Testing the cyclic strain softening behaviour of an ageing puddled clay core dam

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ABSTRACT

Seismic resilience of the 150-year-old Ross Creek puddled clay core water supply dam in Dunedin has recently been enhanced through a programme of refurbishment. The very low permeability moderately plastic soils comprising the embankment dam are susceptible to cyclic strain softening behaviour that had been analysed using published empirical cyclic strain performance relationships based on material classifications and static testing results. The availability of several undisturbed embankment soil tube samples obtained during the installation of piezometer instrumentation has provided the opportunity to undertake complementary specific cyclic strain testing at the University of Canterbury geotechnical laboratory. This included specialist cyclic triaxial (CTX) and cyclic simple shear (CSS) testing. The paper covers the challenges experienced when undertaking undrained cyclic strain testing on soil of this nature, including the preparation and saturation of the very low permeability samples. When compared to values obtained from the traditional empirical design methods, the test results showed a high degree of variability and sensitivity to both the degree of saturation that could be achieved, the respective clay/silt/sand fractions, and the test method (i.e. CTX cf CSS). CTX test results adjusted for anisotropic field consolidation conditions matching the CSS test method appeared to be quite conservative compared to the single CSS test result. However, we found that average results were broadly consistent with the design values previously adopted. Monotonic shear strength testing undertaken when excess pore pressures were still present following cyclic testing revealed considerable softening, but average undrained strengths were similar to previous TX_{CU} results.

Keywords: cyclic strain softening of clay, cyclic triaxial testing, cyclic simple shear testing.

BACKGROUND AND SCOPE

The recently completed refurbishment project undertaken on the ageing Ross Creek puddled clay core embankment dam, owned by the Dunedin City Council, has involved seismic resilience enhancement measures to improve the stability of the downstream shoulder that had been subject to slope deformation with a mid-slope head scarp development following a storm event in 2010. The dam was originally commissioned in 1867 to provide municipal water supply to the city of Dunedin that had been established some two decades earlier in 1848. The facility is a recognised engineering heritage site. The impoundment attracts a high potential impact classification (PIC) in terms of the Building (Dam Safety) Regulations [MBIE 2022], with the requirement to satisfy a seismic resilience performance criterion of not less than a 1in10,000 probabilistic event [NZSOLD 2015]. This paper presents a summary of findings from several laboratory cyclic load tests undertaken on Dames and Moore samples that were obtained during installation of the new piezometer instrumentation after design of the refurbishment works was completed. The moderately plastic clay-like soil originally used to construct the dam was not expected to experience classic liquefaction behaviour, but significant strain softening was expected to influence deformation performance under design cyclic loading. This phenomenon was allowed for in design of the refurbishment works through reference to empirical estimates of the cyclic resistance ratio (CRR) of the soil in relation to the available undrained shear strength values (Su) [Boulanger and Idriss, 2007]. The opportunity to directly measure the CRR by cyclic laboratory testing has been pursued on a limited number of samples as a check on the published relationships previously relied upon. Both cyclic triaxial testing (CTX) and cyclic simple shear testing (CSS) has been undertaken. The laboratory cyclic tests are identified with a CTX or CSS prefix CTX#2 was not suitable for undisturbed testing, so it was not proceeded with.

A representative maximum cross section of the 33.6 m high refurbished dam is shown in Figure 1 below. The section is skewed relative to the dam crest to lie generally along the creek channel beneath the downstream shoulder. Test sample locations are projected onto this representative section. Phreatic lines are not included due to their variability along the embankment.

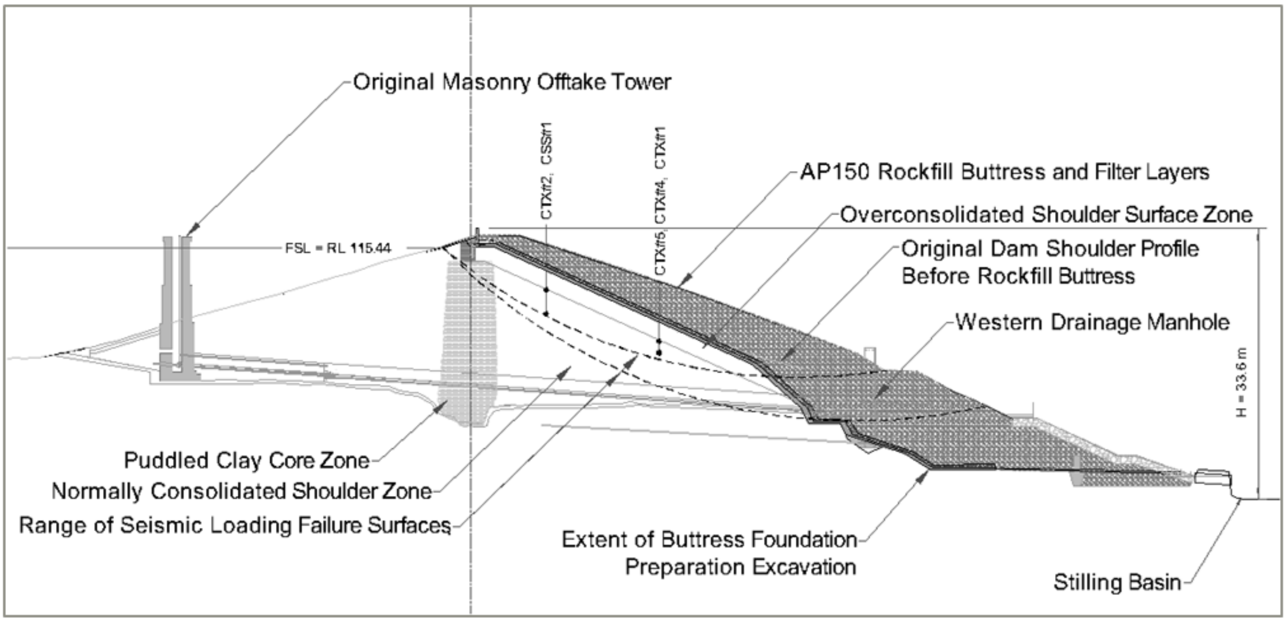

Figure 1 Representative skewed maximum cross section of dam showing projected locations of test samples.

Some 58% of the critical potential failure surface lies within the normally consolidated downstream shoulder zone where the test samples were sourced from. Plane strain stress conditions have been assumed to apply to the stress-strain (PLAXIS 2-D) two-dimensional stability analysis model adopted for design.

STUDY METHODOLOGY

The following sequence of steps has been followed in assessing the quantitative cyclic softening behaviour of the moderately plastic embankment soil potentially subject to dynamic seismic loading. The procedures are generally in accordance with published methods [Boulanger and Idriss, 2007].

1. Seek to standardise all undrained shear strength (S_u) data to an equivalent post refurbishment field consolidation stress state in the vicinity of the potential seismically induced failure surface; presumed to be normally consolidated to anisotropic conditions with horizontal stress ratio (k_0) approximating 0.5 and phreatic conditions matching the measured equilibrated full supply level piezometric profile within the embankment. That is, the equivalent effective vertical consolidation pressure for the samples (σ'_{vo})
2. Review both field and laboratory static undrained maximum shear strength characteristics of the soil in relevant states of consolidation and saturation when subjected to static / monotonic test loading. This data provides the reference basis for quantifying the degree to which the soil strength is expected to be affected under dynamic cyclic loading. Normalised results to be presented in terms of:
   a. Undrained Shear Strength Ratio (USR) = S_u / σ'_{vo}
3. Determine the appropriate equivalent cyclic stress amplitude for testing to apply from the design case ground actions and embankment dynamic response.
4. Determine the undrained cyclic performance characteristics of the soil samples, in relevant states of consolidation and saturation, up to a double amplitude strain limit of 5% (CTX) or 7.5% (CSS) when subjected to the adopted dynamic / cyclic test loading able to be carried out in the available laboratory equipment. Normalised results to be presented in terms of:
   a. Cyclic Stress Ratio (CSR) = τ_{cyc} / σ'_{vc} and the
   b. Cyclic Strength Ratio = τ_{cyc} / S_u, plus the
   c. Cyclic Large Strain Secant Modulus (G_s)
5. Convert the normalised cyclic test results to a standard 1 Hz cyclic frequency and N=30 cycles loading to represent a M_w=7.5 seismic loading event on clay with an associated peak to equivalent uniform cyclic reduction factor r of 0.65.
6. Derive the level ground standardised cyclic resistance ratio CRR

The following further study steps are not included within the scope of this paper.

