken soil
438 and adversely alter the failure mode, which will compromise the ability of the haul road to resist
439 subsequent heavy loads. In agreement with Brown et al. (1977), there is a range of cumulative cyclic
440 strains over which post-cyclic monotonic strength is improved without significant reduction in
441 ductility (Figure 17), i.e. between 0.3% and 2% strain.

442 Crucially, these strengthened samples no longer liquefy. This supports the assertion that the
443 Instability Line is a function of initial fabric which can be altered by load-induced fabric
444 rearrangement. A proportion of the strength increase is related to greater dilatancy (i.e. the peak
445 pore water pressure is reduced and beyond 3% strain, pore pressure dilation rates are higher)
446 implying the fabric is rearranged into a less collapsible form. Beneficial fabric rearrangement is also
447 implied by the post-cyclic strength and stiffness exceeding that of overconsolidated samples (c.f.
448 Figure 12). Thus, another factor besides the change to stress state must be influencing the improved
449 behaviour observed.

450 A minimum amount of fabric rearrangement in order to negate its precarious initial state is implied
451 by a minimum cumulative strain for cyclic load treatment. The lowest-amplitude cyclic tests in Figure

452 17 do not accumulate plastic strain in excess of the liquefaction initiation strain (0.3%) and revert to
453 intact, liquefiable behaviour during monotonic shear. As the minimum strain for stabilisation and the
454 initiation strain for static and cyclic liquefaction correspond closely, it is implied that both
455 phenomena require the same irreversible fabric rearrangement. Similarly, Wang et al. (2014) found
456 the minimum strain for strengthening following cyclic load and consolidation corresponded closely
457 with the cyclic yield strain; this large-strain yield threshold represents the point at which load-
458 induced fabric changes dominate over the initial fabric in terms of behaviour. The dependence of
459 treatment primarily upon cumulative cyclic strain (as opposed to cyclic stress) is further
460 demonstrated by Cyclic 'C' series tests: liquefiable fabric is retained with ϵ_{pl} less than 0.3% regardless
461 of the cyclic stress applied (Figure 18).

462 Conversely, if liquefaction is initiated before starting a monotonic shear stage, no stabilisation occurs
463 and the sample continues to liquefy (test CC-154-7 in Figure 18, starting from $\epsilon_{pl} = 0.4\%$). The main
464 difference between a liquefying and stable response appears to be the energy intensity used to
465 reach the initiation strain, i.e. whether the strain is imposed as a single action or through multiple
466 small actions. In the case of the former, a collapse mechanism is started and continues to propagate
467 until 'recovery' strains (1.5%) are reached, whereas in the latter case internal stability is continually
468 maintained.

469 ***4.4 Use of cyclic traffic loads as ground treatment***

470 This observed strengthening of liquefiable soil through carefully applied cyclic preloading, i.e. below
471 the liquefaction threshold stress but sufficiently large to gradually induce the necessary fabric
472 rearrangement, is expected to have practical implications for improving foundation soils beneath
473 temporary roads. Careful sequencing of heavy vehicle transits could thus greatly improve the haul
474 road serviceability. By running smaller vehicles first, the risk of liquefaction or large plastic strains
475 under subsequent larger loads is reduced. Most importantly, a vehicle which is sufficiently heavy to
476 cause liquefaction (for a given road construction depth and soil profile) may be rendered safe by

477 carefully controlled passages of heavy haul vehicles of lesser weight. This approach should be
478 coupled with an observational method which verifies this load-induced stabilisation. In this way,
479 reduced road construction depths and therefore reduced costs (both in terms of raw materials and
480 also logistics of bringing fill to site) can be attained.

481 Non-liquefying tests cross the Instability Line in effective stress space whilst remaining stable (i.e.
482 below the liquefaction threshold stress), similarly to those overconsolidated by lowering cell
483 pressure (Figure 19). Crossing the Instability Line, i.e. accumulation of a certain pore water pressure,
484 and exceeding the initiation strain for liquefaction are closely linked. This phenomenon is important
485 for monitoring purposes – monitoring strains in the order of 0.3% through settlement measurement
486 is expected to be difficult and measurement errors are likely to obscure the actual strains, but pore
487 water pressures corresponding to the Instability Line can be measured with greater reliability.

488 In order for the liquefiable soils to accumulate the necessary plastic strains during cyclic load
489 treatment, it is important that the stress bulbs developed by the lighter vehicles cover a similar
490 volume of soil (but not similar applied stresses) to those associated with the heaviest indivisible
491 loads. Thus, it would be insufficient to simply use a lighter vehicle (for example a conventional lorry
492 or large construction plant); instead a vehicle with the same wheel-base as the heaviest vehicle
493 (although only transporting a proportion of the final load) is required to transit the haul road. These
494 loads could be progressively increased until the desired treatment been achieved.

495 ***4.5. Sample disturbance***

496 Initially, samples were consolidated to a length longer than required, and the top section trimmed
497 using cheesewire; this was also intended to remove the drier and stiffer top section of the sample,
498 improving uniformity. However water content distributions post-testing indicated no significant
499 change and in some cases when the mould was removed a slight bulge at the sample top was
500 apparent. Due to the potential for such disturbance on sensitive, liquefiable samples, the trimming

501 process was discontinued. This is clearly of importance when attempting to characterise a site using
502 conventional ground investigation to obtain 'undisturbed' samples.

503 Some of these samples showing signs of disturbance were tested; there was a tendency to undergo
504 greater volume change during consolidation (possibly resulting in the drier disturbed top section).
505 Under cyclic load, lower deformation increments, particularly during the first cycle of $\Delta q_{cyc}/\Delta q_{peak} =$
506 1.92 (i.e. cyclic liquefaction; Figure 20) is clear. It is thought this initial disturbance to the sensitive
507 fabric allows a greater rearrangement into a denser, more stable form during subsequent
508 consolidation. Santagata and Germaine (2005) also observed that carefully-controlled disturbance of
509 liquefiable, normally consolidated samples followed by reconsolidation to the in situ stresses can
510 reduce or remove their liquefaction potential and increase the ultimate strength. Without careful
511 handling, soft liquefiable soil samples can be disturbed and develop stability unrepresentative of the
512 in-situ deposit.

513 In this context, field sampling of soft, normally consolidated soils, with associated sampling-induced
514 shear strains (Santagata and Germaine, 2005) and stress relief (Peters, 1988), carries a significant
515 risk of underestimating the liquefaction potential due to disturbance. Santagata and Germaine
516 (2005) found that reconsolidating to well above the in-situ stress and normalising results against
517 consolidation stress after Ladd and Foott (1977) is effective for describing intact behaviour.

518 Reconstituting samples from bulk soil following the method outlined in Section 3 may also represent
519 a cost-effective way to create representative samples. These samples will represent the worst-case
520 in terms of liquefiability as ageing or chemical bonding during preparation is unlikely to take place to
521 the same extent as in the field.

522 **5. Observational design considerations**

523 Using traffic sequencing to induce changes in the behaviour of liquefiable soils, by gradually
524 incrementing vehicle weights as demonstrated through experimental tests herein, could present

525 attractive cost benefits for temporary haul road infrastructure. This method allows reduced
526 construction depths on the basis that traffic loads can strengthen the subsoil over time; larger loads
527 which are transited later are thus rendered safe. Bearing in mind the serious consequences
528 associated with ground failure, this treatment through cyclic load must be verified; observational
529 monitoring is thus necessary.

530 The first step in an observational design approach is to estimate a cyclic liquefaction threshold
531 stress. Experimental results herein suggest this is close to the pre-liquefaction peak stress (q_{peak}); this
532 simplification avoids the need for more complex testing, however this is currently only based on
533 testing on a single soil and confirmation over a wider range of silts is necessary to provide
534 confidence in such an approximation. A sequence of undrained cyclic tests (similar to the Cyclic 'B'
535 tests) is recommended to estimate safe stress levels. From these tests, cyclic pore pressure and
536 strain accumulation relationships can be estimated and used as a basis for interpreting in-situ
537 monitoring data.

538 Monitoring of settlement at the surface, possibly supplemented by monitoring at depth (e.g. by
539 down-borehole plate and rod settlement indicators) presents a simple and inexpensive method to
540 determine when sufficient strain has been accumulated to stabilise the soil and thus when heavier
541 vehicles are safe to pass. As strains may be relatively low (below 0.5%), piezometers monitoring in-
542 situ pore water pressures, whilst potentially more expensive, may offer better insight on when
543 treatment is effective. Cyclic loading levels for treatment will also need to exceed the volume change
544 threshold. This can be determined from relatively straightforward undrained cyclic triaxial tests and
545 corroborated via pore pressure monitoring.

546 The influence depth of the heaviest vehicles needs to be considered in the surcharging or cyclic pre-
547 loading programme and associated monitoring; interaction between wheelsets can stress the
548 ground to a much greater depth than usual (Krechowiecki-Shaw et al., 2017). Ideally a vehicle of the
549 same (or larger) dimensions as those used to carry the largest load, but applying smaller stresses,

550 should be used in cyclic pre-loading. If surface settlement is monitored, a wide area, extending
551 beyond the vehicle extents, should be covered so that strain at depth can be inferred. Finite element
552 modelling of the anticipated load-strain-surface settlement response will be invaluable for
553 verification and back-analysis.

554 Careful control and monitoring of a cyclic preload programme is necessary; liquefaction can
555 propagate rapidly and once triggered, large permanent strains and a brittle response under
556 subsequent loading can be expected. Using laboratory test data, plastic strain and pore water
557 pressure accumulation rates for stresses above and below the threshold can be estimated. This can
558 be compared to in-situ monitoring to indicate whether strain is accumulating too fast (i.e. indicating
559 stresses above the actual threshold, at risk of triggering liquefaction) or too slow (i.e. that attempted
560 improvement is not having the desired effect).

561 These experiments have not considered strengthening from consolidation of residual excess pore
562 pressures following cyclic pre-loading. Wang et al. (2014, 2015) indicate it may be significant,
563 particularly following liquefaction; i.e. up to three times the intact strength for accumulation of 8.5%
564 strain (Wang et al., 2014). However undrained loading following such large strains is brittle and
565 prone to shear banding. The risk of shear banding may be reduced by reconsolidation, which reduces
566 dilatancy (Wang et al., 2015, 2016), but reconsolidated tests are still more brittle than intact tests
567 (Wang et al.; 2014, 2015). A better understanding of the effects of reconsolidation on shear banding
568 risk is required if cyclic loading and consolidation are used to develop significantly increased strength
569 through inducing large strains.

570 **6. Conclusions**

571 Metastable liquefaction of soil, characterised by a loss of strength and rapid strain accumulation, is
572 initiated with little advanced warning following a small initial disturbance (in the order of 0.1%
573 strain), but can only occur if specific initial conditions are met, i.e. a contractant soil and precarious

574 initial fabric which has a sufficiently high potential energy state to trigger a chain reaction. Cyclic
575 stresses exceeding the liquefaction threshold stress (a function of the initial fabric) must be applied
576 and coincide with the cumulative strain exceeding the liquefaction initiation strain, in order to
577 trigger liquefaction. At this point, the effective stress path crosses the Instability Line in q, p' space
578 (also a function of the initial fabric). Liquefaction is predominantly observed in loose sands and
579 normally consolidated, low plasticity silts and clays.

580 This paper demonstrates liquefaction can be averted either by statically overconsolidating the soil
581 (which may be too expensive and time-consuming as a treatment for temporary roads), or applying
582 a sequence of medium-strain cyclic loads to gradually stabilise the liquefiable fabric and induce the
583 effects of overconsolidation (i.e. unload-reload history and reduced contraction in shear).

