Undrained behaviour of intact soft clay under cyclic paths that match vehicle loading conditions

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Abstract

Vehicle traffic loading appears to contribute significantly to long term settlement beneath highways, airport runways and metro lines in China. Wheel loading imposes cycles in both the magnitude and direction of the principal stresses acting on the soils beneath pavement or rail-track structures. Conventional cyclic triaxial testing, which is not capable of imposing such stress paths may underestimate how heavy traffic loading affects any underlying soft clay layers. Hollow cylinder apparatus (HCA) can simulate such traffic loading stress paths more accurately, including rotation of the principal stress directions. This paper presents a systematic experimental study of cyclic HCA (CHCA) tests on K_0 -consolidated saturated soft clay involving cyclic variations in both vertical and torsional shear stresses, along with a parallel programme of cyclic triaxial (CT) tests, considering the undrained response of saturated samples of intact soft clay. It is shown that when applied above certain critical cyclic stress ratios, principal stress rotation accelerates excess pore water pressure and permanent strain development. Corresponding changes are also seen in the resilient modulus and damping ratio trends. The discrepancies between the behaviour of CHCA and equivalent cyclic triaxial tests grow as the cyclic stress ratios increase.

Keywords: Soft clays; laboratory tests; settlement; stress path; repeated loading

Introduction

Many important airport runways, metro, rail and road routes have been built recently on low embankments founded on the deep soft clay outcrops that cover much of SE China. Large subgrade settlements have often been observed that pose difficulties for transport system operations, including high maintenance costs (Guo *et al.* 2013). While ongoing consolidation and creep under the low embankments may have contributed significantly, settlement rates may also have been exacerbated by vehicle wheel loading (Chai and Miura 2002). Such loading is likely to be most intense on very busy multi-lane highways in cases where several lanes of heavily laden multi-wheel trucks often travel in parallel, as indicated in Fig. 1.

The vertical stress increments caused by single wheels reduce rapidly with depth below road level. Pavement design aims to limit the strains developed in the underlying soft clay layers to negligibly low values. The 2m thick road-plus-embankment design shown in Fig. 1(a) illustrates the typical cross-section adopted on the soft clay sections of many Chinese highways. However, groups of heavily laden trucks, which may comprise four or six wheels located over their front sections, develop overlapping stress fields that can impose significant stress changes at depth, as illustrated in Figs. 1(b) and 1(c). The stationary vertical stress profiles shown in Fig. 1 (c) were assessed from three dimensional elastic FE calculations that assumed: the construction shown in Fig. 1(a); loading by either a single wheel or the combined effect of three 31-ton 10 wheel, 2740 mm wide, Dongfeng 241 lorries, whose wheelbases are as indicated in Fig. 1(b) and Young's

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moduli of 35, 35, 30 and 3 MPa in the asphalt concrete, base, embankment and soft clay subsoil units respectively. As shown in Fig. 1(c), the vertical stress increments imposed at Point X, which decline from 18 kPa to ≈ 2 kPa within the first 10 m of the deep soft clay soft clay, far exceed those from the single wheel case.

Brown (1996) and Lekarp *et al.* (2000) reviewed the stress paths imposed by single loaded wheels travelling on pavements and tracks, showing how the principal stresses and their orientations vary lead to the stress path scheme illustrated in Fig. 2. Powrie *et al.* (2007) report extensive two- and three- dimensional numerical analyses of traffic loading paths, while Ishikawa *et al.* (2011) report load observations from a one-fifth reduced scale model of ballasted Japanese railway tracks under repeated train passages. It is now clear that positive vertical ($\Delta\sigma_v$) and horizontal stress ($\Delta\sigma_h$) increments rise and fall following a zero-peak-zero pattern under travelling wheel loading, while the horizontal shear stress ($\Delta\tau_{hv}$) reverse cyclically as the load passes. Fig. 3 shows, after Lekarp *et al.* (2000) how the incremental stress path traces out a cardioid shape in the $\Delta\tau_{hv} - (\Delta\sigma_v - \Delta\sigma_h)/2$ plane and leads to cyclic principal stress axis rotations. The latter are likely to impact considerably on the soil's response (Ishihara *et al.* 1983).

We examine in this paper how idealised cyclic traffic loading might affect the straining developed in soft clay layers under undrained conditions as a first step to exploring its possible contribution to the service settlements of highways, runways, metro rail tracks. Undrained cyclic experiments that model the paths, including principal stress axis rotation, imposed by traffic loading provide the key evidence. We recognise that service performance includes an additional background drainage process in which excess pore pressures dissipate slowly but continuously leading to volumetric strains that add to the settlements caused by undrained shear straining. We anticipate that the associated void ratio and micro-structural changes will also gradually modify, over time, the soft clays' settlement trends and susceptibility to further undrained cyclic loading.

The effects of undrained cyclic shearing on clays have been investigated extensively through cyclic triaxial (CT) and cyclic Direct Simple Shear (DSS) tests by, among others, Brown *et al.* 1975; Andersen *et al.* 1980; Yasuhara *et al.* 1982; Hyde *et al.* 1993; Zhou & Gong 2001; Moses *et al.* 2003 and Li *et al.* 2011. CT tests are restricted to keeping any two of the three principal stresses equal while the major principal stress axis direction (angle α with respect to the vertical) is either 0° or 90° . True triaxial equipment allows the three principal stresses to be controlled independently, and the influence of the intermediate principal stress to be considered. However, only 90° jump rotations in principal stress direction can be achieved. Hollow Cylinder Apparatus (HCA) equipped with four degrees of load control freedom can overcome the above limitations, provided the tests have suitable geometry and do not develop large strains. HCA research into principal stress rotation effects includes inter alia that by Broms & Casbarian 1965; Hight *et al.* 1983; Towhata & Ishihara 1985; Symes *et al.* 1984; Vaid *et al.* 1990; Zdravkovic and Jardine 1997; Nakata *et al.* 1998; Yang *et al.* 2007; Nishimura *et al.* 2007 and Gräbe & Clayton 2009. These authors show that excess pore pressures, non-linear stiffness and shear strength can vary strongly when the principal stress directions are rotated under undrained conditions.