7. Assess the influence of design seismic loadings that differ from the standardised magnitude Mw=7.5 using the magnitude scaling factor for clay $MSF_{(clay)}$
8. Assess the potential influence of "non level ground" static shear stress on the cyclic test results; (as represented by the Ka factor).
9. Determine the embankment shoulder cyclic resistance ratio (CRRa) values from the reduced test data
10. Review the study results in the context of the seismic performance criteria, design parameters and methods used in the refurbishment project.

LABORATORY CYCLIC TEST METHODS

The state of consolidation in the cyclic test samples should ideally match the field post-refurbishment condition where the lateral effective stress is expected to be around 50% of the vertical effective stress. The laboratory test methods do not necessarily automatically achieve this stress state, so effective consolidation pressure used in conditioning the samples is adjusted using classical elastic theory [Ishihara 1996] to achieve as close to the post refurbishment field condition as possible.

CTX sample preparation: The test material was extruded from the brass sample tube and the specimen was cut from the extruded cylinder material clear of the ends. The material was trimmed in a soil lathe to a nominal diameter of 50 mm, before the ends of the specimen were removed using a mitre box, so that the final length of the specimen was nominally 100 mm. Given the low estimated permeability, strips of filter paper were applied to the outer surface of the specimen, to encourage the pore pressures to equalise more rapidly within the specimen during the test. In addition, two latex membranes were placed around the specimen, with a thin layer of silicon grease applied between the two membranes, to reduce the diffusion of air from the triaxial cell confining fluid into the test specimen. This membrane arrangement was employed due to the relatively long overall test duration (approximately five days in total).

Triaxial test rig. During testing the triaxial cell is filled with water above the level of the top platen but leaving an air gap at the top of the cell. The cell is pneumatically pressurised using the central compressed air supply, and cell pressure is measured at the base of the cell. Back-pressures can be applied to the specimen through either of the top or bottom drainage lines, with pressures being measured on the bottom drainage line, at the same elevation as the cell pressure transducer. Axial displacements are measured using a linear variable displacement transducer, which measures the vertical displacement of the loading ram, external to the cell. The drainage from the specimen is monitored using a volume meter, which can accept drainage from both the top and bottom drainage lines. Axial loads are measured inside the triaxial cell, using a pressure compensated 2 kN load cell. Placement of the axial load cell within the triaxial cell provides a direct measurement of the loading on the specimen. The axial loading during cyclic loading is servo-controlled with closed-loop feedback. During a triaxial compression test, the position of the loading ram is locked in place, and the triaxial cell jacked vertically under displacement control.

CTX testing procedure:

1. Dry mounting the specimen within the triaxial device (surfaces of the end platens dry, with drainage lines nominally saturated).
2. A vacuum of approximately 80 kPa applied to the specimen via the top drainage line, with sufficient time allowed for the axial displacement measurements to stabilise. At this point, initial dimensions of the specimen were estimated.
3. The vacuum was gradually released by supplying de-aired water to the top and bottom drainage lines. This ensured that the triaxial platens were filled with water.
4. The cell pressure and back pressure were gradually raised over a 45-hour period, keeping the difference between cell pressure and back pressure equal to 80 kPa.
5. The specimen remained at this combination of back pressure and cell pressure for a further 23 hours prior to beginning the consolidation phase.
6. The specimen was consolidated isotropically to the stress levels expected after the construction of the buttress. After raising the cell pressure with drainage lines closed, the specimen was allowed to drain, using the volume change and axial displacement data to track the progress of consolidation.
7. At the end of the consolidation process, the Skempton B-value of the specimen was measured. Bender tests to measure the very small strain shear wave velocity and to indirectly derive small strain shear modulus and Young's Modulus (G_0 and E_0) for an assumed Poisson ratio close to 0.5 for saturated samples.

8. Cyclic loading was applied to the specimen at a frequency of 0.1 Hz, which is significantly lower than the rate which would be expected under typical earthquake conditions. As the strength of clays tends to increase with increasing rate of loading, it would be expected that the cyclic strength measured in these experiments would be conservative to the extent of an assumed increase of 9 % per log cycle of loading [Boulanger & Idriss, 2007], based on earlier the work [Zergoun and Vaid, 1994] and [Lefebvre and Pfendler, 1996].
9. Loading was stopped once the peak-to-peak axial strain exceeded 10 %.
10. Following cyclic loading, monotonic shear loading was applied to the specimen under displacement control at a rate of 1 mm/minute with the drainage lines closed. Shearing was stopped once the displacement reached 20 mm.
11. The specimen was split in two after being removed from the device to reveal the internal structure. One half of the material was then oven dried to obtain the moisture content of the specimen as tested.

CSS specimen preparation. The 16mm high test specimen is cut from sample material near the top of the cylindrical soil. Specimen preparation was assisted by the soil being moist and soft enough to cut through using a wire saw, however the mitre box set-up and specimen aspect ratio do not result in an ideal preparation methodology (i.e., some specimen disturbance may occur due to soil rotation in the mitre box etc.)

1. The specimen undergoes a back pressure saturation procedure, with a final back pressure of several hundred kPa reached.
2. The specimen is then anisotropically consolidated to approximate the field σ'_v with σ'_h around half of the vertical value, with k_0 therefore approximately = 0.5, and the Skempton B value is determined.
3. The specimen is loaded in stress controlled undrained cyclic simple shear by applying the target CSR, which for cyclic simple shear is defined as CSR = τ/σ'_v. and is 2/3 of the equivalent cyclic triaxial test CSR
4. The cyclic loading was performed at a frequency of 0.05 Hz (i.e., 20 second period per load cycle), with a maximum of 100 load cycles applied. A constant specimen height condition was attempted to be maintained through physically clamping the vertical actuator in place.
5. Following completion of cyclic loading, the specimen is kept undrained (with the vertical actuator still clamped) and subsequently monotonically sheared in the negative horizontal displacement direction until a shear strain of 20% is exceeded. The direction of shearing is chosen to minimise the shear strain required to reach a peak shear strength, bearing in mind that the direction/sign of horizontal displacement shear is essentially arbitrary, unlike triaxial testing, where positive and negative axial displacement refer to compressive and extensive loadings respectively.
6. The post-test water content is then determined.

CLASSIFICATION OF SOIL SAMPLES

Although the dam shoulder comprises a nominally single material zone that attracts a field description [NZGS 2005] of moderately plastic Silty CLAY with minor Sand, the soil is not homogeneous. Plasticity test results in terms of the Unified Soil Classification System [ASTM 2487] are shown in Figure 2 below, where a mean plasticity index (PI) of 23 was determined. This sample mean value is consistent with the mean of all the plasticity tests undertaken on shoulder samples during the refurbishment project. The samples generally fall into the CL classification, with one CH sample. Table 1 presents the constituent soil fractions in the samples. The major fraction is SILT and the subordinate fraction is CLAY in all samples, with SAND as a variable minor fraction.