584 Liquefaction is averted by gradually accumulating plastic strain until it exceeds the liquefaction
585 (large-strain) threshold: this can also be described by the stress state crossing the Instability Line in
586 stable (sub-threshold stress or drained) conditions. It is expected that plastic strain energy input is
587 an important factor; repetition of small loads may be sufficient to gradually mobilise small numbers
588 of particles and rearrange the fabric without precipitating a collapse that would occur if the same
589 strain was applied in a smaller number of cycles. Effective treatment requires cyclic load application
590 which exceeds the volume change (medium-strain) threshold but is below the liquefaction threshold
591 stress.

592 Careful pre-loading with lighter loads may be an effective method for treating liquefiable soil
593 beneath temporary roads carrying large indivisible loads. To be effective, pre-load vehicles need to
594 be of similar dimensions to the main loads to treat similarly sized stress bulbs. Treatment should be
595 carefully monitored to determine when improvement is effective; piezometers at depth and
596 settlement monitoring will be useful in this respect and will also assist in back-analysing predictive
597 models.

598 The silt soil tested herein failed in a brittle manner by initiation of a shear band, governed by the
599 total strain developed in undrained shear (monotonic, cyclic or combined). Whilst temporary roads
600 may be able to deal with a certain amount of strain by re-grading, it seems sensible to maintain
601 cumulative strains well below that expected to cause shear banding.

602 Disturbance during sample preparation can induce an apparent overconsolidation and was found to
603 reduce the liquefaction potential. Laboratory tests on apparently undisturbed samples recovered
604 from site may underestimate the risk of liquefaction. Reconsolidation following Ladd and Foott
605 (1977) may be necessary to recreate realistic in-situ behaviour in laboratory tests. Alternatively, the
606 slurry consolidation method detailed herein may be viable for cost-efficient investigation of
607 liquefaction risk.

608 This paper has only considered cyclic loading without drainage of excess pore pressures; strength
609 gains from consolidation after cyclic loading (observed by others) can be significant. Cyclic pre-
610 loading, allowing consolidation, could thus be a very effective method for strengthening weak
611 subsoils generally but may require sizeable strains to be developed incrementally. Further research
612 into this phenomenon, particularly the interaction with shear band failure, is recommended.

613 **References**

614 Andersen, K. H. 2009. Bearing capacity under cyclic loading - offshore, along the coast and on land.
615 The 21st Bjerrum Lecture presented in Oslo, 23 November 2007. Canadian Geotechnical Journal,
616 46(5): 513-535, 10.1139/T09-003.

617 Andrews, D. C. A., Martin, G. R. 2000. Criteria for Liquefaction of Silty Soils. Proceedings of 12th
618 World Conference on Earthquake Engineering, Auckland, New Zealand, 30th January - 4th February
619 2000.

620 Atkinson, J. H. 2000. Non-linear soil stiffness in routine design. Geotechnique 50(5), 487-508.

621 Been, K., Jefferies, M.G. 1985. A state parameter for sand. *Geotechnique* 35(2), 99-122.

622 Bishop, A.W., Green, G.E. 1965. The influence of end restraint on the compression
623 strength of a cohesionless soil. *Géotechnique*, 15(3): 243-266.

624 Bradshaw, A. S., Baxter, C. D. P. 2007. Sample Preparation of Silts for Liquefaction testing.
625 *Geotechnical Testing Journal* 30(4), Paper ID GTJ00206.

626 BSI (British Standards Institution). 1990. BS 1377-2:1990 - Methods of test for soils for civil
627 engineering purposes — Part 2: Classification tests

628 BSI (British Standards Institution). 2015. BS 5930:2015 - Code of practice for ground investigations.

629 Brown, S. F., Andersen, K. H., McElvaney, J. 1977. The Effect of Drainage on Cyclic Loading of Clay.
630 *Proceedings, 9th International Conference of Soil Mechanics and Foundation Engineering*. 2: 195-
631 200. Tokyo.

632 D'Appolonia, D. J., Poulos H. G., and Ladd C. C. (1971). *Initial Settlement of Structures on Clay*. ASCE
633 *Journal of the Soil Mechanics and Foundations Division* 97(10): 1359-1377.

634 Dean, E. T. R. 2015. Particle mechanics approach to continuum constitutive modelling. *Geotechnical*
635 *Research* 2(1), 3-34.

636 Díaz-Rodríguez, J. A., López-Molina, J. A. 2008. Strain thresholds in soil dynamics. 14th World
637 *Conference on Earthquake Engineering*. October 12-17th 2008, Beijing, China. .

638 Díaz-Rodríguez, J. A., Santamarina, J. C. 2001. Mexico city soil behavior at different strains:
639 *Observations and physical interpretation*. *Journal of Geotechnical and Geoenvironmental*
640 *Engineering* 127 (9).

641 Doanh, T., Finge, Z., and Boucq, S. 2012. Effects of Previous Deviatoric Strain Histories on the
642 Undrained Behaviour of Hostun RF Loose Sand. *ASCE Journal of Geotechnical and Geological*
643 *Engineering* 30: 697-712.

644 Edwards, S. F., Brujić, J., Makse, H. A. 2004. A basis for the statistical mechanics of granular systems.
645 *Unifying concepts in Granular Media and Glasses*. Elsevier, Amserdam.

646 Erken, A., Ulker, B. M. C. 2007. Effect of cyclic loading on monotonic shear strength of fine-grained
647 soils. *Engineering Geology* 89: 243–257.

648 Frost, M.W, Fleming, P.R, Rogers, C. D. F. 2004. Cyclic triaxial tests on clay subgrades for analytical
649 pavement design. *ASCE Journal of Transportation Engineering* 130(3), 378-386.

650 Gong, G. 2008. DEM Simulations of Drained and Undrained Behaviour (PhD thesis). University of
651 Birmingham Department of Civil Engineering.

652 Gu, X., Huang, M. Qian, J. 2014. DEM investigation on the evolution of microstructure in granular
653 soils under shearing. *Granular Matter* 16, 91-106.

654 Head, K. H. 1986. *Manual of Soil Laboratory Testing Volume 3: Effective Stress Tests*. ELE
655 International Ltd. . Pentech Press, London.

656 Heath, D. L., Shenton, M. J., Sparrow, R. W., Waters, J. M. 1972. Design of Conventional Rail Track
657 Foundations. *ICE Proceedings*, 51(2), 251-267.

658 Hsu, C., Vucetic, M. 2006. Threshold shear strain for cyclic pore water pressure in cohesive soils.
659 *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 132: 1325-1335.

660 Hu, M., O'Sullivan, C., Jardine, R.R., Jiang, M. 2010. Stress-induced anisotropy in sand under cyclic
661 loading. *Granular Matter* 12, 369-476. Springer-Verlag.

662 Kirkpatrick, W. M., Belshaw, D. J. 1968. On the Interpretation of the Triaxial Test. *Geotechnique* 18:
663 336-350.

664 Krechowiecki-Shaw, C. J., Royal, A.C., Jefferson, I., Ghataora, G. S. 2017. Routes for exceptional loads:
665 a new soil mechanics perspective. *Proceedings of the ICE: Transport* In Press.

666 Kruyt, N. P. 2010. Micromechanical study of plasticity of granular materials. *Comptes Rendus*
667 *Mecanique* 338, 596-603. Elsevier.

668 Ladd , C.C., Foott, R. 1977. New design procedure for stability of soft clays. *Journal of the*
669 *Geotechnical Engineering Division of the ASCE* 100 (GT4): 763–779

670 Lade, P. V. 1994. Instability and Liquefaction of Granular Materials. *Computers and Geotechnics* 16,
671 123-151. Elsevier.

672 Lee, K. L. 1978. End restraint effects on undrained static triaxial strength of sand. *Journal of the*
673 *Geotechnical Engineering Division of the ASCE* 104(GT6):687–704

674 Lee, K. L., Vernese, F. J. 1978. End restraint effects on cyclic triaxial strength of sand. *Journal of the*
675 *Geotechnical Engineering Division of the ASCE* 104(GT6): 705–719

676 Li, L., Dan, H., Wang, L. 2011. Undrained behavior of natural marine clay under cyclic loading. *Ocean*
677 *Engineering* 38, 1792-1805. Elsevier.

678 Lin, H., Penumadu, D. 2005. Experimental Investigation on Principal Stress Rotation in Kaolin Clay.
679 *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 131(5), 633-642.

680 Lunne, T., Berre T., Andersen, K. H., Strandvik S., Sjursen, M. (2006). Effects of sample disturbance
681 and consolidation procedures on measured shear strength of soft marine Norwegian clays. *Canadian*
682 *Geotechnical Journal* 43: 726-750.

683 Madabhushi, S.S.C., Haigh, S.K. (2015). Investigating the changing deformation mechanism beneath
684 shallow foundations. *Géotechnique* 65(8): 684–693

685 Muhunthan, B., Worthen, D. L. 2011. Critical state framework for liquefaction of fine grained soils.
686 *Engineering Geology* 117, 2-11. Elsevier.

687 Nova, R. 2010. Controllability of Geotechnical Tests and their Relationship to the Instability of Soils.
688 *Micromechanics of Failure in Granular Geomaterials*, Ed. F. Nicot & R. Wan, 1-34. ISTE and John
689 Wiley & Sons.

690 O'Reilly, M.P., Brown, S.F., Austin, G. 1988. Some Observations on the Creep Behaviour of a Silty
691 Clay. *International Conference on Rheology and Soil Mechanics, Coventry*. Ed. M. J. Keedwell, p44-
692 58. Elsevier Applied Science.

693 Osman, A. S., Boulton, M. D. (2005). Simple plasticity-based prediction of the undrained settlement
694 of shallow circular foundations on clay. *Geotechnique* 55(6), 435-447.

695 Overy, R. F. 1982. *The Behaviour of Anisotropically Consolidated Silty Clay Under Cyclic Loading (PhD*
696 *Thesis)*. University of Nottingham Department of Civil Engineering.

697 Peters, J. F. 1988. Determination of undrained shear strength of low plasticity clays. *Advanced*
698 *Triaxial Testing of Soil and Rock*, ASTM STP 977, 460-474. American Society for Testing and Materials.

699 Sadrekarimi, A. 2014. Static liquefaction-triggering analysis considering soil dilatancy. *Soils and*
700 *Foundations* 54: 955–966

701 Santagata, M., Germaine, J. T. 2005. Effect of OCR on sampling disturbance of cohesive soils and
702 evaluation of laboratory reconsolidation procedures. *Canadian Geotechnical Journal* 42: 459-474.
703 NRC Canada.

704 Schofield, A., Wroth, P. 1968. *Critical State Soil Mechanics*. McGraw Hill. London.

705 Sheng, D., Westerburg, B., Mattsson, H., Axelsson, K. 1997. Effects of End Restraint and Strain Rate in
706 Triaxial Tests. *Computers and Geotechnics* 21(3): 163-182.

707 Sitharam, T. G., Vinod, J. S. 2009. Critical state behaviour of granular materials from isotropic and
708 rebounded paths: DEM simulations. *Granular Matter* 11:33–42

709 Togrol, E., Güler, E. 1984. Effect of repeated loading on the strength of clay. *Soil Dynamics and*
710 *Earthquake Engineering* 3(4), 184-190. CML Publications.