This paper reports cyclic triaxial (CT) and cyclic HCA (CHCA) experiments on K_0 -consolidated intact soft clay samples from Wenzhou, SE China that explore the clay's response to the traffic loading patterns shown in Fig. 3, over many repeated cycles. Cai *et al.* (2016) offered a preliminary summary of the samples' vertical strain development. This paper synthesises these data with further detailed observations regarding the shear strength behaviour of the clay and the accumulated cyclic pore water pressure trends as well as the resilient modulus and damping behaviours observed under cyclic loading.

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78 **Test equipment and test scheme**

79 **Test equipment**

80 The tests reported in this study were performed with a Dynamic Hollow Cylinder Apparatus (DYNHCA), manufactured by
81 GDS Instruments Ltd. The HCA, which tests specimens with 100 mm and 60mm outer and inner diameters (d_o and d_i) and
82 200mm height (H), is described by Gräbe & Clayton 2009; Cai *et al.* 2015 and Xiong *et al.* 2015 who give details of the
83 actuators, signal conditioning and cyclic control systems. Axial force and torque are measured by a dual-axis load
84 transducer mounted above the test specimen, while vertical displacements and rotation angles are measured through
85 encoders mounted on the actuator motor shafts. Digital controllers regulate the pressures and measure the volume
86 changes of the inner and outer chambers and the soil specimen. Cyclic pore-water pressure drift is measured through stiff
87 tubes that connect the top and base drainages to pressure transducers. Such external systems cannot measure the pore
88 pressure changes accurately over individual cycles under the 1Hz cycling described below.

89

90 **Stresses and strain conditions**

91 Hollow cylinder apparatus inevitably imposes a stress regime that is imperfectly uniform; the degree of non-uniformity
92 depends on the specimen geometry, the properties of the soil under test and the stress paths imposed. Hight *et al.* (1983)
93 argue that stress uniformity may be tolerable provided that the inner to outer diameter ratio is greater than or equal to 0.6
94 and $H/d_o > 1$. The GDS apparatus' H/d_o and d_i/d_o ratios of 2 and 0.6 respectively meet these conditions. Survey of 19
95 other peer-reviewed HCA studies reveals a practical d_i/d_o range of 0.54 to 0.85. While a d_i/d_o greater than 0.6 would be
96 desirable from a stress uniformity perspective, it would incur more damage when preparing natural intact samples and
97 also hinder system operability (Gräbe & Clayton 2009).

98 Fig. 4 shows, after Hight *et al.* (1983) the scheme of the average stress and strain conditions developed within test
99 specimens. Varying the vertical load (W), torque (M_T), inner cell pressure (p_i) and outer cell pressure (p_o) allows the
100 average vertical σ_z , radial σ_r , circumferential σ_θ and shear $\tau_{z\theta}$ stresses to be controlled independently. The equivalent strain
101 components are: the vertical ϵ_z , radial ϵ_r , circumferential ϵ_θ and shear $\gamma_{z\theta}$ strains. We adopt herein the Hight *et al.* (1983)
102 expressions summarized in Table 1 to calculate the average stresses and strains.

103

104 **Sampling method and soil properties**

105 The soft clay tested is from Wenzhou, SE China, whose widespread high compressibility, low permeability and low
106 bearing capacity clays have been characterised by Li *et al.* 2011; Gu *et al.* 2012; Guo *et al.* 2013; Wang *et al.* 2013, among
107 others. Multiple samples were taken from a 3 m deep excavation located 1 km from the Wenzhou High-Speed Beltway,
108 much of which rests on 1.5 to 2 m thick granular road-plus-embankment fill structures over soft clay profiles similar to that
109 at the sampling site. The ground water table was found about 0.8 m below the surface. Thin-walled (2mm thick) stainless
110 steel tubes (160 mm in diameter, 250 mm in height and with 8° cutting angles) were pressed gently into a horizontal pit
111 bench. Each tube was excavated with great care, sealed at both ends and stored in a temperature and humidity controlled
112 room.

113 Table 2 summarises the samples' typical index properties, while Fig. 5 presents a typical stage loaded oedometer test on
114 a retrieved sample. The vertical effective yield stress $\sigma'_{vy} \approx 30$ kPa, falls marginally above the in-situ vertical effective stress
115 (σ'_{v0}) ≈ 27 kPa, indicating light apparent overconsolidation, due perhaps to ageing in-situ or ground water variations.
116 Reconsolidation to $\sigma'_{v0} = 27$ kPa led to a void ratio of $\Delta e \approx 0.04$ e_0 allowing the samples' quality to be classified as "good"
117 according Lunne *et al.*'s (2006) criterion. The tests reported below involved consolidating the specimens to 74.5 kPa, well

above σ'_{vy} to model the conditions expected in soil elements positioned below typical 2 m high road-plus-embankment formations.

Test Procedures

Specimen preparation and K_0 -consolidation

Triaxial and HCA specimens were trimmed carefully after undergoing a gentle extrusion process. The procedure described by Zhou *et al.* (2007) was employed to sculpt the HCA specimens. Filter paper strips were placed over the specimens' outer surfaces to accelerate consolidation and reduce pore pressure response time, before installing inner and outer membranes and sealing O-rings. Specimens were saturated under a back pressure of 300 kPa for 24 hours, with an initial effective stress of 10 kPa (Point O' on Fig. 6(a)). The samples were saturated with a back pressure of 300 kPa and B value checks all exceeded 0.98. Li *et al.* (2011) report a mean $K_0 \approx 0.55$ for the clay and consolidation involved increasing the vertical stress alone by 0.05 kPa/min until the specimen stress states reached Point A on Fig. 6, with $\sigma'_z = 18.2$ kPa and $\sigma'_r = \sigma'_\theta = 10$ kPa. The effective stress path continued to $\sigma'_{z0} = 74.5$ kPa, $\sigma'_{r0} = \sigma'_{\theta 0} = 41$ kPa (Point B on Fig. 6(a)) with the same $K_0 \approx 0.55$, simulating conditions under a 2m thick road-plus-embankment. Consolidation was conducted under stress control with $d\sigma_z/dt = 2.5$ kPa/h and the final stresses were maintained for 72 hours under drained conditions (Point C on Fig. 6(a)) in order to achieve slow residual creep rates. Fig. 6(b) shows the variation of axial strain with time during K_0 -consolidation; the average axial strain rate reduced to 0.002%/h over the final hours of drained creep.

