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Effect of Variable Confining Pressure on Cyclic Triaxial Behavior of K_0-consolidated Soft Marine Clay

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Abstract

The effect of variable confining pressure (VCP) on the cyclic deformation and cyclic pore water pressure in K_0-consolidated saturated soft marine clay were investigated with the help of the cyclic stress-controlled advanced dynamic triaxial test in undrained condition. The testing program encompassed three cyclic deviator stress ratios, CSR=0.189, 0.284 and 0.379 and three stress path inclinations η^{ampl}=3, 1 and 0.64. All tests with constant confining pressure (CCP) and variable confining pressure (VCP) have identical initial stress and average stress. The results were analyzed in terms of the accumulative normalized excess pore water pressure r^q_u recorded at the end of each stress cycle and permanent axial strain, as well as resilient modulus. Limited data suggest that these behavior are significantly affected by both of the VCP and CSR. For a given value of VCP, both of the pore water pressure r^q_u and permanent axial strains are consistently increase with the increasing values of CSR. However, for a given value of CSR, the extent of the influence of VCP and the trend is substantially depend on the CSR.

Keywords: Soft Marine Clay; K_0-Consolidated; Cyclic Stress Ratio; Stress Paths; Cyclic Deformation; Pore Water Pressure.

1. Introduction

Engineering structures founded on soft marine clay deposits are often designed to withstand cyclic loading such as earthquakes, traffic, or ocean waves etc. during their operational life. Due to the low permeability of soft clays, cyclic loading is essentially undrained. When fully saturated soils are subjected to cyclic loading in undrained conditions involving moderate and large cyclic stress levels, their structure is permanently altered, the generation of excess pore-water pressure and thus to a deterioration of bearing capacity and soil stiffness [1-3]. In undrained conditions, cyclic degradation of stiffness and cyclic pore water pressure changes are among the most important phenomena in soil dynamics [4]. The degradation of soils will then significantly influence the development of the accumulation of plastic strains and resilient stiffness. Both of these can significantly reduce the life of the engineering structures.

A number of investigations have considered the effects of cyclic loading on soil behavior and simulated using the cyclic triaxial or simple shear apparatuses, as widely reported in the literature [5-13]. As these studies only applied a single deviator stress or horizontal shear stress, thus relatively few have addressed the coupling effects of the vertical and horizontal stress changes to which soft clays are subjected. Actually, the in situ stress fields in soil layers induced by earthquakes or traffic loads are combinations of varying normal and shear stresses [14]. Field measurements have also shown that, although vertical stress changes attenuate with depth, horizontal stress changes become more pronounced up to a certain depth below the sleeper base before then reducing with further increases in depth [15].

Cyclic triaxial tests with a simultaneous variation of the deviator stress q^{ampl} and confining pressure σ_3^{ampl} (VCP) can be used to simulate the coupling effect of cyclic vertical and horizontal stresses induced by earthquakes or traffic loading

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approximately. Rascol [16] conducted a series of undrained VCP and CCP tests to study the influence of cyclic normal stresses on the dynamic behavior of saturated sands in earthquakes and showed that the VCP stress paths strongly influence the cyclic degradation and pore water pressure, saturated sands is much easier to liquefy under certain VCP stress paths. In the case of traffic loads on saturated materials, three types of CCP and VCP tests with identical maximum stress, identical average stress, and identical initial stress were conducted. For example, Allen and Thompson (1974) carried out a comparative study between the CCP and VCP tests using stress paths that had identical values of maximum stresses (p^{max}, q^{max}) and showed that CCP tests yielded larger permanent strains than VCP tests [17]. Brown and Hyde (1975) argued that comparative cyclic stress paths with an identical average stress (i.e., identical values of p^{av} and q^{av}) were more meaningful and they concluded that the accumulation of permanent strains was similar for the two types of tests [18]. Rondón et al. (2009) conducted tests using the comparative method proposed by Brown and Hyde (1975). While, they found that VCP stress paths delivered different permanent strains in comparison with CCP stress paths and argued that CCP tests might underestimate the settlement induced by traffic loading in some cases [19].

In more recent work on saturated soft clays [14], very large increases in cyclic strain and magnitude of cyclic strength have been observed, associated with cyclic pore water pressure increases, when cyclic confining pressure have been applied with identical initial stress (p^0,q^0) in undrained conditions to simulate seismic loading. Cai et al. (2013) conducted a series of undrained and partially drained one-way cyclic triaxial tests on remolded saturated clay under various VCP and CCP stress paths with identical initial stress (p^0, q^0) to simulate traffic loading and observed that the cyclic confining pressure has great influences on the characteristics of pore water pressure and axial strain under both undrained and partially-drained conditions [20]. Especially under partially-drained conditions, the cyclic confining pressure promotes the accumulation of axial strain significantly [21]. However, some studies concluded VCP could promote the development of permanent deformation compared to the tests with CCP, while others drew the opposite conclusions. Even the same comparative tress paths could yield contradictive conclusions. Sun et al. (2017) conducted the above three types of comparative stress paths for CCP and VCP tests to study the effect of VCP on the permanent deformation of sand subjected to cyclic loading in free-to-drain conditions, and showed that VCP tests generate more permanent strains than CCP tests when they have identical maximum or average stress, while, CCP and VCP tests with identical initial stress accumulate similar permanent strains [22].