711 Valls-Marquez, M., Chapman, D. N., Ghataora, G. S. 2008. Preparation of K0-consolidated
712 reconstituted samples in the laboratory. *International Journal of Geotechnical Engineering* 2: 343-
713 354.

714 Wang, S., Luna, R. 2012. Monotonic behavior of Mississippi River Valley Silt in Triaxial Compression.
715 *Journal of Geotechnical and Geoenvironmental Engineering* 138 (4): 516–525.

716 Wang, S., Luna, R., Onyejekwe, S. 2015. Postliquefaction behavior of low-plasticity silt at various
717 degrees of reconsolidation. *Soil Dynamics and Earthquake Engineering* 75, 259-264.

718 Wang, S., Luna, R., Onyejekwe, S. 2016. Effect of Initial Consolidation Condition on Post-Liquefaction
719 Undrained Monotonic Shear Behavior of Mississippi River Valley Silt. *Journal of Geotechnical and*
720 *Geoenvironmental Engineering* 142(2): 04015075-1 to 11.

721

722 Wang, S., Luna, R., Stephenson, R. W. 2011. A slurry consolidation approach to reconstitute low-
723 plasticity silt specimens for laboratory triaxial testing. *Geotechnical Testing Journal* 34(4), Paper ID
724 GTJ103529.

725

726 Wang, S., Onyejekwe, S., Yang, J. 2014. Threshold Strain for Postcyclic Shear Strength Change of
727 Mississippi River Valley Silt Due to Cyclic Triaxial Loading. *Journal of Testing and Evaluation* 42(1): 1-
728 9. Ward, S. J. 1983. The stability of a silty clay under repeated loading (PhD thesis). Loughborough
729 University.

730 Yamamuro, J. A., Lade, P. V. 1999. Experiments and modelling of silty sands susceptible to static
731 liquefaction. *Mechanics of Cohesive-Frictional Materials* 4, 545-564. John Wiley & Sons Ltd.

732 Zhang, H., Garga, V. K. 1997. Quasi-steady state: a real behaviour? *Canadian Geotechnical Journal*
733 34: 749–761

734 Zhao, J. & Guo, N. 2015. The interplay between anisotropy and strain localisation in granular soils: a
735 multiscale insight. *Géotechnique* 65(8), 642–656.

736 List of tables

737 [Table 1: Summary of intact monotonic tests](#)

Test	Consolidation σ'_3 (kPa)	Consolidation σ'_1 (kPa)	Liquefaction stress increment Δq_{peak} (kPa)	Ultimate strength Δq_{ult} (kPa)	Final void ratio, e	Δe_c
M-01	150	337	26	68	0.490	0.073
M-02	150	337	29	81	0.489	0.070
M-03	150	348	24	50	0.490	0.062
M-04	150	316	28	42	0.492	0.057
ML-05	150	336	26	69	0.486	0.059
M-06	75	168	11	11	0.492	0.029
M-07	50	112	7	16	0.493	0.025
M-08	200	445	38	N/A	0.460	0.111

738

739 [Table 2: Summary of overconsolidated monotonic tests](#)

Test	Consolidation σ'_3 max/final (kPa)	Consolidation σ'_1 max/final (kPa)	Δq_{ult} (kPa)	Final void ratio, e	Δe_c max/final
M-09	150/100	335/267	68	0.490	0.077/0.001
M-10	150/119	337/310	81	0.489	0.064/0.0003

740

741 Table 3: Summary of cyclic series A tests (no correction for increasing area, see Figure 5). N.b. all tests are consolidated to
 742 $\sigma'_1 = 150\text{kPa}$, $K = 0.45$.

Test	Δq_{cyc} (kPa)	$\Delta q_{cyc}/$ Δq_{peak} (kPa)	N	Final void ratio, e	Δe_c	Final ϵ_{pl} (%)	Notes
CA-96	25	0.96	200	0.482	0.061	1.7	Post-cyclic sheared
CA-154-1	40	1.54	200	0.489	0.064	5.0	Post-cyclic sheared
CA-154-2	40	1.54	200	0.474	0.063	8.0	Post-cyclic sheared
CA-154-3	40	1.54	200	0.490	0.068	7.3	
CA-154-4	40	1.54	200	0.482	0.051	7.4	Post-cyclic sheared
CAL-154-5	40	1.54	200	0.481	0.070	5.9	Post-cyclic sheared
CA-154-6	40	1.54	169	0.456	0.074	1.4	Deviator stress lost (control error) – unloaded and reloaded
CA-192-1	50	1.92	129	0.484	0.070	16.3	Shear banding during cyclic
CA-192-2d	50	1.92	135	0.476	0.076	4.6	
CA-192-3d	50	1.92	200	0.468	0.133	4.2	
CAL-192-4	50	1.92	200	0.476	0.067	11.8	
CA-192-5d	50	1.92	200	0.459	0.077	6.1	
CA-192-6	50	1.92	19	0.481	0.067	10.3	Shear banding during cyclic

743

744 Table 4: Summary of cyclic series B tests (corrected for increasing area, see Figure 5). N.b. all tests are consolidated to $\sigma'_1 =$
 745 150kPa , $K = 0.45$.

Test	Δq_{cyc} (kPa)	$\Delta q_{cyc}/$ Δq_{peak} (kPa)	N	Final void ratio, e	Δe_c	Final ϵ_{pl} (%)	Notes
CB-31-1	8	0.31	200	0.479	0.066	0.06	Post-cyclic sheared
CB-31-2	8	0.31	200	0.483	0.063	0.05	
CB-31/77	8/20	0.31/0.77	50/200	0.467	0.077	0.01/0.5	Post-cyclic sheared
CB-57	15	0.57	200	0.473	0.060	0.3	Post-cyclic sheared
CB-57/77	15	0.57	670/500	0.474	0.075	0.1/1.0	0.1Hz cyclic frequency, Post-cyclic sheared
CB-77-1	20	0.77	200	0.460	0.057	0.7	Post-cyclic sheared
CB-77-2	20	0.77	500	0.474	0.108	1.1	0.1Hz cyclic frequency
CB-96-1	25	0.96	200	0.482	0.053	8.7	Post-cyclic sheared
CB-96-2	25	0.96	840	0.478	0.068	7.4	0.1Hz cyclic frequency
CB-154		0.154	100	0.466	0.044	9.4	
CB-173	45	0.173	200	0.473	0.050	15.5	

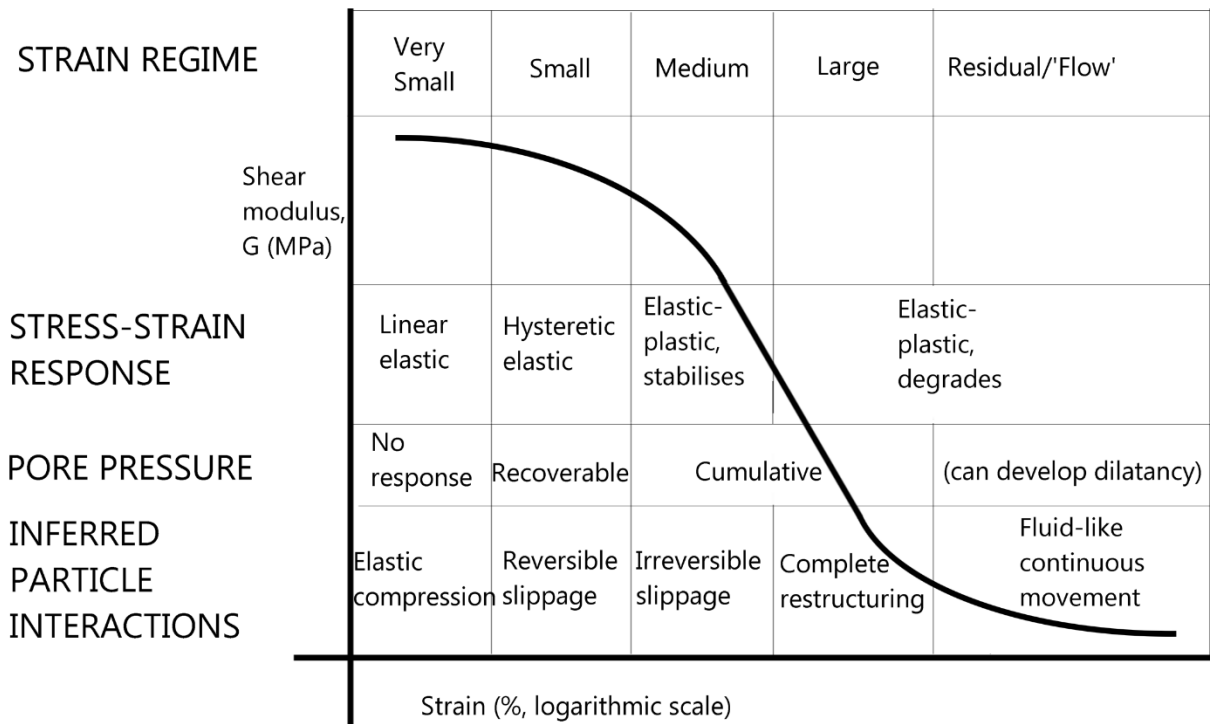
746

747 Table 5: Summary of cyclic series C tests (corrected for increasing area, see Figure 5). N.b. all tests are consolidated to $\sigma'_1 =$
 748 150kPa , $K = 0.45$.

Test	Δq_{cyc} (kPa)	$\Delta q_{cyc} / \Delta q_{peak}$ (kPa)	N	Final void ratio, e	Δe_c	Final ϵ_{pl} (%)	Notes
CC-154-1	40	1.54	1	0.480	0.064	0.2	Post-cyclic sheared
CC-154-7	40	1.54	7	0.468	0.073	0.4	0.1Hz cyclic frequency, post-cyclic sheared
CC-77-10	20	0.77	10	0.479	0.072	0.1	Post-cyclic sheared