Similar K_0 consolidation stress paths were applied to the triaxial specimens whose diameters were 50 mm and heights 100 mm prior to conducting undrained triaxial compression and extension tests. The tests were performed under a fixed strain rate $\dot{\epsilon}_z = 0.01\%/min$. Fig. 7 presents the stress-strain relationships and effective stress paths. The K_0 -consolidated clay showed a typical strain softening response in both compression and extension. Peak q developed after approximately 900 minutes in compression, leading to an undrained strength S_u^{TC}/σ'_{v0} ratio = 0.34, while peak q developed at larger strains in extension after shearing for around 2300 minutes (at the same strain axial rate) giving S_u^{TE}/σ'_{v0} ratio = 0.28. The difference between the latter two ratios is comparatively modest and does not suggest that the Wenzhou clay will manifest particularly marked mechanical anisotropy. The critical state M and ϕ' values are interpreted to be 1.24, 31° (in compression) and 1.15, 36.8° (in extension) respectively.

Cyclic Test Scheme

Undrained cyclic tests were performed using both triaxial and HCA equipment. The first Series involved Cyclic Triaxial (CT) tests in which cyclic vertical stress increments σ_z^{cyc} were applied above the initial stresses with the waveform given in Fig. 8(a) while the radial stresses were held constant and no principal stress axis rotation took place. The second Series of Cyclic HCA tests varied both the vertical stress σ_z^{cyc} and the torsional shear stress $\tau_{z\theta}^{cyc}$, following the cyclic incremental waveforms shown in Fig. 8(b), which are defined as follows:

$$\sigma_z^{cyc} = \Delta \sigma_z^{max} \left[\frac{1}{2} \cos(2\omega t) - \cos(\omega t) + 0.5 \right] \quad (1)$$

$$\tau_{z\theta}^{cyc} = \Delta \tau_{z\theta}^{max} \left[\sin(\omega t) - \frac{1}{2} \sin(2\omega t) \right] \quad (2)$$

where $\omega = 2\pi/T$ ($T = 1$ s), and t is the elapsed time. The CHCA tests were intended to represent idealized versions of the traffic loading cycles outlined in Fig. 3. In this case the inner and outer effective confining pressures were kept constant and equal (at 41 kPa). As a result, the intermediate principal stress parameter $b = (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$ varied during cycling as α rotated, according to $b = \sin^2 \alpha$. This step may be helpful in reducing specimen stress non-uniformity.

Table 3 summarises the 25 cyclic tests presented in this study. The three tabulated cyclic loading parameters, which are defined below, are the vertical cyclic stress ratio (VCSR), the ratio between the maximum increments of torsional shear stress and vertical cyclic stress (η) and the cyclic stress ratio (CSR),

$$\text{VCSR} = \Delta\sigma_z'^{\max} / 2p_0' \quad (3)$$

$$\eta = \Delta\tau_{z\theta}^{\max} / \Delta\sigma_z'^{\max} \quad (4)$$

$$\text{CSR} = \frac{\sqrt{(\Delta\sigma_z'^{\max} - \Delta\sigma_\theta')^2 + (2\Delta\tau_{z\theta}^{\max})^2}}{2p_0'} = \text{VCSR} \sqrt{1+4\eta^2} \quad (5)$$

where, p_0' is the initial effective mean normal stress and can be written as $p_0' = (\sigma'_{z0} + \sigma'_{r0} + \sigma'_{\theta0})/3$, and $\Delta\sigma_\theta'$ is kept equal to zero in all the tests.

The loading frequency was set to 1 Hz in both Series to represent traffic loading (Ishihara 1996; Chazallon *et al.* 2006). Full sets of transducer output, with 50 data sets per cycle (one per 0.02s) were recorded over the first cycle and after each decadal cyclic number (that is when $N = 10, 20, 30$ etc). Most specimens were subjected to 10,000 loading cycles. Two CHCA tests failed at an earlier test stage; see Table 3. Noting that road loading may impose larger numbers of cycles, a single longer duration CT test was conducted that applied 10^5 cycles at $\text{CSR} = 0.124$. The data plotted in Fig. 9(a) indicate a linear $\log(\epsilon_p) - \log(N)$ trend over the 10^2 to 10^5 range, suggesting that for low CSR tests the effects of large numbers of cycles on the permanent strains follow a simple power-law relationship, in which the exponents are approximately 0.31 for the trough and 0.23 for the peak strain conditions respectively. For simplicity, Δu is normalized by the initial effective mean normal stress p_0' , and characterised by the ratio $\Delta r_u = \Delta u / p_0'$. Fig. 9(b) presents the $\log(\Delta r_u) - \log(N)$ relationship over the same range. Unlike the vertical strain development, the pore pressure appears to be stabilised after large number of cycles.

Test results and discussion

Total Stress paths

The main tests Series' involved undrained cycling. Fig. 10 shows as an example the total stress changes applied over each full cycle of the triaxial test CT 107 and the CHCA 208 experiment. The waveforms of the cyclic vertical stress and cyclic shear stress are as defined by Eqs. (1) and (2) and both tests were conducted with $\text{VCSR} = 0.143$. The target stress paths, which are shown as dashed lines in Fig. 10(a) confirm that the stress control system could impose the desired stress path accurately. The incremental CT test paths plot as horizontal lines in Fig. 10(b), where the equivalent CHCA test paths appear as cardioids whose areas increase with η .