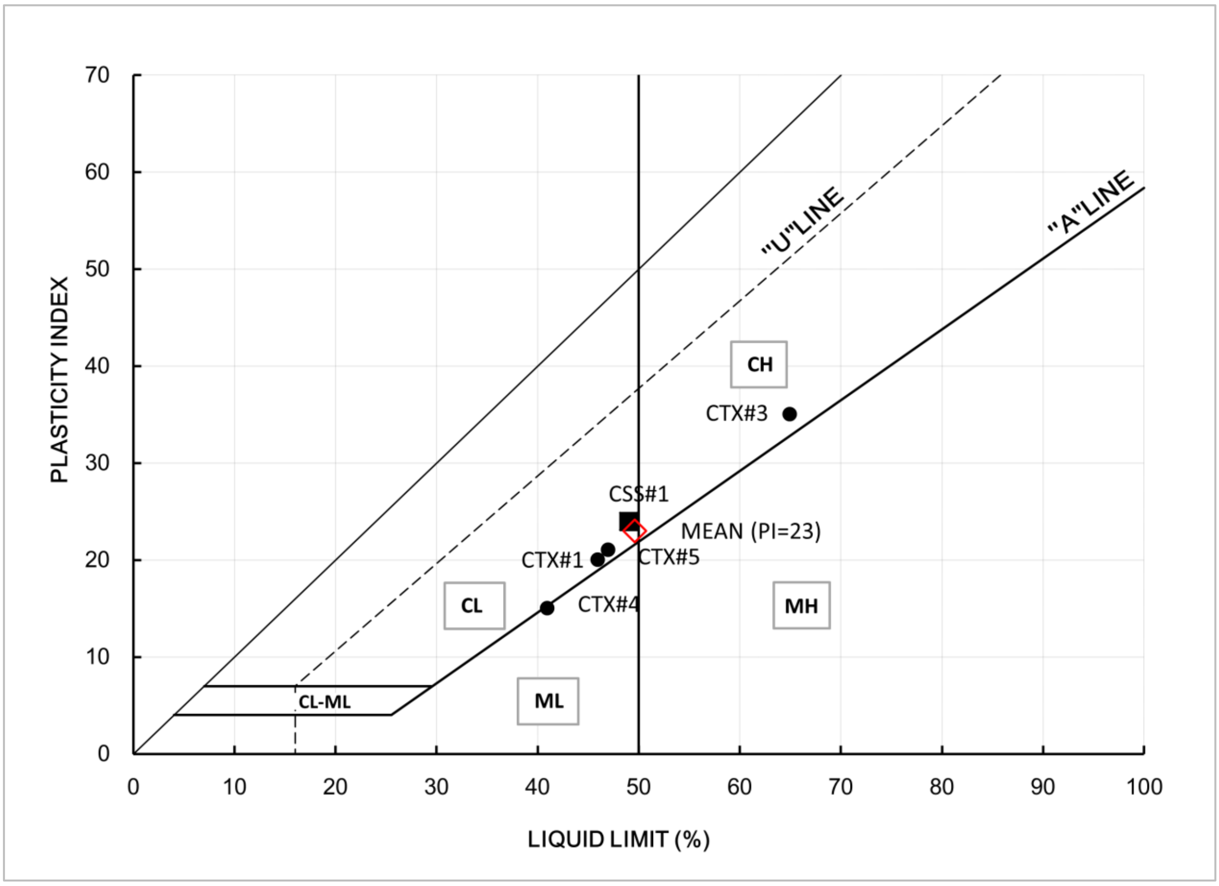

Figure 2 Plasticity classification of cyclic test samples

The cyclic triaxial test samples had the following properties prior to re-saturation and reconsolidation.

Table 1 Initial CTX sample properties as extracted from Dames and Moore samplers

TEST ID	PI	w/c (%)	Liquidity Index	Dry Density (t / m^3)	Void Ratio (e_0)	Air Void Ratio (e_A)	Degree of Saturation (DoS) (%)	Clay, Sand Fractions (%)
CTX#1	20	32.7	0.33	1.422	0.905	0.011	97.6	32, 3.5
CTX#3	35	34.9	0.14	1.358	0.997	0.024	95.1	31.5, 3.5
CTX#4	15	31.5	0.37	1.407	0.926	0.037	92.4	21.5, 19.5
CTX#5	21	32.0	0.28	1.414	0.917	0.026	94.6	20, 12
CSS#1	24	-	-	-	-	-	-	31.5, 3.5
Mean	23	32.7	0.28	1.400	0.936	0.024	94.9	-, -

While the embankment shoulder soil was not expected to be susceptible to “sand like” liquefaction behaviour, it is still useful to identify where the individual soil samples lie in relation to established

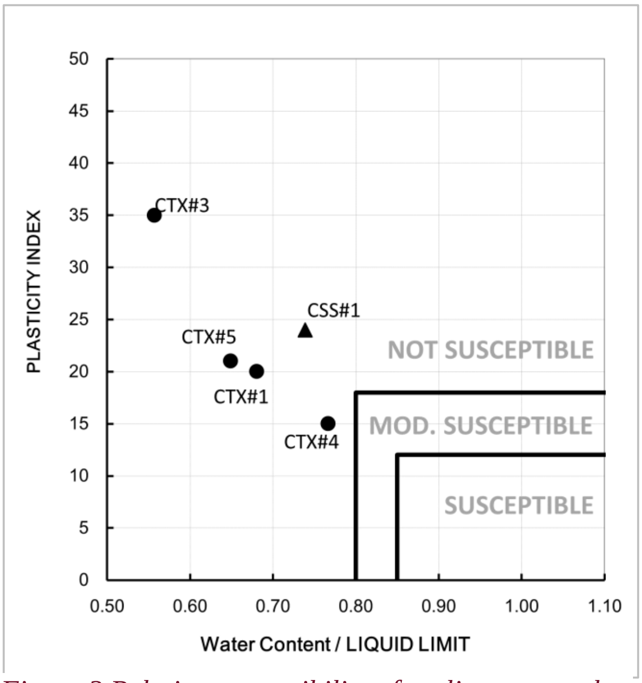

Figure 3 Relative susceptibility of cyclic test samples to liquefaction after Bray and Sancio 2006

susceptibility criteria for fine grained soils of low to moderate plasticity. The following Figure presents this information for the samples in their post tested state (i.e. their likely maximum water content) using accepted susceptibility criteria [Bray and Sancio, 2006].

Sample ranking in increasing order of expected susceptibility to cyclic softening effects is CTX#3 (least susceptible), CSS#1, CTX#5, CTX#1, CTX#4 (most susceptible).Shear wave velocity (Vs) Bender tests undertaken in triaxial isotropic consolidation conditions showed small strain velocity values in the range 208 to 246 m/s, with lower values correlating with the higher clay and lower sand content samples. Small strain elastic properties can be derived from Vs for comparison with the larger strain cyclic test behaviour.

SELECTED CYCLIC TEST RESULTS

Raw results from two selected triaxial samples CTX#3 and CTX#5 are presented in the following figures that represent higher and lower degrees of seismic resilience performance respectively during the CTX testing. These raw results have not been adjusted for equivalent anisotropic stress conditions nor for a higher frequency of seismic loading.