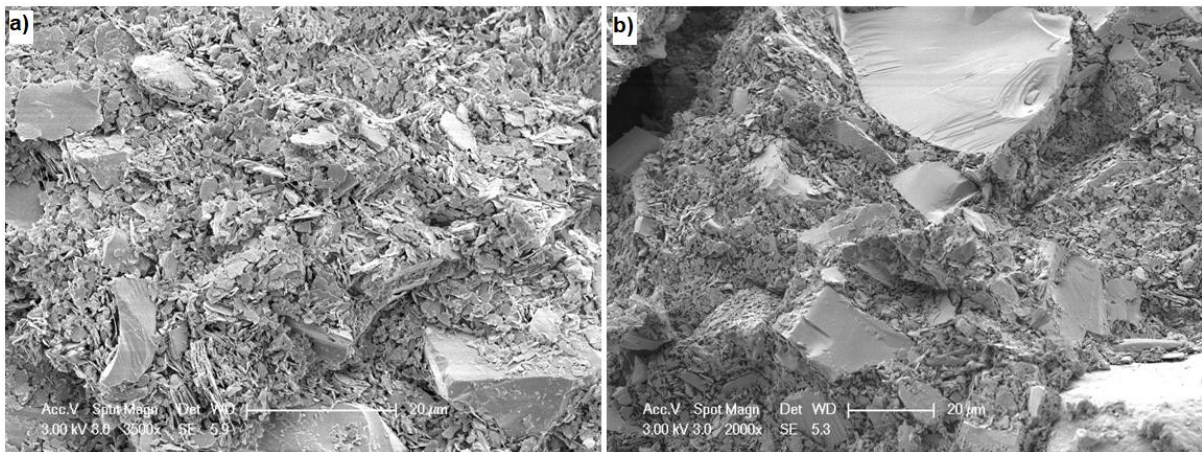
749

750 **List of figures**



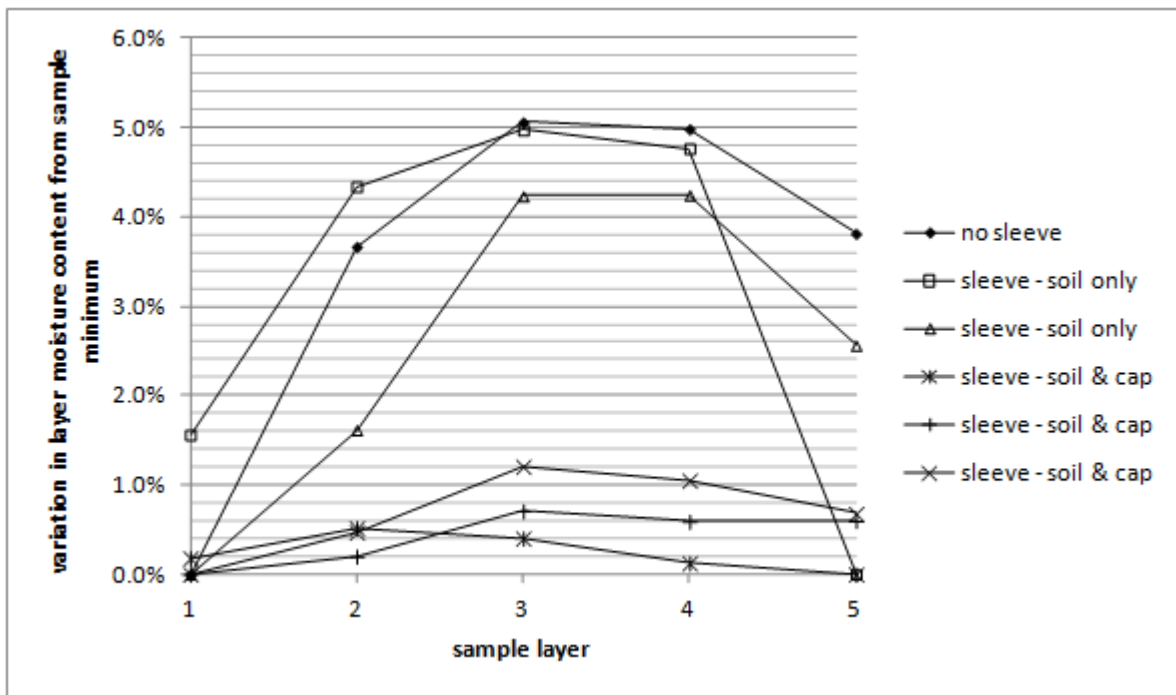
751

752 Figure 1: Strain regimes after Díaz-Rodríguez and López-Molina (2008) with possible particle
753 interactions inferred from DEM literature



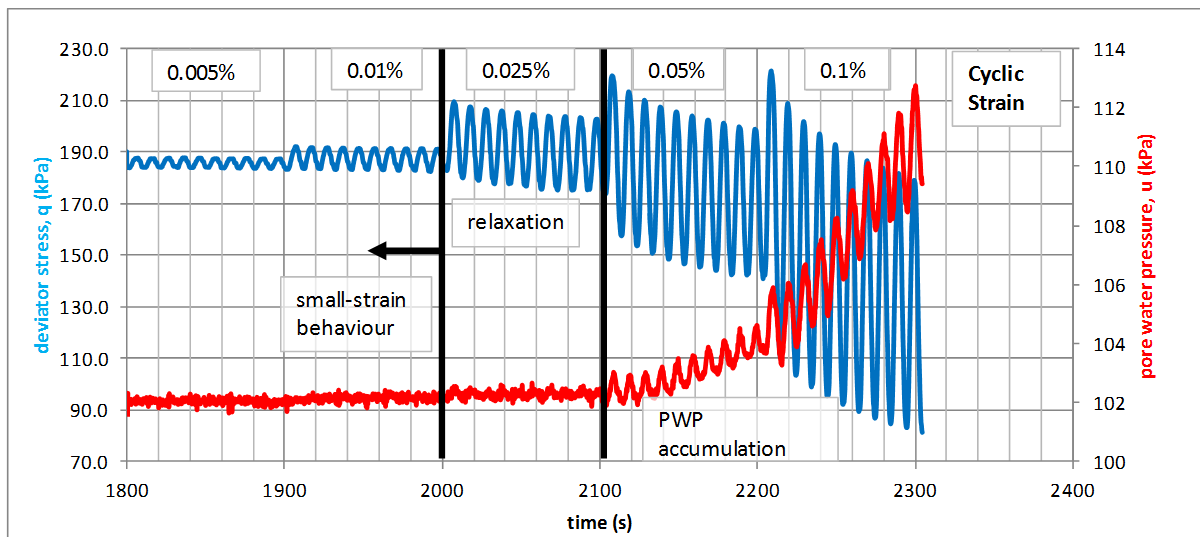
754

755 Figure 2: Electron micrograph of vertically-oriented slices from samples of the Silt Mix used in tests.
 756 a) – prepared by compaction, random particle arrangements. b) - prepared by slurry consolidation,
 757 preferential orientation.



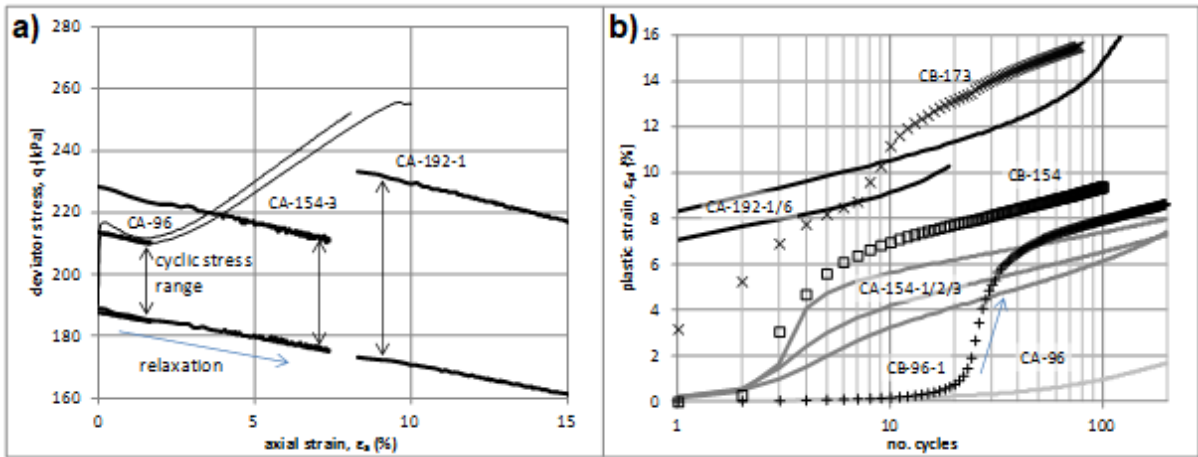
758

759 Figure 3: Intra-sample water content variation from top (layer 1) to bottom (layer 5) of 6 separate
 760 test samples with different mould arrangements; with and without a 'sleeve' comprising a 100mm
 761 diameter triaxial membrane, either fixed to the mould (soil only) or extended over the top cap (soil
 762 & cap).



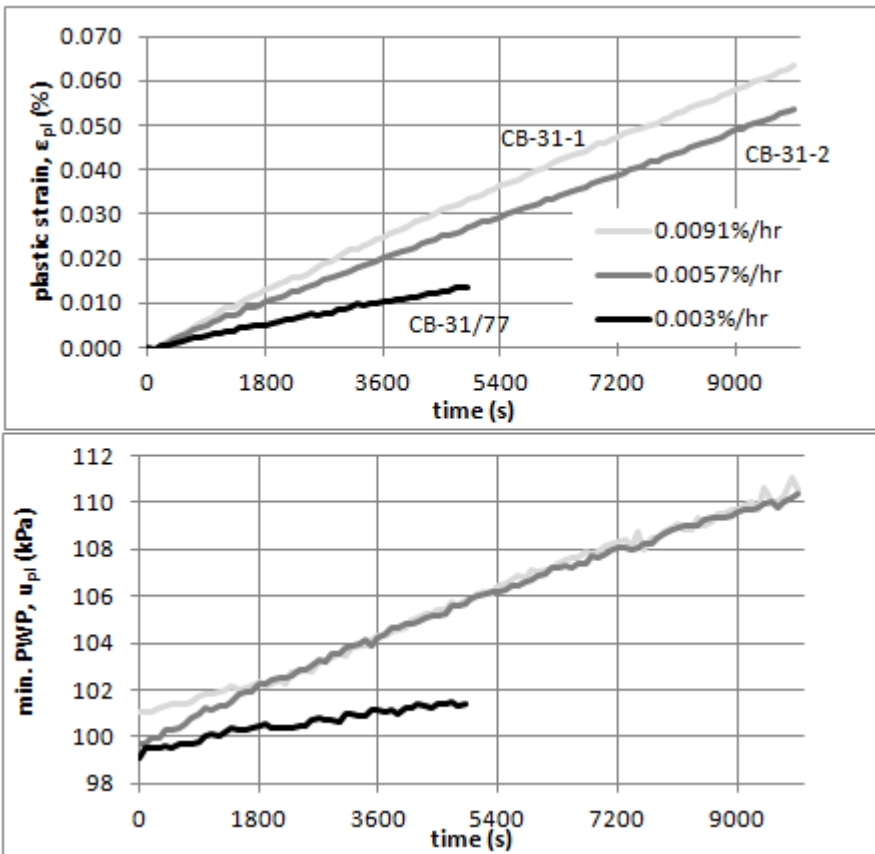
763

764 Figure 4: Strain-controlled testing to determine the initiation of medium-strain plastic behaviour by
 765 detection of soil skeleton contraction (rising pore water pressure) and plastic strain (relaxation of
 766 the deviator stress). Note differences in strain between these different behaviours, expected to be
 767 as a result of base pore pressure measurement and stress/strain non-uniformities from end
 768 restraint.