The angle α can be calculated as:

$$\tan 2\alpha = \frac{2\tau_{z\theta}^{\text{cyc}}}{\sigma_z'^{\text{cyc}} + \sigma_{z0}'} \quad (6)$$

It can be seen in Fig. 10 (b) that α rotates continuously over the $\pm 8^\circ$ range in this particular CHCA test, while the major principal stress axis remains vertical in the CT test.

Development of pore pressure

The full set of experimental trends for accumulated cyclic excess pore water pressures Δu are presented in Fig. 11 (a) to

(i) on arithmetic scales with one plot for each of the ten VCSRs listed in Table 3 that combines the CT and CHCA results. The pore pressure trends cover 10,000 cycles for all tests, except the $\eta = 1/3$ cases at the two highest VCSR, which failed at lower numbers of cycles. Pore pressure development was negligible at the lowest VCSR (Fig. 11a) and then grew with VCSR for any particular test type. Initially the CT and CHCA tests developed similar pore pressure ratio trends, although their trends dispersed with increasing VCSR and with increasing η . The latter trends are compatible with earlier studies on other geomaterials; see for example Symes *et al.* 1984 and Yang *et al.* 2007.

The results are synthesised in Fig. 12 by plotting the Δr_u values determined after 100 cycles and 10,000 cycles for the 22 non-failing CT and CHCA tests listed in Table 3. When plotted against the CSR ratio defined in Eq. 5, the CT and CHCA tests follow the same patterns. The interpretation shown divides the clay samples' responses into three phases of behaviour. Up to $\text{CSR} \approx 0.03$, the pore pressure response is considered negligibly small and so fully *Stable*. Between this lower limit and $\text{CSR} \approx 0.16$, the incremental Δr_u - CSR relationship is nearly linear: the pore pressure ratios are modest and their rates of increase slow markedly with increasing N . In this context, we term this phase of behaviour *Metastable*. The rates of development increase sharply at $\text{CSR} > 0.16$, indicating an *Unstable* pattern that would most probably lead to failure provided the cycling was continued to sufficiently large numbers of cycles.

Considering the two tests that failed under cycling, the CHCA tests with $\eta = 1/3$ at $\text{VCSR} = 0.201$ and 0.228 failed under fast 1 Hz cycling at $N < 10,000$ with maximum $(\sigma'_1 - \sigma'_3)/2$ values of 27.3 kPa and 28.8 kPa that can be compared with the static triaxial compression $(\sigma'_1 - \sigma'_3)/2$ value (or s_u) of 25.5 kPa given in Fig. 8, which developed after 900 minutes of shearing at a rate that was approximately 10^5 time slower. It appears that the positive effect on undrained shear strength of applying a fast cycling rate offset the combined negative effects of cyclic degradation and principal stress axis rotation. Fig. 13 presents the typical fast (1 Hz) cyclic effective stress paths plotted in the q - p' plane for tests that showed stable ($\text{CSR} = 0.029$), metastable ($\text{CSR} = 0.151$) and unstable ($\text{CSR} = 0.274$) responses, with the slow static triaxial (CAU) compression effective stress path and CSL line shown for reference. We acknowledge that the 1Hz cyclic pore pressure measurements may not be representative at the peaks and troughs of each cycle, due to instrument response times. However, the overall drift trends show the stable cyclic test paths remaining almost static and falling well below the critical state line (CSL) and the CAU test's effective stress path. However, the unstable fast paths cross these boundaries; the metastable test showed an intermediate style of behaviour.

Permanent strain evolution

We move next to consider the experiments' strain response. Fig. 14 presents first the general scheme by which the cyclic vertical ε_z and torsional shear strains $\gamma_{z\theta}$ are reported; ε_z^t is the total vertical strain, ε_z^r the resilient strain and ε_z^p is the additional permanent strain developed over single cycle, with similar rules applying to $\gamma_{z\theta}$. Note also that in undrained tests the radial and circumferential strains must sum to zero with the vertical strain, but are not considered in detail herein.

The relative magnitudes of the strain components are explored in Fig. 15 by plotting the cyclic $\sigma'_z - \varepsilon_z$ and $\tau_{z\theta} - \gamma_{z\theta}$ responses from a relatively high VCSR ($= 0.172$) test involving the highest $\eta = 1/3$ ratio (where the $\tau_{z\theta}$ shear stress changes were $1/3$ of the vertical stress increments). It is clear that the vertical strains dominate over the 10^4 cycles, with ε_z growing steadily over the whole test to reach 1.6% while the torsional shear strains remained below 0.2% and showed negligible drifted after the first 1000 cycles.

The evolution of vertical strain with N is summarized in Fig. 16 (a) to (j) for the 24 cyclic tests. As shown in Fig. 16 (a) and (b) permanent vertical strains of around 0.01% developed in both the CT and CHCA tests conducted at the lowest VCSRs. Recalling that the drained vertical creep rates prior to shearing were $\approx 0.002\%/h$, up to 0.006% additional creep could have been expected over the 3 hours required to complete each batch of 10^4 cycles even if no cycling had been imposed. We

therefore consider the permanent strain rates as being negligible in these four experiments. As with the pore pressure trends discussed above, the permanent strain rates grew with increasing VCSR and the CHCA trends became progressively steeper than the CT equivalents. All of the CT tests showed permanent strain gradients $d\varepsilon_z^p/dN$ that declined monotonically with N . The same feature was seen in the CHCA tests up to VCSR = 0.172, above which the higher η ($\eta = 1/3$) experiments developed failure at $N < 10,000$.