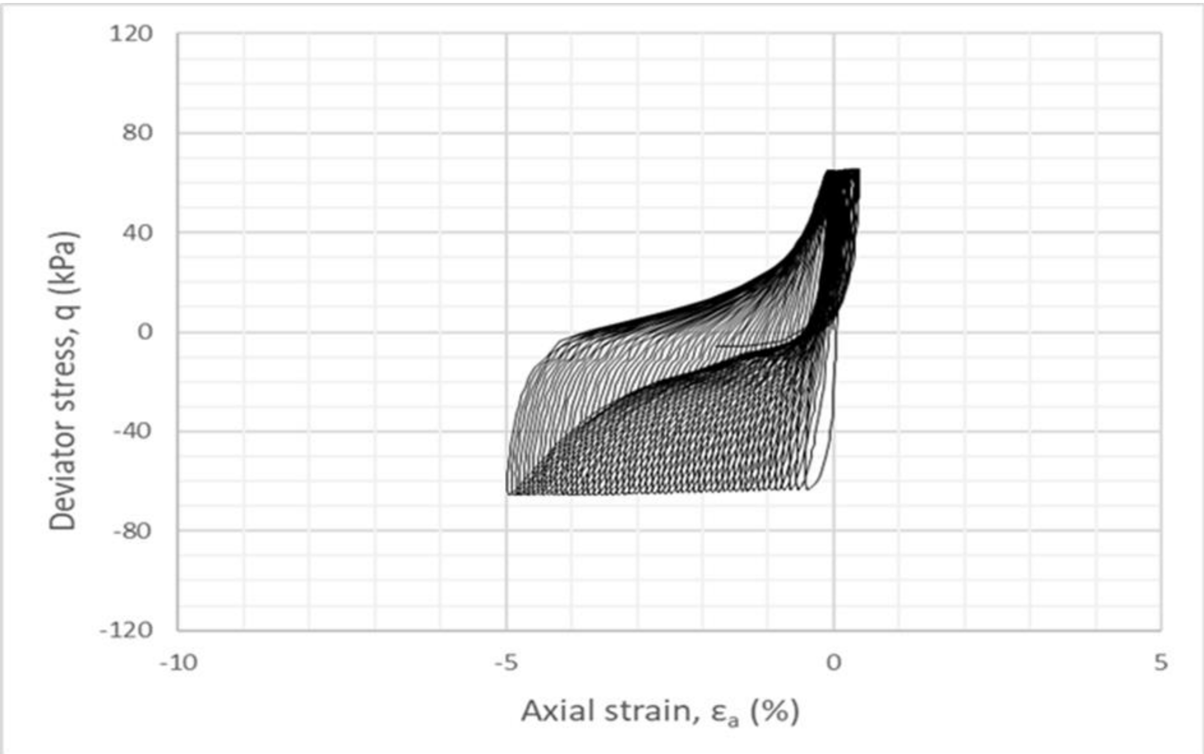

Figure 4 CTX#3 Raw data cyclic deviator stress-axial strain plot

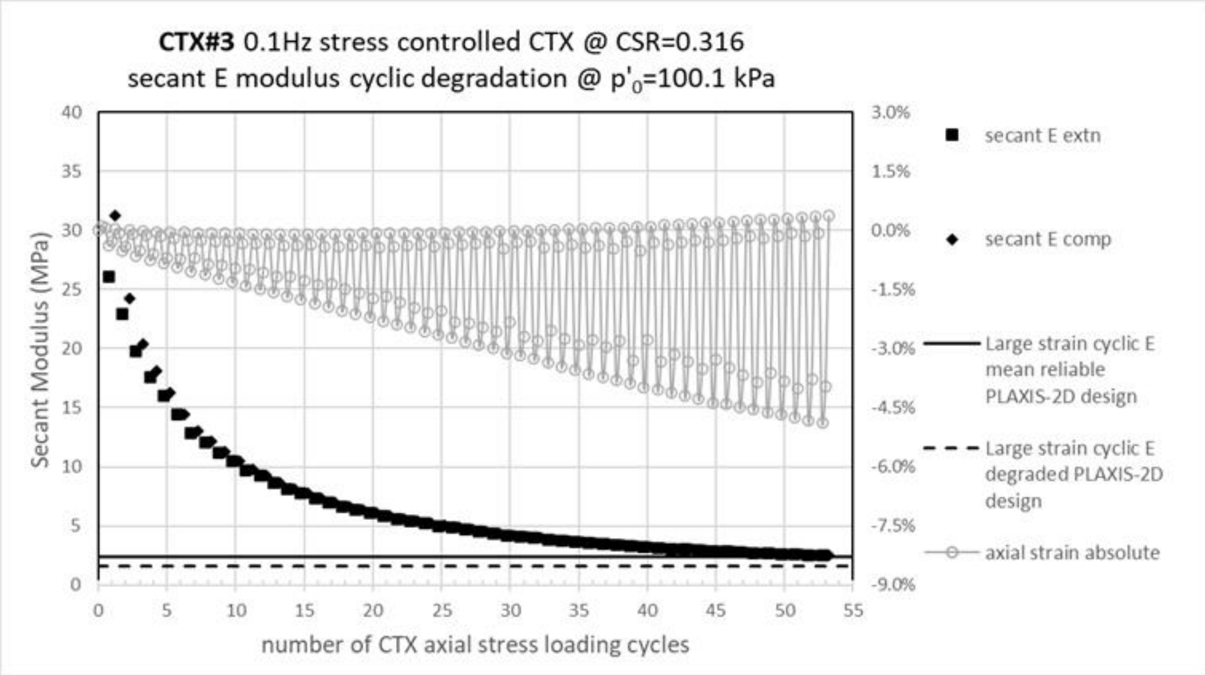

Figure 5 Progressive cyclic change in secant Young's Modulus (Es) for sample CTX#3