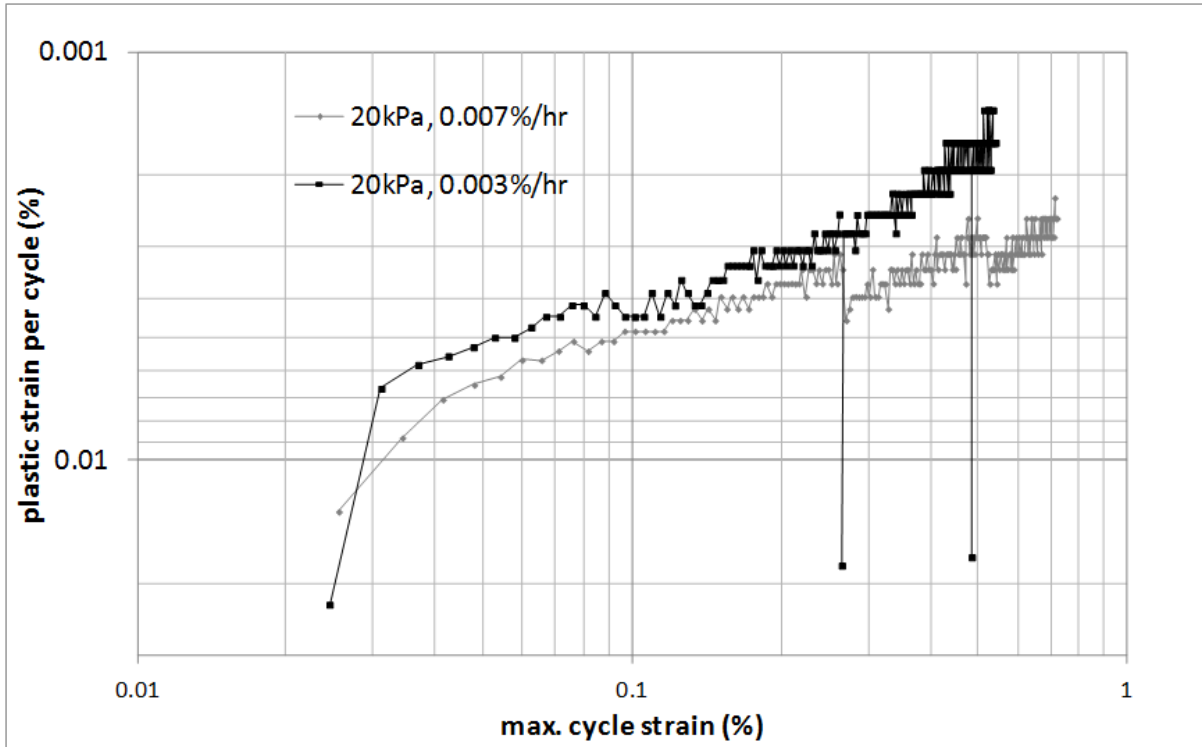
769

770 Figure 5: a) Maximum cyclic deviator stress (q_{max}) for cyclic 'A' series tests, i.e. without manual
 771 corrections applied (thick lines). Monotonic tests M-02 and M-03 (thin lines) shown for comparison;
 772 note that even the relaxation of $\sim 4\text{kPa}$ from $q_{max} = 214\text{kPa}$ to 210kPa is significant in this context. b)
 773 increased strain accumulation and lower threshold stress due to manual stress correction (i.e. CA-
 774 96-1 is stable but relaxation corrected test CB-96 liquefies) as a result of increased axial force
 775 (according to Equation 1).



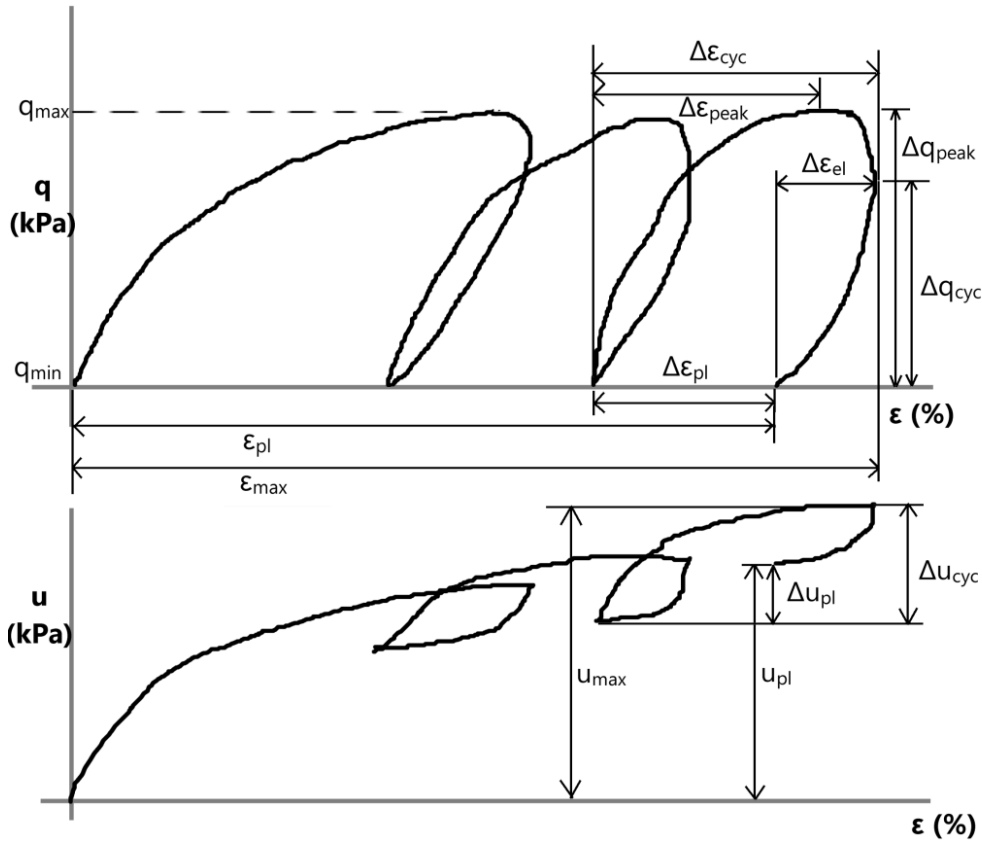
776

777 Figure 6: Plastic strain accumulation rates for low-amplitude, small-strain ($\Delta q_{cyc} = 8\text{kPa}$) cyclic tests
 778 experiencing different creep strain rates (in legend) at the end of consolidation, achieved by longer
 779 secondary consolidation periods.



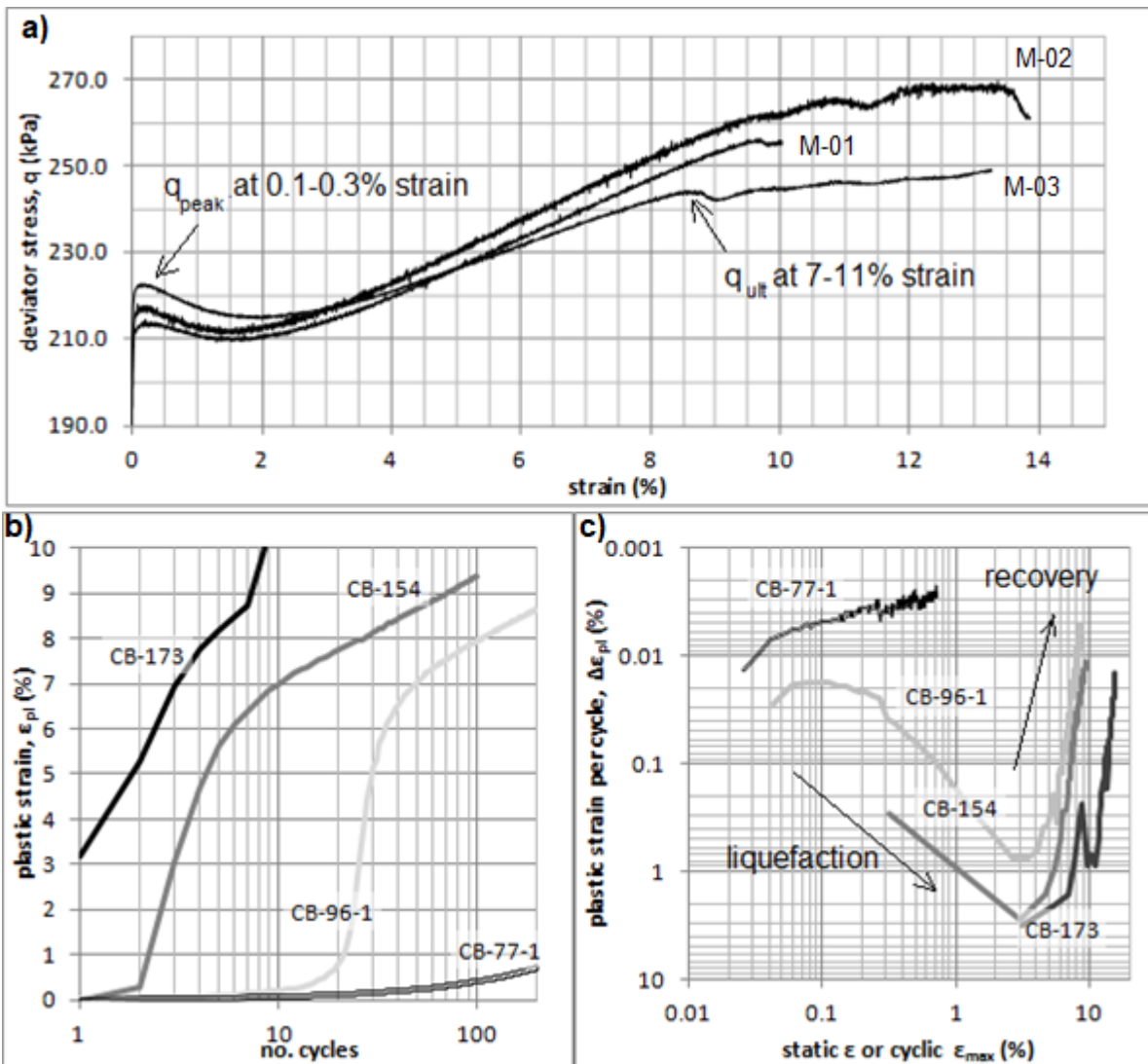
780

781 Figure 7: Comparison of medium-strain, stabilising ($\Delta q_{cyc} = 20\text{kPa}$) cyclic tests with differing final
 782 consolidation creep rates. Strains per cycle equivalent to 30x final consolidation creep rates are
 783 0.006% and 0.0024% respectively.