The permanent strain trends are synthesised in Fig. 17 by plotting the permanent vertical strains determined after 100 cycles and 10,000 cycles for the 22 non-failing tests, mirroring the Δr_u plots presented in Fig. 11. As with pore pressure, CT and CHCA tests follow the same patterns when plotted against CSR. In a similar way, our interpretation divides the permanent strain responses into a fully *Stable* phase up to CSR ≈ 0.03 , and a *Metastable* region over $0.03 < \text{CSR} < 0.16$ in which the $d\varepsilon_z^p$ - CSR relationship is nearly linear for each N value. The strains remain modest and the rates of increase slow markedly with increasing N . In this context, we term this phase of behaviour *Metastable*. As with the pore pressures, the rates of development increase sharply at CSR > 0.16 , indicating an *Unstable* pattern that would most probably lead to failure provided the cycling was continued to sufficiently large numbers of cycles. The metastable permanent strain trends data are plotted against N with semi-log and fully logarithmic axes in Fig. 18. The fully logarithmic plots show approximately constant gradients at once N exceeds around 100, confirming near power-law relationships between the permanent strains and number of cycles.

Guo (2013) shows that empirical functions can be fitted to the data on the basis of Figs. 17 and 18 to describe the permanent displacement trends accumulated in the metastable range when $N > 100$ that have the form:

$$\varepsilon_z^p = a (\text{CSR} - b) [N/100]^c \quad (7)$$

Where $a (\text{CSR} - b)$ is the permanent strain developed after 100 cycles, with values of $a = 1.7$ and $b = 0.03$ being found from analysis of Fig. 17. Reference to Fig. 18 indicates that the exponents c range from 0.3 to 0.4 and increase with the severity of cycling. Similar functions could be found to fit the cyclic pore pressure ratio trends reported in Fig. 12.

Resilient strains and modulus trends

The trends for the resilient strains defined in Fig. 14 can also be distinguished from Fig. 16. At low VCSRs the strains remain almost constant with N ; tests at higher VCSRs showed the resilient strains growing with N . It is also clear that the vertical resilient strain trends of CT and CHCA tests diverged with increasing VCSRs, with principal stress rotation increasing the vertical straining markedly. For example, the resilient strains found in CHCA tests with $\eta = 1/6$ and $1/3$ at VCSR = 0.143 are 0.06% and 0.08% at $N = 10,000$, respectively, 22% and 52% greater than those developed in the corresponding CT test (0.05%).

Noting that the cycles all imposed constant cyclic loads, the resilient strain response can be summarised conveniently by considering the trends for vertical resilient modulus, which is defined in Fig. 8 (following Seed *et al.* 1966) as $M_z^r = \Delta\sigma_z'^{\max}/\varepsilon_z^r$. Fig. 19 illustrates the typical trends seen in CT and CHCA tests conducted at VCSR = 0.172. First we see in Fig. 19(a) the decline in M_z^r with N seen at higher CSR and also the divergence between the CT and CHCA trends. Next, the relationship between mean effective stress reduction caused by pore pressure development and loss of M_z^r as cycling continued is made clear in Fig. 19(b). These aspects are clearly closely correlated.

The resilient modulus trends are synthesised in Fig. 20 by plotting the M_z^r values applying after 100 cycles and 10,000 cycles for the 22 non-failing tests, mirroring the Fig. 12 and 17 plots. Here too we see the CT and CHCA tests following the same patterns when plotted against CSR. As before, the resilient moduli can be divided up into a notionally *Stable* portion

up to $CSR \approx 0.03$ in which stiffness is at a maximum, a *Metastable* $0.03 < CSR < 0.16$ phase in which the M_z^r - CSR relationship declines on a nearly linear incremental relationship for both N values, and finally a trend towards more steeply falling and *Unstable* behaviour at higher CSR.

Evolution of hysteretic vertical stress-strain loops

The final cyclic attribute that we consider from the tests is the evolution of the hysteretic stress strain loops. Two typical CHCA tests (203 and 210) are considered in Fig. 21 which imposed $VCSR = 0.029$ and 0.172 respectively and correspond to the normalised incremental stress ratios expected under multi-vehicle loading at depths below road level of 6m and 2m, in the illustrative example shown in Fig. 1(c). Note that the horizontal strain scales adopted for Figs. 21 (b) and (d) correspond to the strain increments with their strain origins being re-zeroed at the start of each cycle. We recall that the experimental load waveform shown in Fig. 8 (b) led to vertical stress decreasing slightly over the first part of each cycle. The resulting incremental strain responses illustrated in Fig. 21 (a) and (b) indicate some time and strain rate dependency in the clay's response, which we cannot address in this paper, that leads to rounding of the stress-strain curves on unloading and re-loading.

The overall vertical stress-strain response illustrated in Fig. 21 (a) and (c) highlights the evolution of permanent strain development, while Figs. 21 (b) and (d) show how the hysteretic vertical stress-strain loops evolve as the cycling progresses to the $N = 10, 100, 1000$, and $10,000$ stages, showing a clear and progressive loss of stiffness. The latter aspects of the CT and CHCA tests are explored further in Fig. 22, considering again the $VCSR = 0.172$ cases. As in Fig. 22, the loops all resemble inclined ellipses whose long axes tend to rotate clockwise as N increases, indicating gradual loss of stiffness in conjunction with pore pressure increases and reductions in effective stress. The unloading stage of the stress-strain curves do not coincide with the loading stages and the loops encompass areas in the $\sigma'_z - \varepsilon_z$ plane which increase with η at fixed $VCSR$, indicating increasing energy dissipation, which can be expressed in terms of a damping ratio D computed for each of the cycles considered.

The damping ratio trends from the non-failing tests are synthesised in Fig. 23 by plotting the D values found for the 100^{th} cycles and $10,000^{\text{th}}$ cycles for the 22 non-failing tests. As with all the other cyclic attributes, the CT and CHCA tests follow the same three-stage pattern when plotted against CSR. The damping ratio trends can also be interpreted as showing: first low, notionally *Stable* values, up to $CSR \approx 0.03$; next a *Metastable* $0.03 < CSR < 0.16$ phase in which the incremental D - CSR relationship is practically linear for both N values; and finally a trend towards far higher and *Unstable* ratios at higher CSR.