The literature listed above revealed that, no consensus has been reached on the coupling effect of cyclic deviator stress and cyclic confining pressure on the cyclic behavior of soils, apart from the cyclic stress amplitude and the number of cycles, the cyclic deformation and cyclic pore water pressure in clays are significantly affected by the type of VCP tests and drained conditions. Although a number of investigations have considered the coupling effects of cyclic deviator stress and cyclic confining pressure on soil behavior, but the cyclic deformation and cyclic pore water pressure changes have not been adequately investigated in undrained conditions, relatively few have addressed the coupling effects of cyclic deviator stress and cyclic confining pressure on intact soft clays in a K_0-consolidation condition in their in-situ sedimentary process in undrained conditions.

The aim of the research reported in this paper was to investigate these; the primary topic of this paper is how the cyclic deformation behaviour (i.e., permanent axial strain, resilient modulus) and the accumulative normalized excess pore water pressure of K_0-consolidated soft clay is affected by the VCP. Knowing the extent of the effect and the trend is important in the evaluation of the dynamic properties of soil for the analyses of cyclic response of soil deposits.

2. Test Equipment and Test Scheme

The laboratory tests were carried out using an advanced cyclic triaxial device, which was designed and manufactured by GDS corporation in Great Britain described by Gu et al.(2016) [23]. In this apparatus, the vertical stress is applied by a servo-loading system, while the confining pressure is supplied through an oil pressure type of piston. Images of the cyclic triaxial device and schematic diagram of the apparatus is shown in Figure 1.

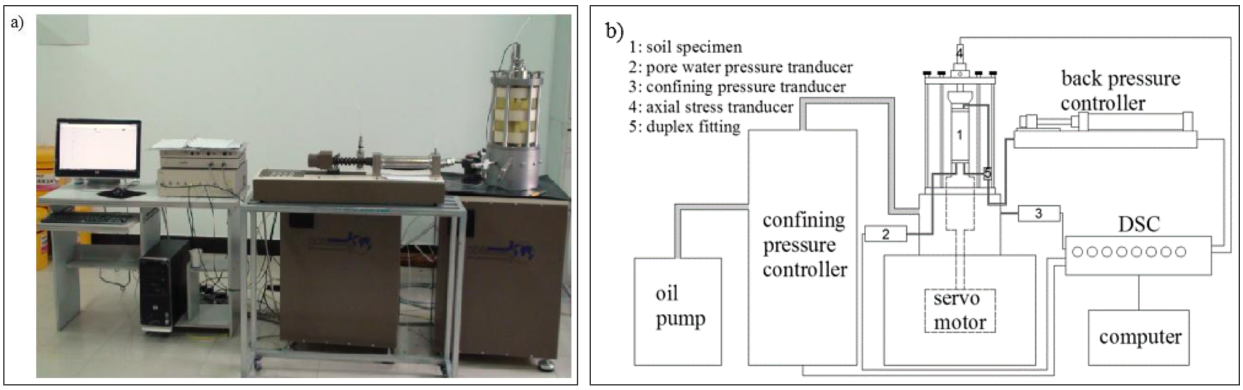


Figure 1. (a) General view of the apparatus; (b) Schematic diagram of advanced cyclic triaxial device

Undisturbed samples were used this experimental investigation. The natural soil was obtained at the bottom of a deep excavation with a depth of 9~11 m in Wenzhou city, Zhejiang province of China, where problematic soils with high water content, high compressibility, low permeability and low bearing capacity are often encountered. The primary index properties are shown in Table 1.

Table 1. The primary index properties of tested soft clay

Index properties	Values
Specific gravity, G_s (g/cm^3)	2.69~2.72
Natural water content, w_n (%)	59~63
Initial density, ρ_0 (g/cm^3)	1.62~1.67
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
permeability coefficient, k_v (cm/s)	2.5×10^{-6}