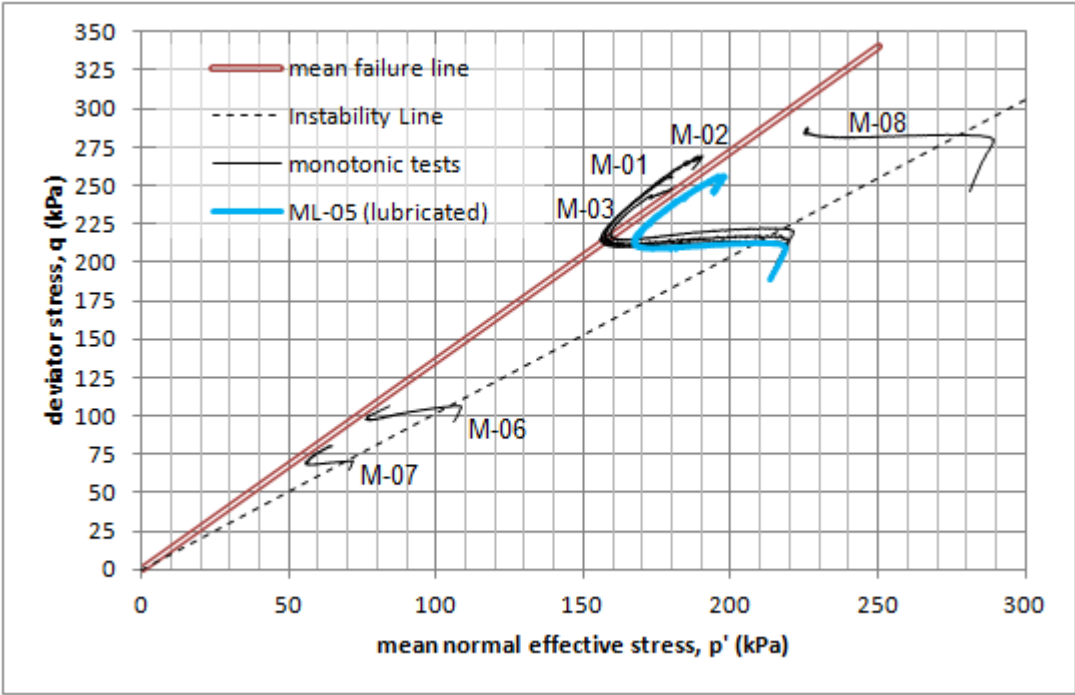
784

785 Figure 8: Definition of cyclic stress and strain symbols



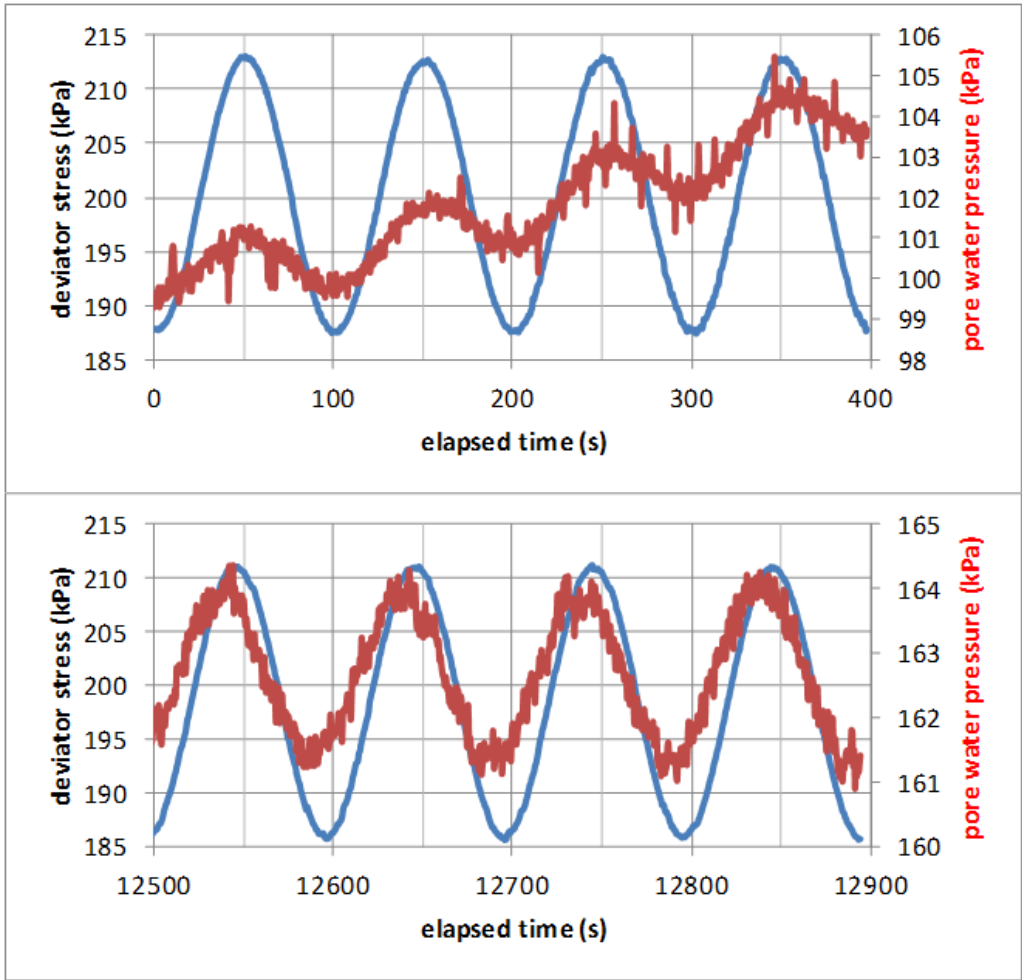
786

787 Figure 9: Static liquefaction and cyclic threshold stress effect for Silt Mix soil samples. a) Stress-strain
 788 relationship for monotonic loading. b) Accumulation of strain under varying cyclic stresses (area-
 789 corrected tests). c) Accumulation of plastic strain as a function of maximum strain achieved in the
 790 corresponding cycle; note similarity in strain levels to initiation of static liquefaction and recovery
 791 above.



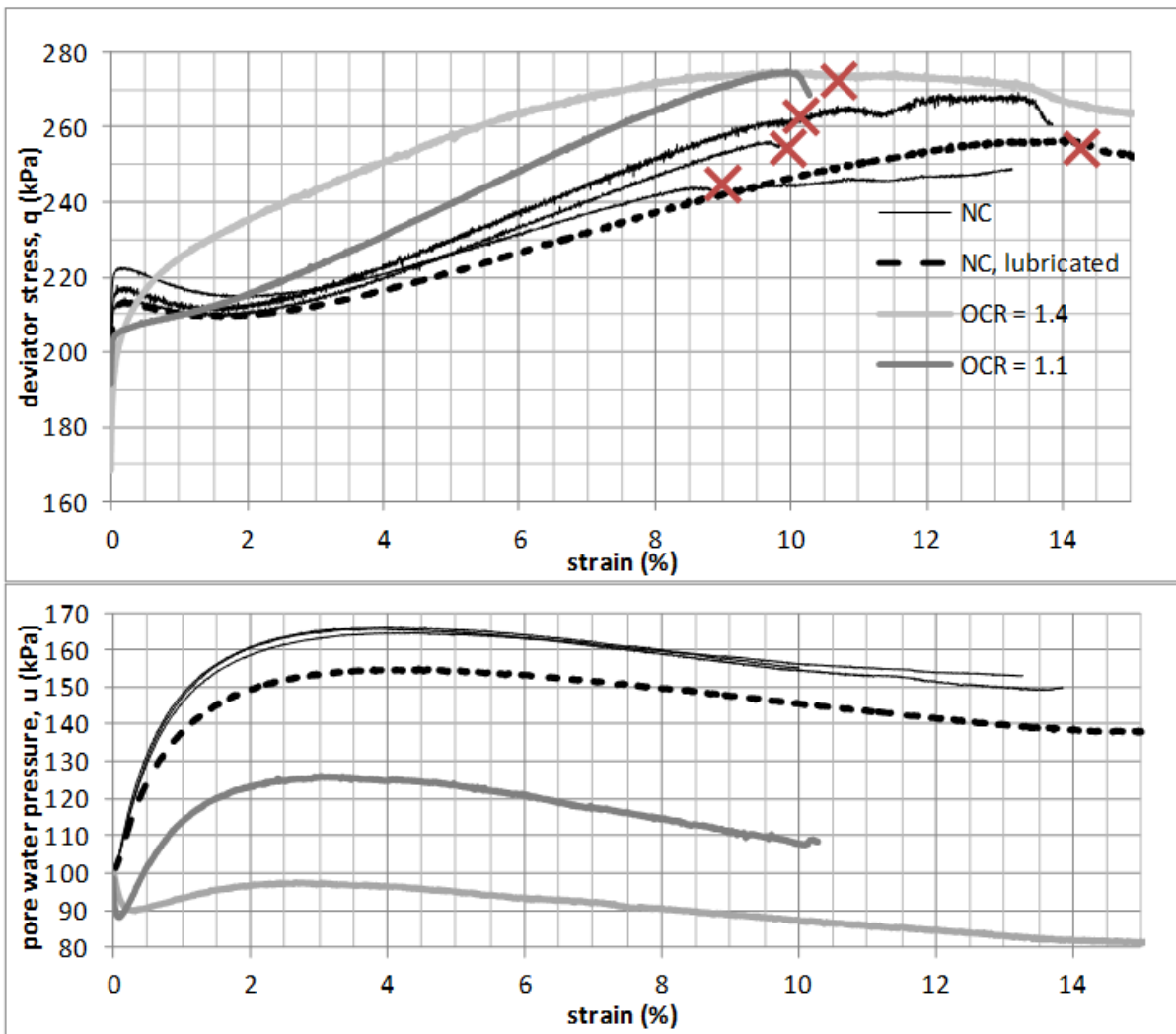
792

793 Figure 10: Undrained effective stress paths of monotonic tests indicating stress states for instability
 794 and failure



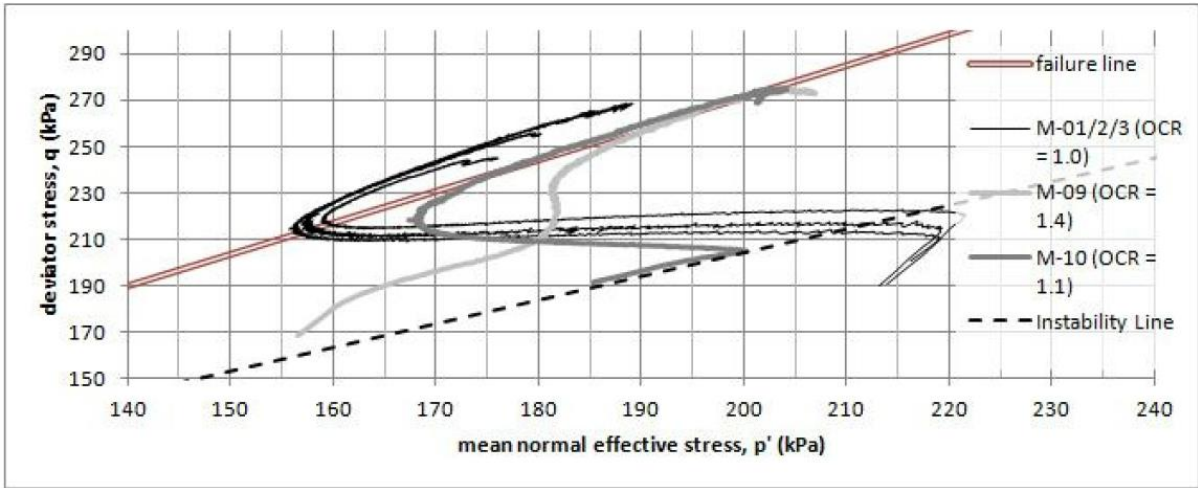
795

796 Figure 11: Pore water pressure response compared to stress pulse for test CB-96-1. Above: at small
 797 strains (0 to 0.1%); contractant response to shear, PWP lagging the stress pulse. Below: at large
 798 strains (8.1 to 8.2%); possible dilation at maximum shear, PWP slightly leading the stress pulse.



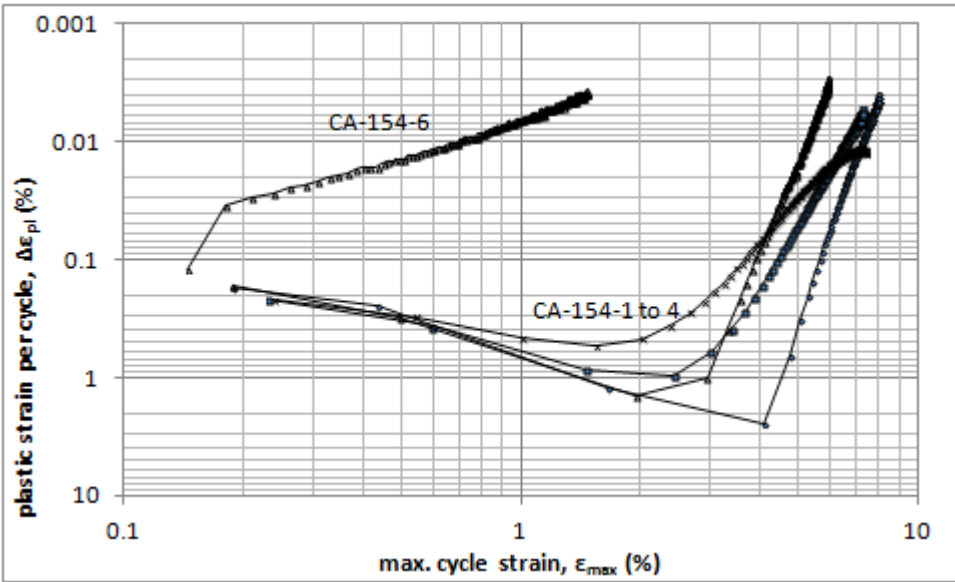
799

800 Figure 12: Changes to undrained shear response and prevention of liquefaction from increasing
 801 overconsolidation. 'X' marks indicate initiation of shear bands. Also note increased failure strain of
 802 lubricated-end sample (ML-05).