Clay response and shakedown behaviour

Overall, the CT and HCA test responses indicated a similar three phase pattern when correlated with CSR. We suggest that the critical CSR ranges identified from the experiments can be interpreted within a similar framework to the shakedown treatment of engineering structures under cyclic loading. The original shakedown concept proposes four categories of response under repeated loading: purely elastic, elastic shakedown, plastic shakedown and incremental collapse (Johnson 1986). We suggest that similar categories apply to the observed soft clay response:

- (1) Elastic: where the cyclic stress remains within the small Y_1 region, as described for example by Jardine (2014). This region behaviour could not be resolved unambiguously within the present experiments.
- (2) Elastic shakedown: where higher levels of cyclic loading lead to the specimen undergoing first Y_1 yielding but remaining within the Y_2 kinematic yield surface reported by Jardine (2014) and exhibiting slight plastic deformation such as that seen up to the $CSR \geq 0.03$ limit.
- (3) Plastic shakedown: with the increase of cyclic loading the specimen undergoes Y_2 yielding and the degree of plastic

deformation becomes progressively more important and permanent deformations accumulate markedly over the initial cycles. However, the Y_2 kinematic surface gradually adjusts its position and size and the specimen gradually achieves a more stable response to continued cycling. This behaviour appears to correlate with the metastable $0.03 < CSR < 0.16$ range.

- (4) Incremental collapse: which commences once the cyclic stress paths engage the outer Y_3 yield surface described by Jardine (2014), the specimen undergoes sharp yielding and progressively poorer cyclic behaviour as observed at $CSR \geq 0.16$.

Discussion on possible application to settlement prediction for highway design

The cyclic experiments on Wenzhou soft clay suggest several key points regarding possible future application in practical highway engineering. Firstly, the fully *Stable* CSR limit is very low and it is probably impossibly conservative to aim to keep all road cyclic loading within this fully stable region of soft clay foundation response. Economic road design then hinges on keeping the extreme traffic loading paths within a tolerable part of the *Metastable* region. Detailed surveys are required to assess how frequently extreme parallel truck loading events occur in highway service.

Cyclic tests such as those reported above can then be undertaken to define optimal road plus embankment thicknesses that would allow suitable road lives under the anticipated numbers of extreme cycles imposed by multiple parallel heavy trucks. The example shown in Fig. 1 indicated that the extreme cyclic stress conditions beneath 2m embankments are likely to generate cycling with $CSR \approx 0.1$. The Authors' tests indicate that vertical strains amounting to approximately 0.4% could be expected after 10^4 such extreme cycles under undrained conditions. Considering an average strain of 0.2% over the 2 m to 6 m depth range below road surface suggests around 8mm of settlement over 10,000 extreme load cycles within the soft clay layer. Consideration will also be required of the additional effects on settlement and stiffness trends of the cyclically induced pore water pressures draining slowly over the roads' lifetime. These features could be explored in further programmes of CT and CHCA tests. It is important to recall from Fig. 1(c) that the imposed CSR values decrease sharply with depth. Even marginal increases in road structure or embankment thicknesses will reduce the stress increments significantly and improve the road's ability to carry vehicle loading.

It is also worth noting that the experiments reported in this paper all considered soil elements that were normally consolidated, aged for a moderate period and tested without experiencing any pre-cycling. Aghakouchak *et al.* (2015) show that applying more extended ageing, pre-stressing and/or drained pre-cycling can improve the behaviour of soils under any subsequent cyclic loading. Highway construction procedures that apply any such measures could be effective in reducing the in-service settlement performance settlements referred to in this paper's introduction. These facts of behaviour could also be explored through further advanced cyclic testing.

Summary and conclusions

Cyclic triaxial (CT) and cyclic hollow cylinder apparatus (CHCA) experiments have been conducted on K_0 -consolidated samples of an intact soft clay to explore their response to cyclic loading. The CT tests involved vertical stress cycling with no principal stress axis rotation. In the CHCA tests, the stress path induced by wheel loading is simulated by applying combined axial load and torque through user-defined waveforms that also rotate the major principal stress axis continuously by up to $\pm 8^\circ$ range, leading to 'cardioid' total stress paths whose areas increase with VCSR and η . Six main conclusions follow from the study:

- (1) Cyclic principal stress axis rotation has significant influence on the vertical strain behaviour of the soft clay specimens.

Both resilient strain and permanent strain in CHCA tests are larger than their counterparts in CT tests conducted with

the same value of VCSR. The divergence in the vertical straining developed under these different stress paths widens with VCSR and shear stress level η .

- (2) Cyclic principal stress rotation accelerates the accumulation of pore water pressure and cyclic degradation, leading to different vertical stress-strain hysteresis loops in CT and CHCA tests. The latter are softer and decline more rapidly with N . Both trends become more acute as VCSR and η increase.
- (3) The responses of the CT and CHCA tests can be brought together by plotting the cyclic attributes as functions of a CSR that accounts for the full set of cyclic stress components.
- (4) For $CSR < 0.03$, pore pressure development ratio Δr_u can be considered as negligibly small and the soil response *Stable*. Over the $0.03 < CSR < 0.16$ *Metastable* range, the pore pressures increase practically linearly with CSR. The response to cycling becomes markedly more sensitive to CSR and the response is considered *Unstable* at $CSR > 0.16$. An upper CSR limit was identified at $CSR > 0.24$ above which full failure developed after a limited number of cycles.
- (5) In common with the permanent strain trends, the experimental measurements of permanent vertical strain, resilient modulus M_z^r and damping ratio D all support the classification of the cyclic response into stable ($CSR \leq 0.03$), metastable ($0.03 < CSR < 0.16$) and unstable ($0.16 \leq CSR < 0.24$) regions. These critical limits can be associated with the onset of four categories of shakedown response to cyclic loading.
- (6) The experimental results may be used to provide estimate how severe traffic loading, such as that outlined in Fig. 1, would affect soft clay layers under undrained conditions. The key results indicate how empirical expressions can be defined from the test data that provide good representations over the metastable range that is most important to practical highway or rail engineering. More elaborate tests can be designed to consider the influence of additional factors such as pre-cycling and pore pressure dissipation in service.