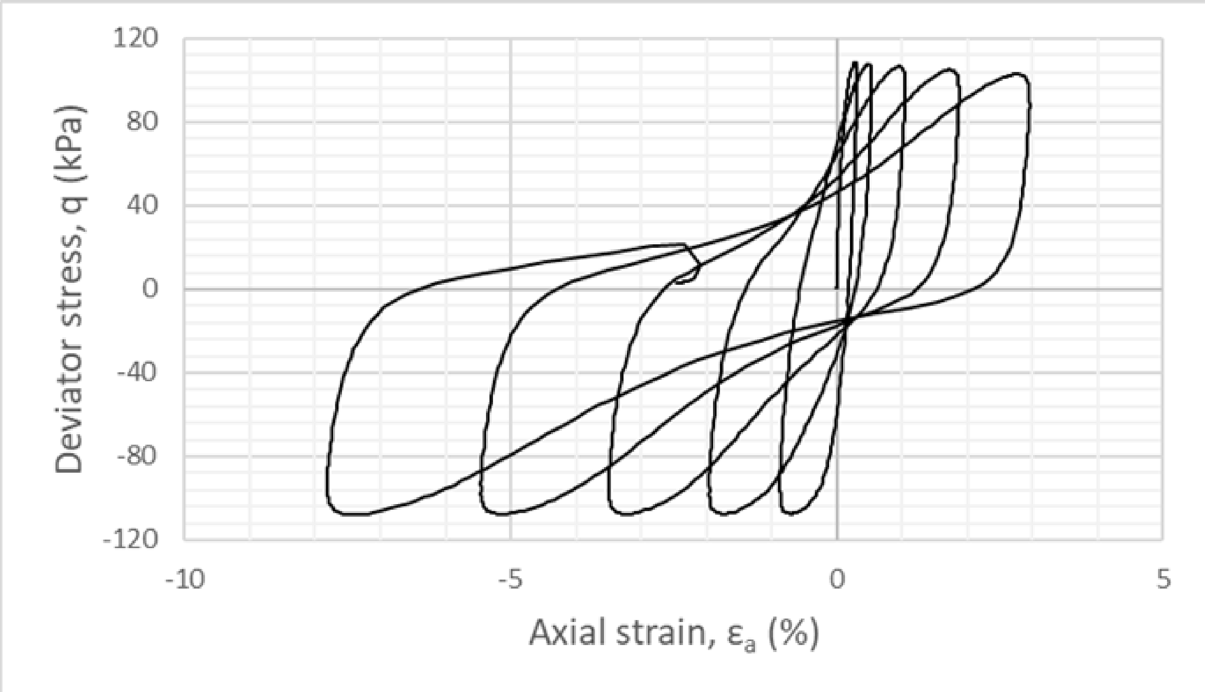

Figure 6 CTX#5 Raw data cyclic deviator stress – axial strain plot

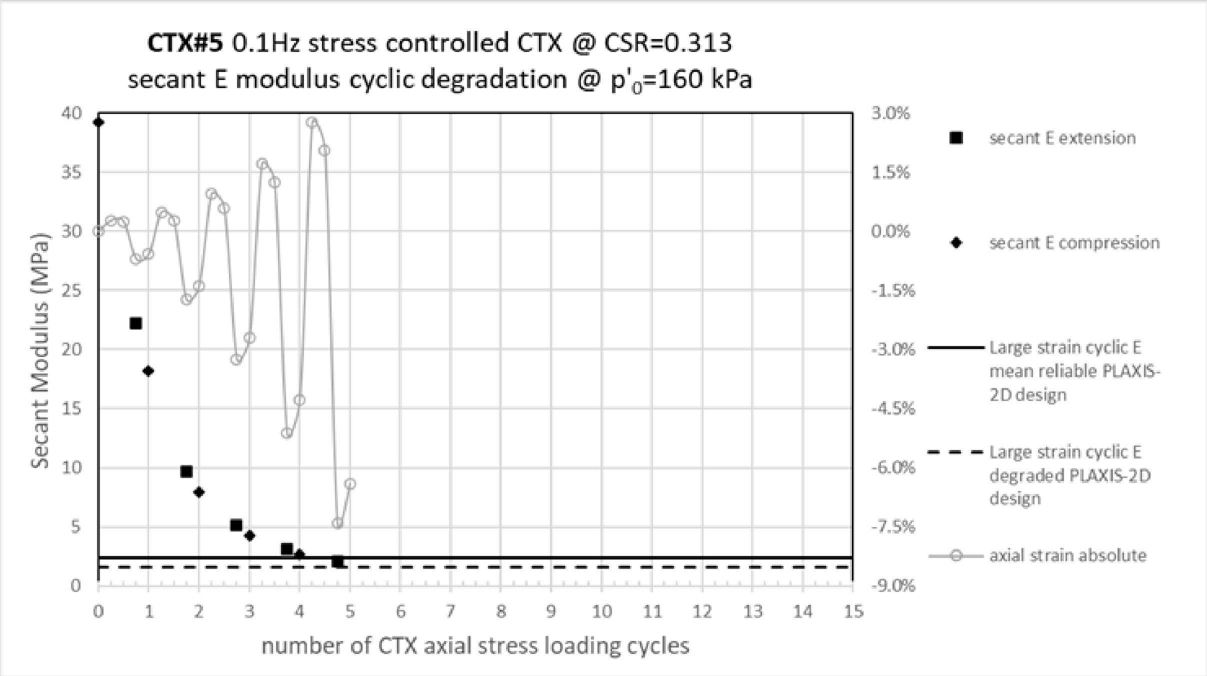

Figure 7 Progressive cyclic change in secant Young's Modulus (Es) for sample CTX#5

The cyclic strain softening results approach values previously adopted for design stress – strain analysis at axial double amplitude strain levels that very closely match the "failure" criteria limit of 5% adopted in the CTX testing. This alignment of the laboratory test "failure" data with the large strain secant modulus Es adopted during design (to reflect the expected cyclic seismic performance), is pleasing. However, this confirmed alignment of the failure criteria does not address the matter of the disparity between the cyclic resilience and associated transient excess pore pressure response of the two selected test samples at very similar triaxial CSR test levels of 0.316 and 0.313 respectively.

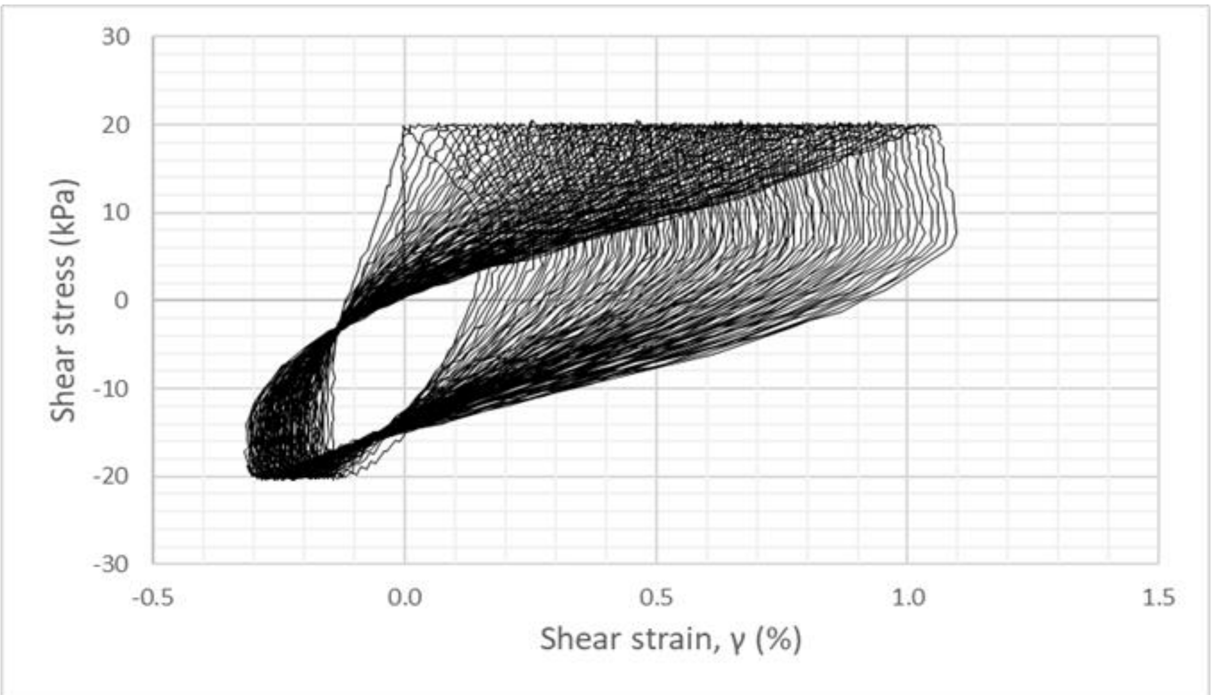

Figure 8 CSS#1 Raw data cyclic shear stress – shear strain plot

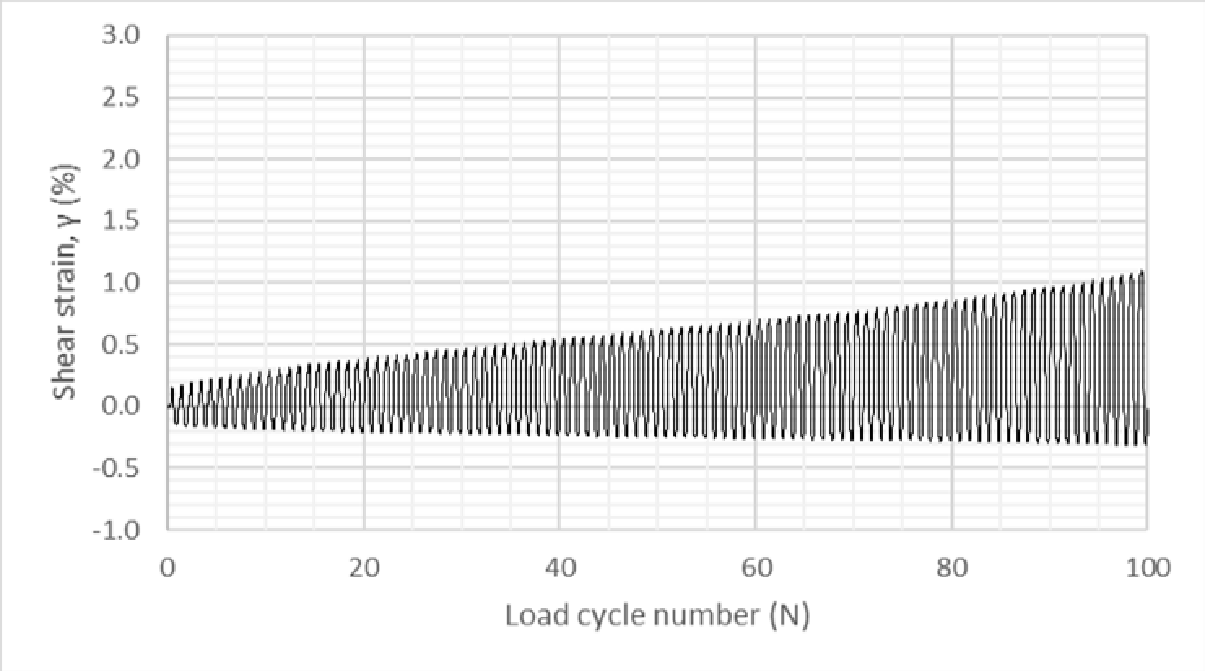

Figure 9 CSS#1 progressive cyclic strain softening effect at σ'v = 98 kPa, CSR =0.204