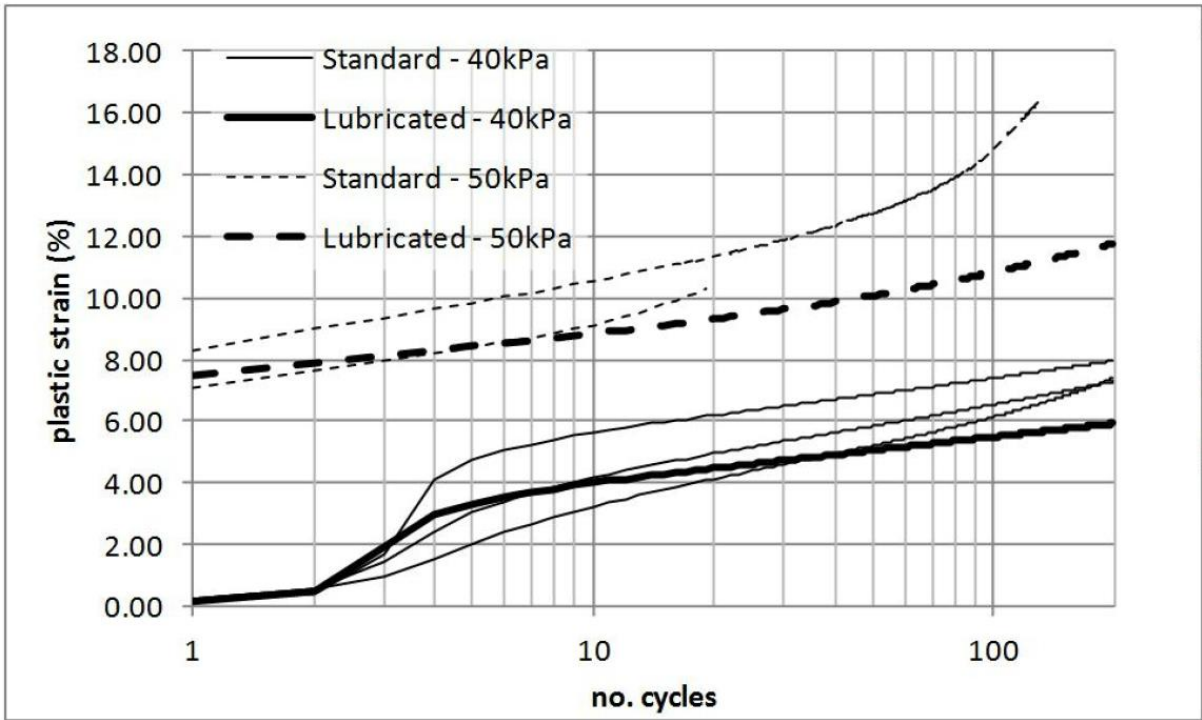
803

804 Figure 13: Undrained effective stress paths of normally consolidated and overconsolidated
 805 monotonic tests with reference to the Instability Line, which separates intact (liquefiable) and
 806 stabilised responses



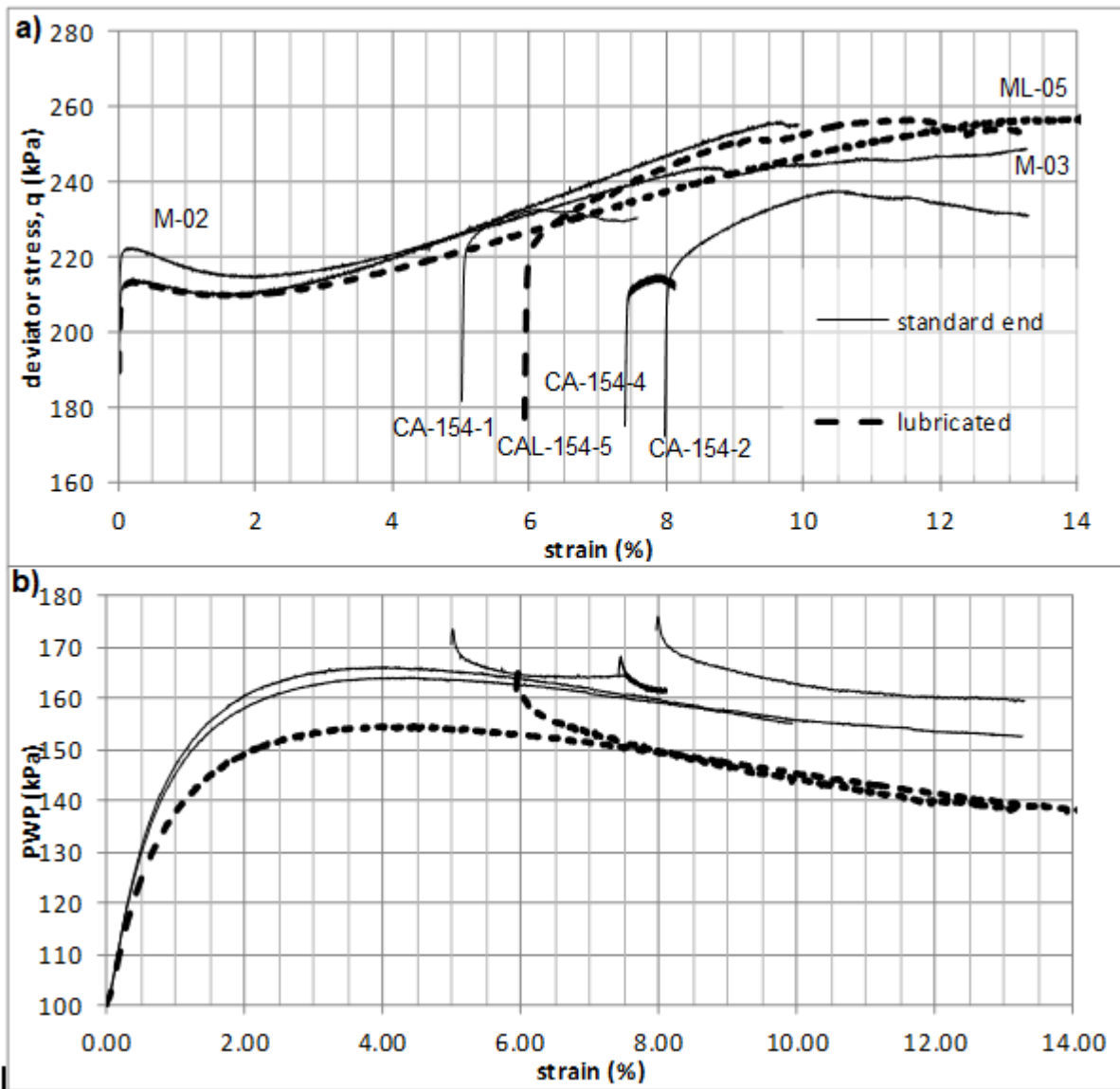
807

808 Figure 14: Reduction in plastic strain accumulation resulting from complete removal and re-
 809 application of maximum deviator stress (CA-154-6) during normal consolidation (c.f. CA-154-1 to 4,
 810 for which consolidation stress was not interrupted)



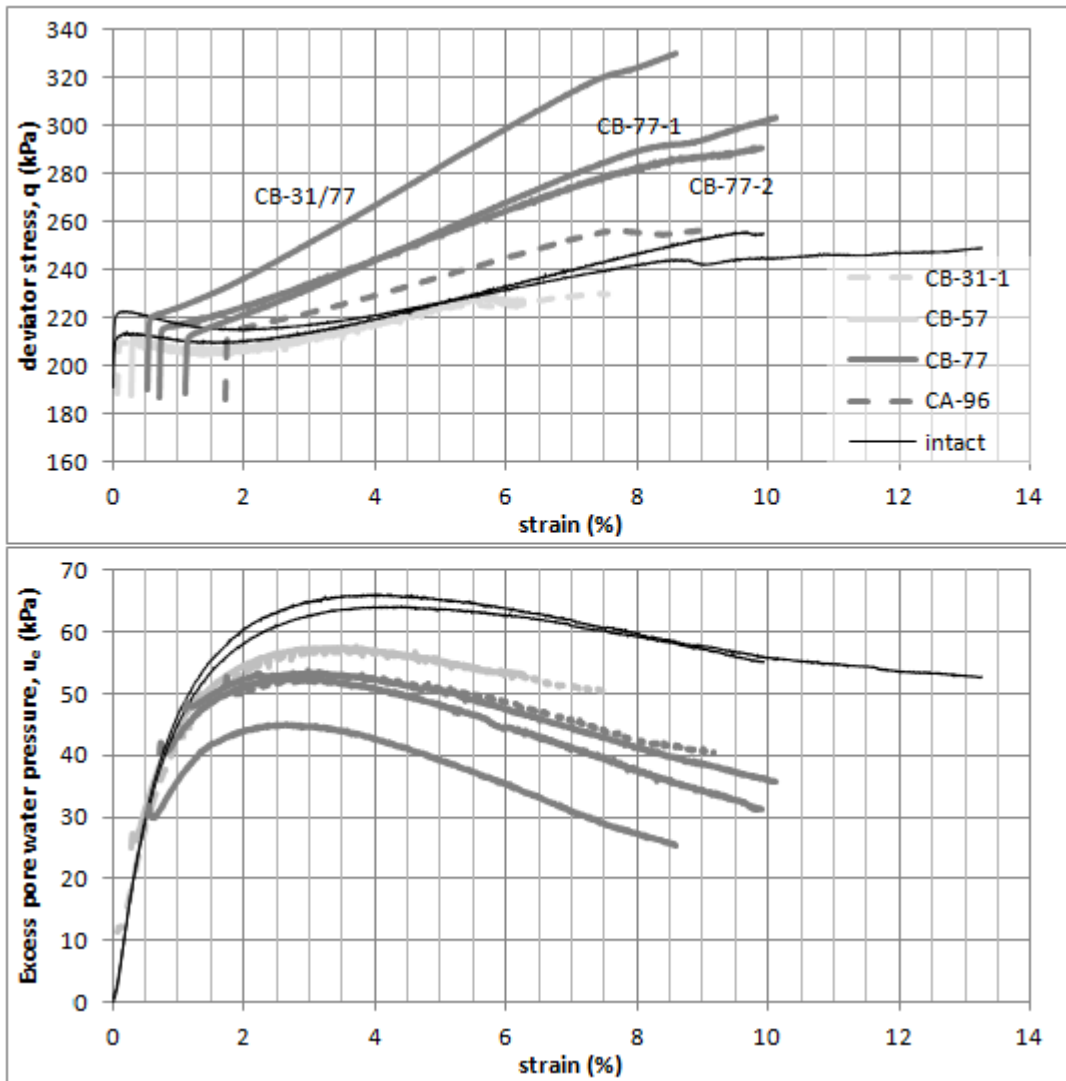
811

812 Figure 15: Reduction in strain accumulation rates within very large-strain regime resulting from end
 813 lubrication for Series 'A' tests (i.e. without stress-correction), in agreement with findings of Lee and
 814 Vernese (1978).