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List of symbols

τ_{vh}	shear stress induced by vehicle loading in subsoil
$\Delta\sigma_v$	vertical stress induced by vehicle loading in subsoil
$\Delta\sigma_h$	horizontal stress induced by vehicle loading in subsoil
d_o	outer diameter of clay specimen
d_i	inner diameter of clay specimen
H	height of clay specimen
W	vertical load
M_T	Torque
p_i, p_o	inner and outer cell pressure
$\sigma_z, \sigma_r, \sigma_\theta, \tau_{z\theta}$	vertical, radial, circumferential and shear stresses
$\varepsilon_z, \varepsilon_r, \varepsilon_\theta, \gamma_{z\theta}$	vertical, radial, circumferential and shear stresses
σ'_{v0}	in-situ vertical effective stress
σ'_{vy}	vertical effective yield stress
e	void ratio of clay specimen
K_0	coefficient of earth pressure
S_u^{TC}	triaxial compression strength
φ'	effective internal friction angle
b	intermediate principal stress parameter
α	orientation of major principal stress relative to vertical axis
VCSR	vertical cyclic stress ratio
η	ratio between shear stress and vertical cyclic stress
CSR	cyclic stress ratio
Δu	pore water pressure
Δr_u	pore water pressure ratio
p'_0	initial effective mean normal stress
M_z^r	resilient modulus
D	damping ratio
ε_z^t	total vertical strain
ε_z^r	resilient vertical strain
ε_z^p	permanent vertical strain
N	cycle number

Table Captions

Table 1 Equations for data interpretation

Table 2 Index properties of tested Wenzhou soft clay

Table 3 Summary of two series tests

Table 1 Equations for data interpretation

	Stresses	Strains
Vertical	$\sigma_z = \frac{W}{\pi(r_o^2 - r_i^2)} + \frac{p_o r_o^2 - p_i r_i^2}{(r_o^2 - r_i^2)}$	$\varepsilon_z = \frac{z}{H}$
Circumferential	$\sigma_r = \frac{p_o r_o + p_i r_i}{r_o + r_i}$	$\varepsilon_\theta = -\frac{u_o + u_i}{r_o + r_i}$
Radial	$\sigma_\theta = \frac{p_o r_o - p_i r_i}{r_o - r_i}$	$\varepsilon_r = -\frac{u_o - u_i}{r_o - r_i}$
Shear	$\tau_{z\theta} = \frac{T}{2} \left[\frac{3}{2\pi(r_o^3 - r_i^3)} + \frac{4(r_o^3 - r_i^3)}{3\pi(r_o^2 - r_i^2)(r_o^4 - r_i^4)} \right]$	$\gamma_{z\theta} = \frac{\theta(r_o^3 - r_i^3)}{3H(r_o^2 - r_i^2)}$
Major principal	$\sigma_1 = \frac{\sigma_z + \sigma_\theta}{2} + \sqrt{\left(\frac{\sigma_z - \sigma_\theta}{2}\right)^2 + \tau_{z\theta}^2}$	$\varepsilon_1 = \frac{\varepsilon_z + \varepsilon_\theta}{2} + \sqrt{\left(\frac{\varepsilon_z - \varepsilon_\theta}{2}\right)^2 + \gamma_{z\theta}^2}$
Intermediate principal	$\sigma_2 = \sigma_r$	$\varepsilon_2 = \varepsilon_r$
Minor principal	$\sigma_3 = \frac{\sigma_z + \sigma_\theta}{2} - \sqrt{\left(\frac{\sigma_z - \sigma_\theta}{2}\right)^2 + \tau_{z\theta}^2}$	$\varepsilon_3 = \frac{\varepsilon_z + \varepsilon_\theta}{2} - \sqrt{\left(\frac{\varepsilon_z - \varepsilon_\theta}{2}\right)^2 + \gamma_{z\theta}^2}$

Table 2 Index properties of tested Wenzhou soft clay

Index properties	Value
Specific gravity, G_s	2.71
Natural water content, w_n (%)	60 ~ 62
Initial density, ρ_0 (g/cm ³)	1.61 ~ 1.64
Initial void ratio, e_0	1.64~1.71
Liquid limit, w_L (%)	60
Plasticity index, I_p	32
Clay fraction, (%)	41
Silt fraction, (%)	55

Table 3 Summary of two series tests

Series	Test ID	$\Delta\sigma_z^{\max}$ (kPa)	$\Delta\tau_{z\theta}^{\max}$ (kPa)	VCSR	CSR	η	No. of cycles, N
	001	13	0	0.124	0.124	0	100,000
1: CT	101	0.6	0	0.005	0.005	0	10,000
	102	1.5	0	0.014	0.014	0	10,000
	103	3	0	0.029	0.029	0	10,000
	104	6	0	0.057	0.057	0	10,000
	105	9	0	0.086	0.086	0	10,000
	106	12	0	0.114	0.114	0	10,000
	107	15	0	0.143	0.143	0	10,000
	108	18	0	0.172	0.172	0	10,000
	109	21	0	0.201	0.201	0	10,000
	110	24	0	0.228	0.228	0	10,000
2: CHCA	201	0.6	0.2	0.005	0.006	$1/3$	10,000
	202	1.5	0.5	0.014	0.017	$1/3$	10,000
	203	3	1	0.029	0.035	$1/3$	10,000
	204	6	2	0.057	0.069	$1/3$	10,000
	205	9	3	0.086	0.103	$1/3$	10,000
	206	12	4	0.114	0.137	$1/3$	10,000
	207	15	2.5	0.143	0.151	$1/6$	10,000
	208	15	5	0.143	0.172	$1/3$	10,000
	209	18	3	0.172	0.181	$1/6$	10,000
	210	18	6	0.172	0.207	$1/3$	10,000
	211	21	3.5	0.201	0.212	$1/6$	10,000
	212	21	7	0.201	0.242	$1/3$	9,060
	213	24	4	0.228	0.240	$1/6$	10,000
	214	24	8	0.228	0.274	$1/3$	3,740