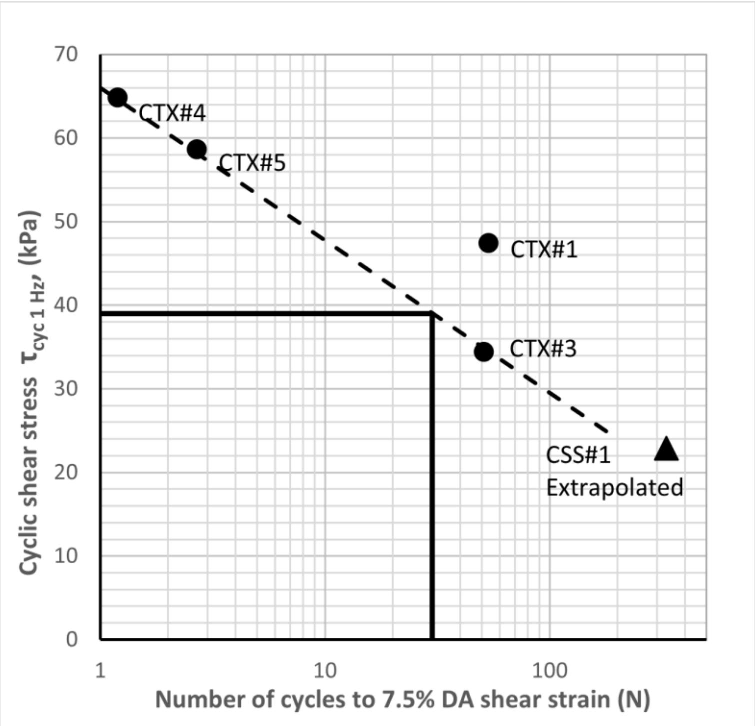

Figure 10 Presentation of all results in terms of cyclic simple shear 1 Hz parameters

While it was hoped to undertake several CSS tests to complement the CTX results, only a single CSS test was able to be performed due to a combination of sample quality factors and limited availability of the laboratory equipment. The following Figure presents raw data summary for the CSS#1 test for CSR=0.204 (0.31 in equivalent CTX terms). The shear "failure" criterion for the CSS test is 7.5% double amplitude strain for equivalence with the CTX test 5% value. This test was provisional in nature to prove the detailed method on a test sample that was of less than ideal test quality. Notwithstanding the slower rate of cyclic loading, the very high number of CSS cycles (N =100) undertaken without approaching the 7.5% double amplitude strain limit was more favourable than resilience expectations generated from CTX#5 and CTX#3 results. The tangent shear Modulus (G) degraded to 2 MPa from the initial 16 MPa after 100 cycles and 1% shear strain as shown in Figure 9. Combining the CTX and CSS results requires standardisation of test parameters covering the frequency of cyclic loading and the limiting strain value. Adjusting results for 1 Hz cyclic test frequency and 7.5% shear strain allows consistent presentation of results as per Figure 10. While Figure 10 suggests an indicative N_{30} value applicable to clays would relate to a cyclic shear stress of 39 kPa, this presentation omits reference to the effective confining pressure parameter used in the cyclic stress ratio (CSR) approach or alternatively the

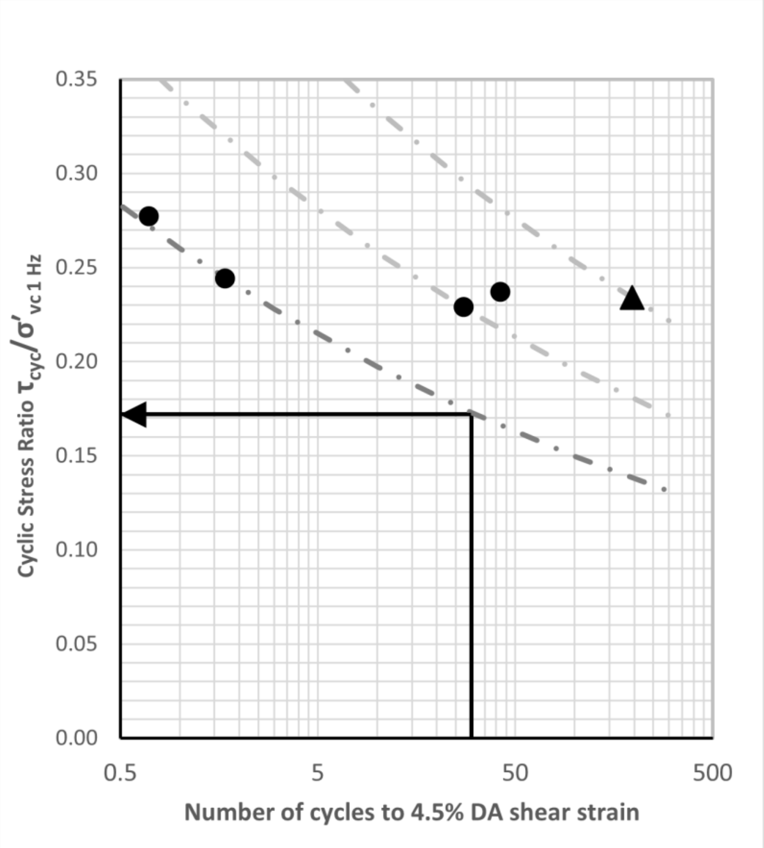

Figure 11 Cyclic stress ratio results at a more conservative 4.5% DA shear strain -0.12 exponent trend lines [I&B 2008]

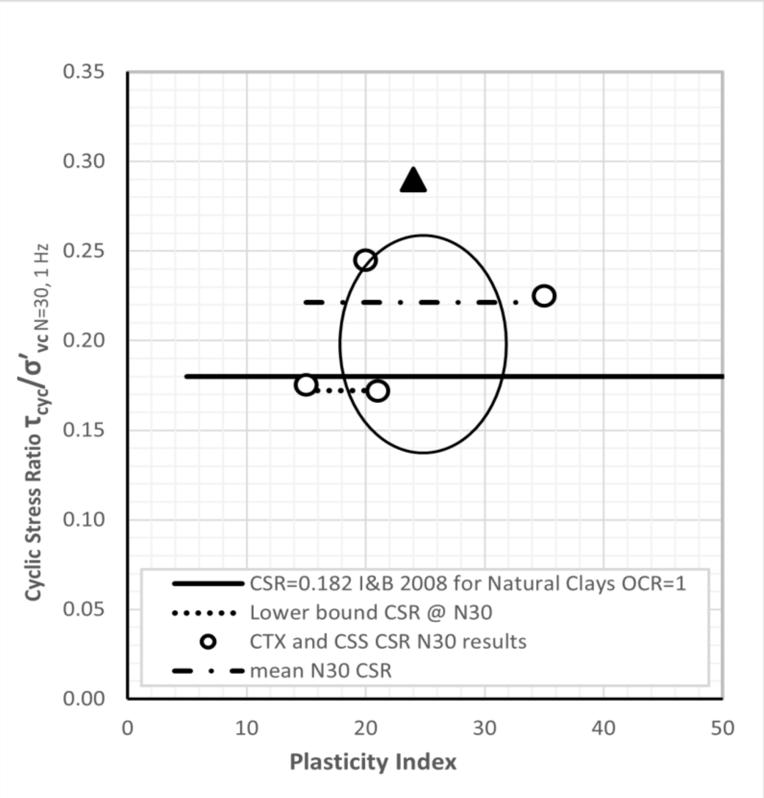

Figure 12 Derived N_{30} cyclic stress ratio values cf published empirical relationship [Idris and Boulanger 2008]

Su value used in the cyclic strength ratio approach [Idris and Boulanger, 2008]. However, converting the results to the (CSR) parameter as presented in the Figure 11 reveals little relationship to the number of cycles to reach the limiting strain value. This suggests that sample variability has significantly influenced the results, and there is a need for care when interpreting the test data in a design context. Adopting a more conservative 4.5% DA limiting shear strain for embankment dams and normalising the test data to N_{30} values using a typical exponential function for natural clays [Idris and Boulanger, 2008] indicates that a CSR value of 0.172 is exceeded for all samples, and the mean CSR value that may be reasonably applicable to a significant slope failure could well lie above 0.20. This is shown in Figure 11.