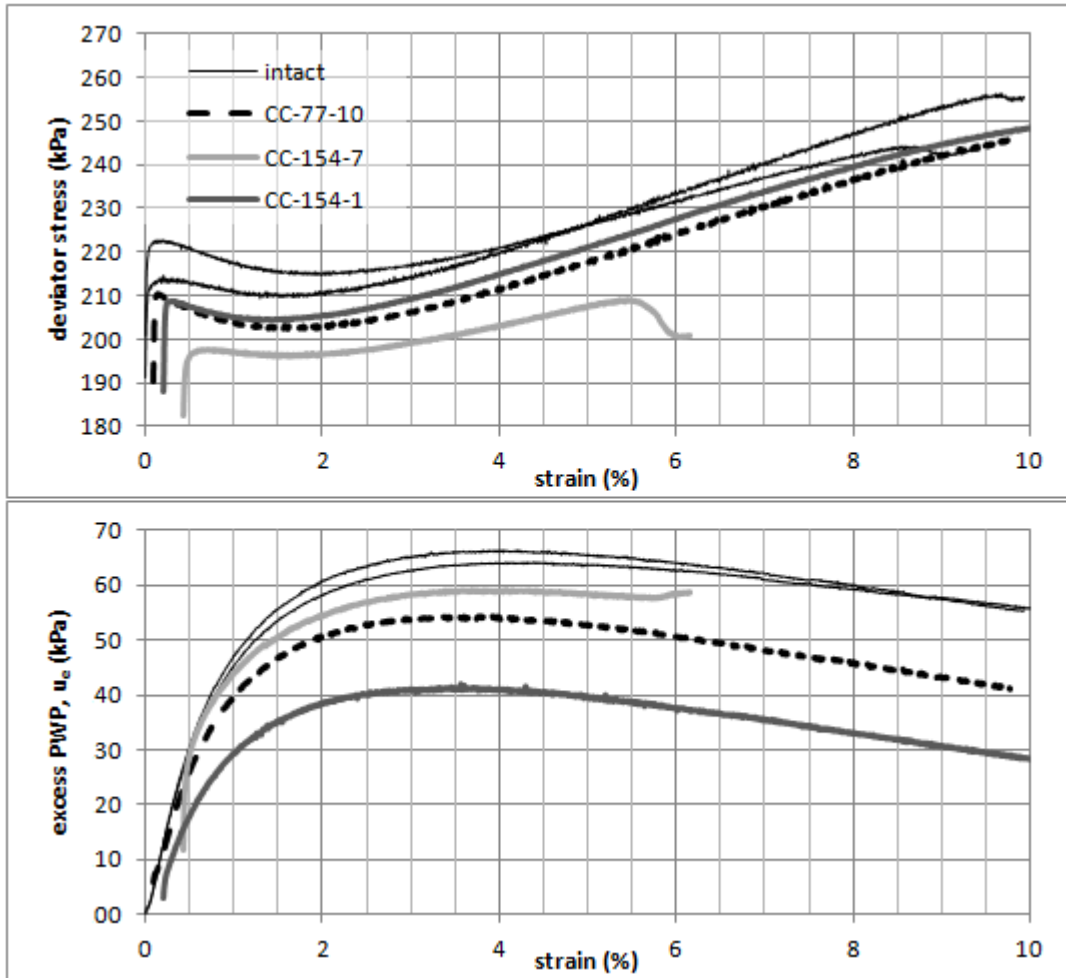
815

816 Figure 16: Post-cyclic monotonic shear response of cyclically liquefied Series 'A' tests with varying
 817 accumulated cyclic strain (final cumulative cyclic strain is the starting strain for post-cyclic curves);
 818 pore water pressure and deviator stress of cycled samples converge to the intact curve but can
 819 trigger shear banding at lower strengths, particularly with standard sample ends.



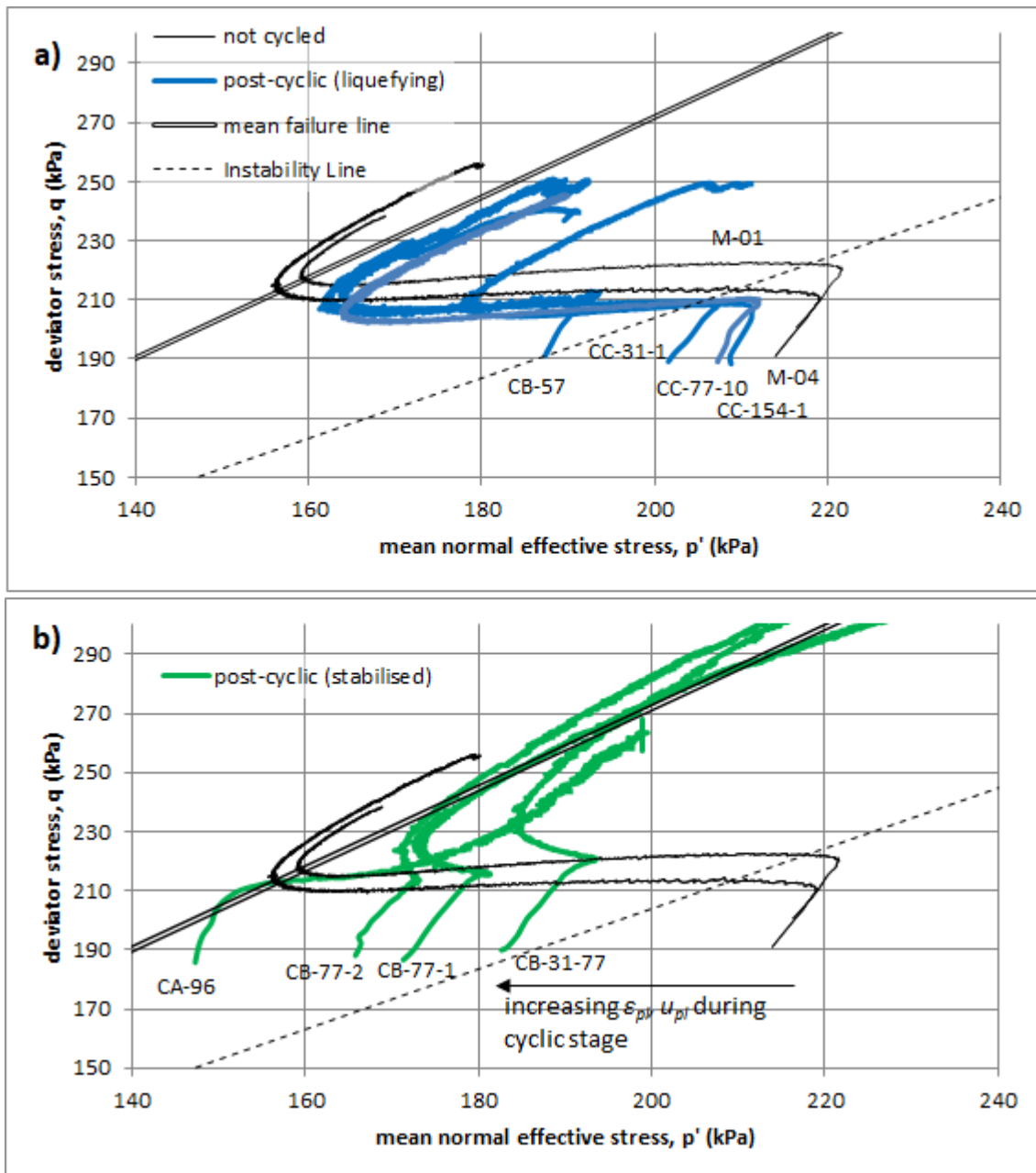
820

821 Figure 17: Post-cyclic monotonic shear response of samples not liquefying under cyclic load. N.b. CB-
 822 77-1 and 2 were conducted with different final consolidation creep rates (see Figure 7) to CB-31/77.



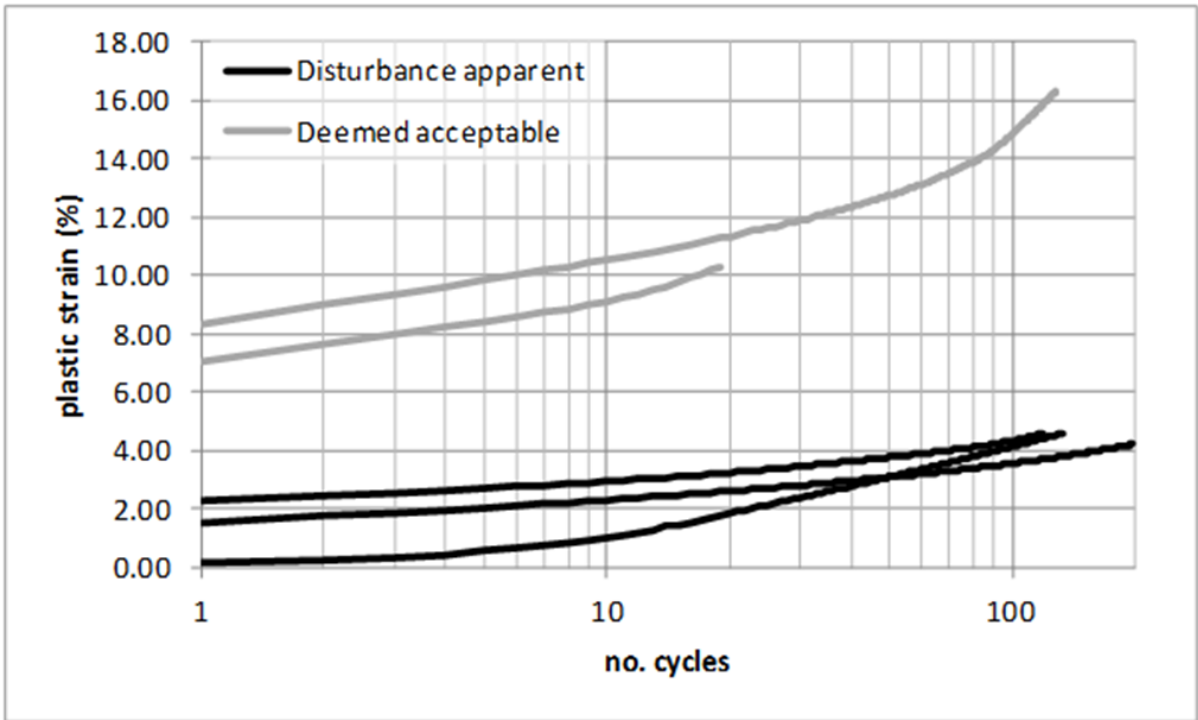
823

824 Figure 18: Monotonic post-cyclic Series 'C' tests: liquefaction behaviour retained for sub-threshold
 825 cycling ($\Delta q_{cyc} = 20\text{kPa}$; CC-77-10) terminated before exceeding liquefaction initiation strain (i.e. at
 826 0.095%). and above-threshold cyclic stress (CC-154-1 and CC-154-7) either halted before liquefaction
 827 initiation (at 0.20%) or after (at 0.43%).



828

829 Figure 19: Undrained effective stress paths of post-cyclic monotonic tests with reference to the
 830 Instability Line and monotonic tests not subject to cyclic load. Similarly to Figure 12, the Instability
 831 Line separates liquefying (a) and stabilised (b) states for undrained monotonic shear.



832

833 Figure 20: Reduced first cycle strain and ongoing accumulation as a result of sample disturbance
 834 prior to consolidation (CA-192 tests).

835