- Fig. 1 Illustration of vertical stress increments imposed on soft clay subgrade by heavy traffic on a low embankment road
- Fig. 2 Schematic diagram of various stress states beneath a rolling wheel
- Fig. 3 Schematic diagram of typical stress paths induced by traffic load in the stress space of deviator stress and torsional shear stress
- Fig. 4 Stress and strain states in hollow cylindrical specimen
- Fig. 5 Oedometer compression curve of samples from 3 m at Wenzhou clay sampling site
- Fig. 6 Schematic diagrams for K_0 consolidation: (a) loading sequence; (b) strain development
- Fig. 7 Results of triaxial compression and extension tests: (a) stress-strain relationship; (b) effective stress path
- Fig. 8 Sketches of incremental loading waveforms in two Series: (a) CT tests; (b) CHCA tests
- Fig. 9 Development of vertical strain and pore water pressure versus cycle numbers during 100,000 cycles (CSR = 0.124 and N=100,000)
Vertical strain; (b) pore water pressure
- Fig. 10 Example of stress conditions in CT and CHCA tests conducted at VCSR = 0.143: (a) Total stresses versus elapsed time; (b) Incremental total stress paths
Note: Point O denotes state after K_0 consolidation
- Fig. 11 Pore water pressure buildup in CT and CHCA tests
- Fig. 12 Excess pore pressure after $N = 100$ and 10,000 cycles versus CSR
- Fig. 13 Typical effective stress paths for (a) a stable test (CSR = 0.029); (b) a metastable test (CSR = 0.151); (c) an unstable test (CSR = 0.274) in q - p' space
Note: The effective stress paths are based on external pore pressure measurements.
- Fig. 14 Typical strains developed in a single cycle: (a) vertical strain in CT test; (b) vertical strain in CHCA test; (c) shear strain in CHCA test
- Fig. 15 Development of stresses and strains for test of VCSR = 0.172 and $\eta = 1/3$: (a) $\sigma_z - \varepsilon_z$; (b) $\tau_{z\theta} - \gamma_{z\theta}$; (c) ε_z^p & $\gamma_{z\theta}^p - N$
- Fig. 16 Vertical strain evolutions in CT and CHCA tests
- Fig. 17 Vertical strain after $N = 100$ and 10,000 cycles versus CSR
- Fig. 18 Permanent vertical strain in CT and CHCA tests: (a)(c) semi-log axes; (b)(d) double log axes
- Fig. 19 Vertical resilient modulus in CT and CHCA tests (VCSR = 0.172): (a) vertical resilient modulus versus number of cycles; (b) vertical resilient modulus versus effective mean principal stress
- Fig. 20 Resilient modulus after $N = 100$ and 10,000 cycles versus CSR
- Fig. 21 Typical vertical stress-strain hysteresis loops in VCSR = 0.029, $\eta = 1/3$ and VCSR = 0.172, $\eta = 1/3$: (a)(c) complete vertical stress-strain loops; (b) (d) vertical stress-strain loops at different cycles
- Fig. 22 Vertical stress-strain hysteresis loops at different cycles in CT and CHCA tests (VCSR = 0.172): (a) $N=10$; (b) $N=100$; (c) $N=1,000$; (d) $N=10,000$
- Fig. 23 Damping ratio after $N = 100$ and 10,000 cycles versus CSR

(a) Typical cross section of a low embankment road carrying four axle trucks

(b) Wheel loading acting by three static 10 wheel, four axle, trucks positioned side by side on parallel highway lanes (unit: mm)

(c) Variation with depth of vertical stress increment $\Delta\sigma_z$ developed under Point X on

Fig. 1(b) by three stationary 31-ton trucks positioned as shown above

Fig. 1 Illustration of vertical stress increments imposed on soft clay subgrade by heavy traffic on a low embankment road

Fig. 2 Schematic diagram of various stress states beneath a rolling wheel

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(a) Vertical strain; (b) pore water pressure

Fig. 11 Pore water pressure buildup in CT and CHCA tests

Fig. 12 Excess pore pressure after $N = 100$ and 10,000 cycles versus CSR

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 Note: The effective stress paths are based on external pore pressure measurements.

Fig. 14 Typical strains developed in a single cycle: (a) vertical strain in CT test; (b) vertical strain in CHCA test; (c) shear strain in CHCA test

Fig. 15 Development of stresses and strains for test of VCSR = 0.172 and $\eta = 1/3$

(a) $\sigma_z - \varepsilon_z$; (b) $\tau_{z\theta} - \gamma_{z\theta}$; (c) ε_z^p & $\gamma_{z\theta}^p - N$

Fig. 16 Vertical strain evolutions in CT and CHCA tests

Fig. 17 Vertical strain after $N = 100$ and 10,000 cycles versus CSR

Fig. 18 Permanent vertical strain in CT and CHCA tests
(a)(c) semi-log axes; (b)(d) double log axes

Fig. 19 Vertical resilient modulus in CT and CHCA tests (VCSR = 0.172)
 (a) vertical resilient modulus versus number of cycles;
 (b) vertical resilient modulus versus effective mean principal stress

Fig. 20 Resilient modulus after $N = 100$ and 10,000 cycles versus CSR

Fig. 21 Typical vertical stress-strain hysteresis loops in $VCSR = 0.029$, $\eta = 1/3$ and $VCSR = 0.172$, $\eta = 1/3$

- (a) (c) complete vertical stress-strain loops;
(b) (d) vertical stress-strain loops at different cycles

Fig. 22 Vertical stress-strain hysteresis loops at different cycles in CT and CHCA tests
(VCSR = 0.172)

(a) $N=10$; (b) $N=100$; (c) $N=1,000$; (d) $N=10,000$

Fig. 23 Damping ratio after $N = 100$ and $10,000$ cycles versus CSR