Extracting the adjusted N_{30} index values from the above Figure and plotting them in the empirical method format [Figure 133(b) of Idriss and Boulanger 2008] reveals a generally consistent outcome. The CSR values shown in Figure 12 are applicable to single axis flat field (i.e. non sloping) conditions. The two-axis flat field N30 CSR value needs to be reduced by the C2D factor of 0.96 for clays to account for contributions from orthogonal effects. The lower bound and mean flat field N30 CSR2D values from the cyclic testing therefore become 0.165 and 0.212 respectively.

Moving on to the cyclic strength ratio (τ_{cyc} / S_u) presentation format, some care has been found to be required for the determination of appropriate S_u values to be adopted due to the degree of scatter in the S_u data obtained from various field and laboratory sources. Three alternative S_u functions were considered to assist in identifying possible trends or consistency in the test data are carried through to the following assessment. These alternatives are:

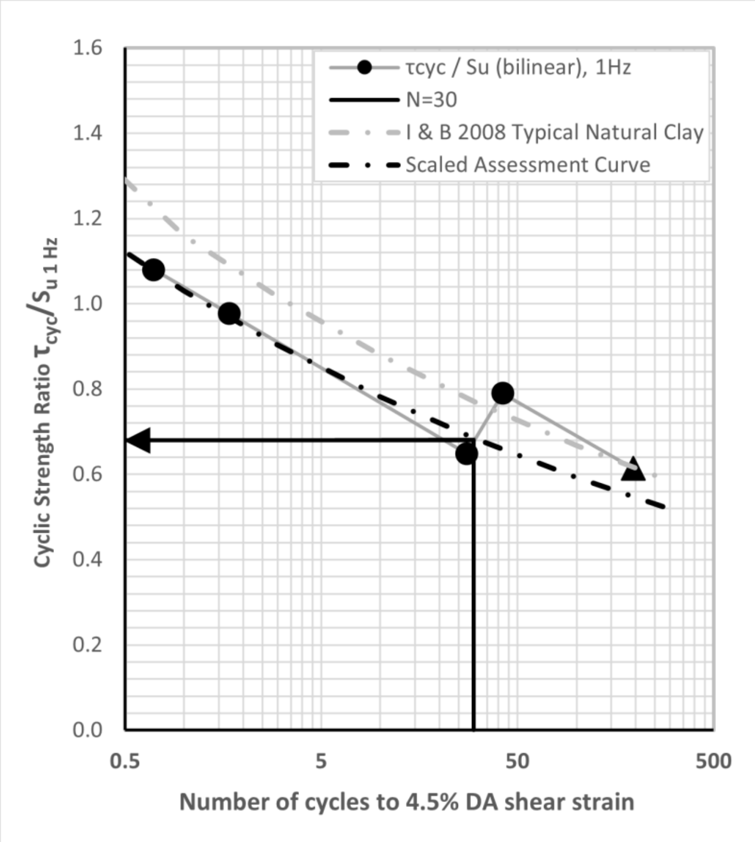

Figure 13 Presentation of all results in terms of cyclic strength ratio using a bilinear S_u function and showing -0.12 exponent trend lines after Idriss and Boulanger 2008

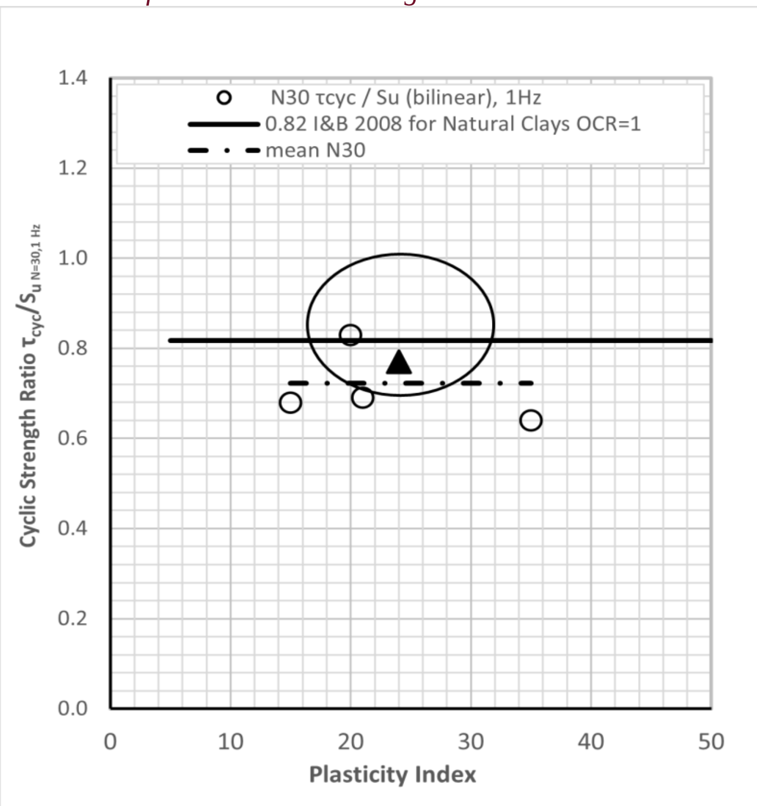

Figure 14 Cyclic strength ratios cf published empirical relationship [Idris and Boulanger 2008]

- the tested post cyclic monotonic peak strength for the individual test samples,
- a conservative constant S_u value as used in the refurbishment design analysis (40 kPa), and
- a calculated mean bilinear S_u function capped at 60kPa.

The bilinear function generated the least scatter in the cyclic strength ratio values and the derived results are presented in Figure 13.

On this basis the results indicate a 0.68 Cyclic Strength Ratio (1 Hz, N_{30}) applicable to an Mw=7.5 design earthquake. The N_{30} derived results are presented in Figure 14 in the context of published standardised generic results [Figure 133(a) of Idriss and Boulanger, 2008] for "Normally Consolidated Moderately Plastic Soil". The plotted Cyclic Strength Ratio 1 Hz, N_{30} (τ_{cyc} / S_u) results fall slightly below the suggested empirical values and may be seen to more broadly consistent with the "Tailings and Natural Silt" points [Fig 133a Idris and Boulanger 2008] that have not been re-presented.

The N_{30} derived mean cyclic strength result for all test samples is 0.722, and the mean PI value of the samples is 23, with the 15 to 35 PI range in the test samples shown by the dash-dot line. While the mean values of the cyclic stress ratio and cyclic strength ratio derived from the cyclic test data have shown a reasonable degree of correlation with published empirical relationships [Idris and Boulanger, 2008] any factors that may influence any non-conservative findings should ideally be identified.

The following Table presents a summary of the cyclic tests presented in order of increasing cyclic resilience. The fitted bilinear shear strength function was derived from the full suite of previously available classification tests. The summarised derived test values have been normalised to isotropic stress conditions with assumed $k_0 = 0.5$. Values are normalised for clay behaviour in an Mw=7.5 design earthquake. i.e. 30 cycles (N_{30}), 4.5% limiting shear strain, and 1 Hz frequency.

Table 2 Summary of normalised cyclic test results (refer to Table 1 for material classifications)

	CTX#4	**CTX#1**	**CTX#5**	**CTX#3**	**CSS#1**
Equivalent σ'_{vo} (kPa)	234	240	150	200	98
Test CSR $_{1\ Hz}$ ✶(in CSS format)	0.277	0.244	0.229	0.237	0.234
N $_{1\ Hz,\ 4.5\%\ shear\ strain}$	0.7	1.7	27.5	42	195 (extrapolated)
CSR $_{N30,\ 1\ Hz}$	0.175	0.172	0.225	0.245	0.29
Fitted bilinear shear strength function S_u (kPa)	60	60	53	60	37.3
Cyclic Strength Ratio $_{N30,\ 1\ Hz,\ 4.5\%\ shear\ strain}$	0.68	0.69	0.64	0.83	0.77
Post cyclic peak shear strength S_u (kPa)	52.7	101	58.5	108	34.9
Post cyclic secant shear modulus to peak. G (MPa)	0.341	1.27	0.516	0.662	0.161
Skempton B test value during consolidation	0.94	0.07	0.94	0.65	0.70
Post testing degree of saturation (DoS)	0.993	0.999	0.993	0.993	0.993

✶CSR values in CTX format are 1.5 times the CSS format values shown.

The selection of test CSR value for individual tests was influenced by previous test performance as well as theoretical seismically induced field stresses. i.e. the essentially random selection of the small number of test samples may, for example, have led to the poor performance of CTX#4 with its relatively high test CSR value. However, the summary tabulations indicate, not surprisingly, that the plasticity index (PI) values have had a significant influence.

The field cyclic resistance ratio CRRM=7.5 is derived from the standardised flat field CSRM=7.5 at N_{30}, using the anisotropic effective vertical pressure variable approach and correcting for slope effects and associated prevailing static stress state, plus adjusting for design earthquake magnitude using the magnitude scaling factor (MSF) for clay. Further adjustments for loading frequency at other than 1 Hz may also apply to the dam embankment, as well as stress-strain analysis to reflect the holistic dynamic behaviour of all the contributing embankment zones. These design aspects fall outside the scope of this paper.

MONOTONIC SHEAR STRENGTH FOLLOWING CYCLIC TESTING

The summarised values included in Table 2, have been normalised to isotropic stress conditions with assumed $k_0 = 0.5$. The fitted bilinear shear strength function was derived from the full suite of previously available classification tests.

The post-cyclic monotonic test results indicate that the undrained strength remains similar to the undrained strength without prior cyclic loading as shown in Figure 15. However, the secant shear modulus to peak strength is relatively low; indicating the degree of softening that has occurred.

Skempton B test values and post testing degree of saturation results included in Table 2 illustrate the challenges experienced in reliably measuring the pore pressures within the very low permeability samples while also apparently achieving an acceptably high degree of saturation. However, the low B values obtained for samples CTX#1 and CTX#3 may help explain the higher strengths obtained by these samples.

As per the test methodology previously described, the undrained cyclic test specimens were held in their undrained state (retaining any excess pore pressure) and subject to monotonic shear strength testing to determine their post-cyclic peak strength values. These post-cyclic strengths were then compared to previously obtained monotonic shear strength test results on the normally consolidated dam shoulder material.

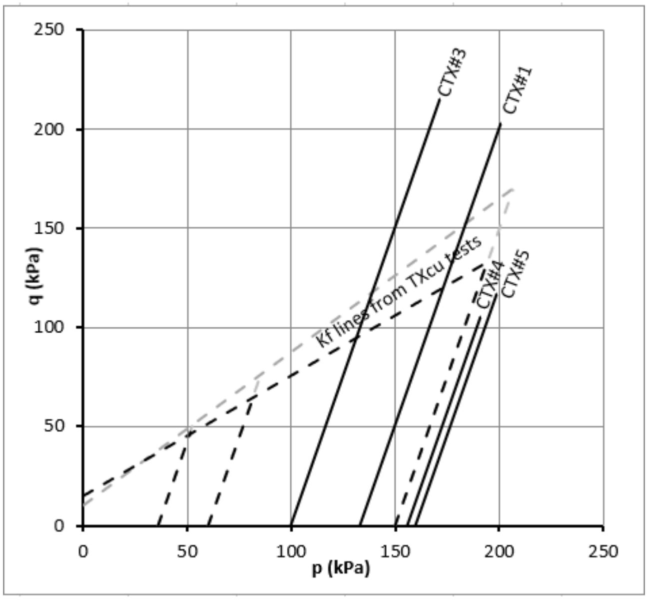

Figure 15 Comparing post CTX monotonic strengths with previous TX_{CU} tests

Figure 15 illustrates all four post-cyclic monotonic triaxial total stress path results along with the triaxial total stress envelopes from the earlier TX_{CU} testing for comparison.

It is apparent from Figure 15 that the overall (mean) undrained monotonic performance of the samples following cyclic testing is quite comparable to the previous monotonic TX_{CU} test results undertaken on samples that had not been subject to this prior cyclic loading. That is, the mean values of the two sets of total stress data are very similar, but the variability in both sets is also clearly apparent, with the maximum strength of the CTX#4 and CTX#5 samples being significantly lower. This factor may be partially accounted for by the relatively higher normalised CSR test values for these test samples, but there appears to have been a significant influence from the plasticity index and possibly the relative clay and sand fractions as summarised Table 2 where the tests are presented in order of increasing test cycles N value.

CONCLUSIONS

The cyclic testing of a small number of undisturbed soil samples from the normally consolidated downstream shoulder of an 1867 puddled clay core embankment dam has provided useful laboratory data to complement the empirical relationships used during design of refurbishment works to determine cyclic strain softening performance from static test data. However, some lessons that may be useful to others have been learned from the limited laboratory testing programme.

- The sealed test samples had been in storage for many months prior to commencing laboratory testing when the degree of saturation was found to range between 0.92 and 0.98 when removed from the samplers. With permeabilities in the order of $1x10^{-10}$ m/s, achieving full saturation proved to be very challenging despite the substantial sample preparation efforts employed. Cyclic resilience performance did generally correlate inversely with the Skempton B values obtained during the consolidation phase, and also partially with the PI values.

- Selecting the cyclic loading frequency for very low permeability samples is inevitably a compromise between reflecting field seismic ground actions and accurately capturing the pore pressure response.
- Cyclic stress ratio results based on the theoretical effective vertical stress and $k_0 = 0.5$ assumption were found to be more readily derived than cyclic strength ratio values that are reliant on defining the relevant corresponding S_u values. The very wide scatter in the S_u values obtained from previous field and laboratory testing required the adoption of a mean regression function. In this case a bilinear function with the S_u capped at 60 kPa was found to be useful. However, sensitivity to the variability needed to be considered in a design context.
- The derived Cyclic Strength Ratio 1 Hz, N_{30} (τ_{cyc} / S_u) results fall slightly below the suggested and expected empirical values for "Normally Consolidated Moderately Plastic Soil" and may be seen to more broadly consistent with the "Tailings and Natural Silt" points [Figure 133(a) of Idriss and Boulanger, 2008]. This outcome may be related to the original 1860's puddled core construction methods.
- The strain softened secant and tangent moduli obtained from cyclic testing showed a high degree of variability between tests, but the mean values were found to be generally consistent with the values used previously in stress–strain analysis for design of the refurbishment works.
- While the limited availability of the CSS test equipment eventually resulted in only one trial CSS test being undertaken, the ability of this test method to represent the field embankment anisotropic stress state and plane strain failure mechanism more realistically was worthwhile. The indicated higher cyclic resistance performance obtained from this single test was consistent with expectations relative to the CTX test method. However, given the observed sample variability, little confidence can be placed in a single test result, other than to reinforce the apparent conservative nature of the CTX test method in this embankment context.
- The cyclic strain softening performance of the individual test specimens was found to corelate very well with the liquefaction susceptibility approach [Bray and Sancio, 2006] as shown in Figure 3. There may be opportunity to usefully extend this concept to strain softening susceptibility zones with more data.
- Monotonic undrained shear strength testing of specimens immediately following undrained cyclic testing has shown mean undrained shear strength values commensurate with similar specimens previously tested in staged monotonic TX_{CU} tests. However, individual test values vary considerably, and appear to be influenced most strongly by the consolidation phase Skempton B values and somewhat by the PI values.

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