Cylindrical specimens of 5 cm diameter and 10 cm height were firstly hand-trimmed from the core of every sample by a wire saw, and then mounted in the triaxial cell. Following this, a back pressure of 300 kPa with an effective stress of 10 kPa (point O' in Figure 2) was applied until B values of the B-check process greater than 0.98. Then, a vertical stress was applied at a rate of 2 kPa/h to reach the K_0 line, point A in Figure 2 (σ_1= 18.2 kPa, σ_3= 10 kPa). K_0-consolidation along stress path AB was applied for a K_0= 0.55 according to the former studies of Wenzhou clay[11], using the stress control method at the rate of vertical stress described above to avoid the generation of pore pressure. Specimens were subjected to a constant K_0 value of 0.55 in the normally consolidated area with effective lateral consolidated stress σ'_{30} of 41 kPa and axial stress of 75.4 kPa according to the in-situ depth prior to cyclic loading. The final stresses were maintained for 72 hours under drained conditions to allow residual vertical creep rates to diminish to 0.002%/h or less [24]. After K_0-consolidation, close the drain valve and then stress controlled sinusoidal wave loading with three cyclic stress ratios, CSR=q^{ampl}/p'_0=0.189, 0.284, and 0.379, in which q^{ampl} is the amplitude of cyclic deviator stress calculated as $q^{ampl}=\sigma_1^{ampl}-\sigma_3^{ampl}$, p'_0 is the mean effective principal stress after K_0-consolidation (52.8 kPa). While the deviatoric part of the stress amplitude q^{ampl} was held constant the isotropic amplitude p^{ampl} calculated as $p^{ampl}=(\sigma_1^{ampl}+2\sigma_3^{ampl})/3$ was varied, i.e. three different stress path inclinations $\eta^{ampl}=q^{ampl}/p^{\mathrm{ampl}}$ (3, 1 and 0.64, see Figure 1) in the *p*-*q*-plane at the same phase were used. The inclination η^{ampl}=3 corresponds to σ_3=const (CCP test), η^{ampl}=0.64 corresponds to K_0-consolidation line. All tests were conducted at 0.001 Hz. The low frequency of 0.001 Hz was chosen due to reliable operation of the pressure valve regulating σ_3 in this range and benefit for the discharge of the pore water. Reliable measurements of excess pore water pressure in the specimen during cyclic loading are necessary for a confident assessment of effective stresses that control its deformation response [25]. All the tests were performed at room temperature (approximately 20℃) according to the annual average temperature of Wenzhou. The detailed test program is illustrated in Table 2. Image of the typical test preparation are presented in in Figure 3.

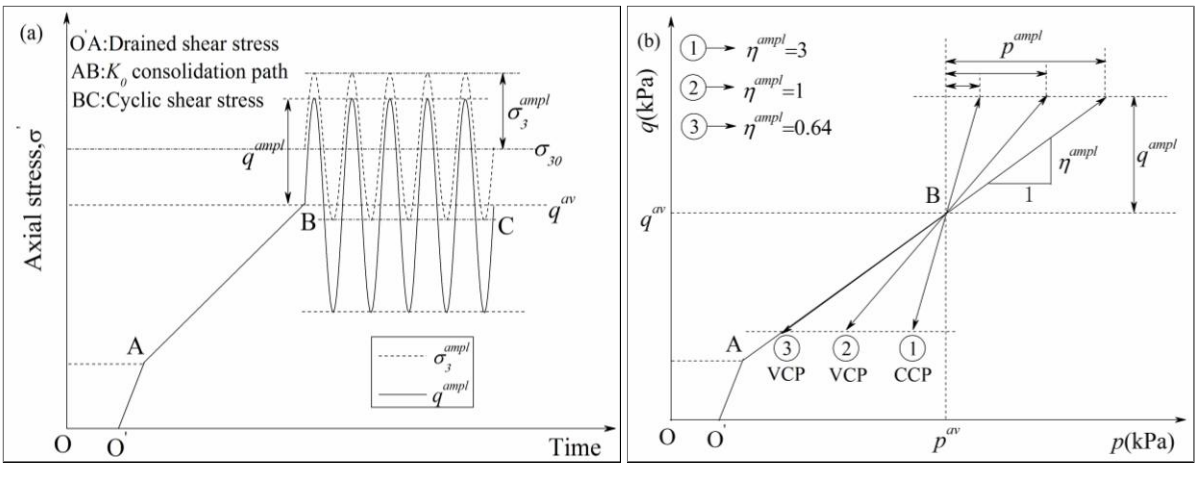


Figure 2. Schematic diagram for testing: (a) sequence of loading and (b) Stress paths

Table 2. Conditions of cyclic loading tests

Test number	p'_0 (kPa)	σ_3^{ampl} (kPa)	q^{ampl} (kPa)	CSR	η^{amp}	f (Hz)	Time (s)	$\varepsilon^p_{a,20}$ (%)
CCP1	52.8	0	10	0.189	3	0.001	20000	1.81
VCP1-1	52.8	6.67	10	0.189	1	0.001	20000	1.72
VCP1-2	52.8	12.23	10	0.189	0.64	0.001	20000	0.95
CCP2	52.8	0	15	0.284	3	0.001	20000	3.73
VCP2-1	52.8	10	15	0.284	1	0.001	20000	2.75
VCP2-2	52.8	18.34	15	0.284	0.64	0.001	20000	5.14
CCP3	52.8	0	20	0.379	3	0.001	20000	6.12
VCP3-1	52.8	13.33	20	0.379	1	0.001	20000	8.11
VCP3-2	52.8	24.45	20	0.379	0.64	0.001	20000	33.65

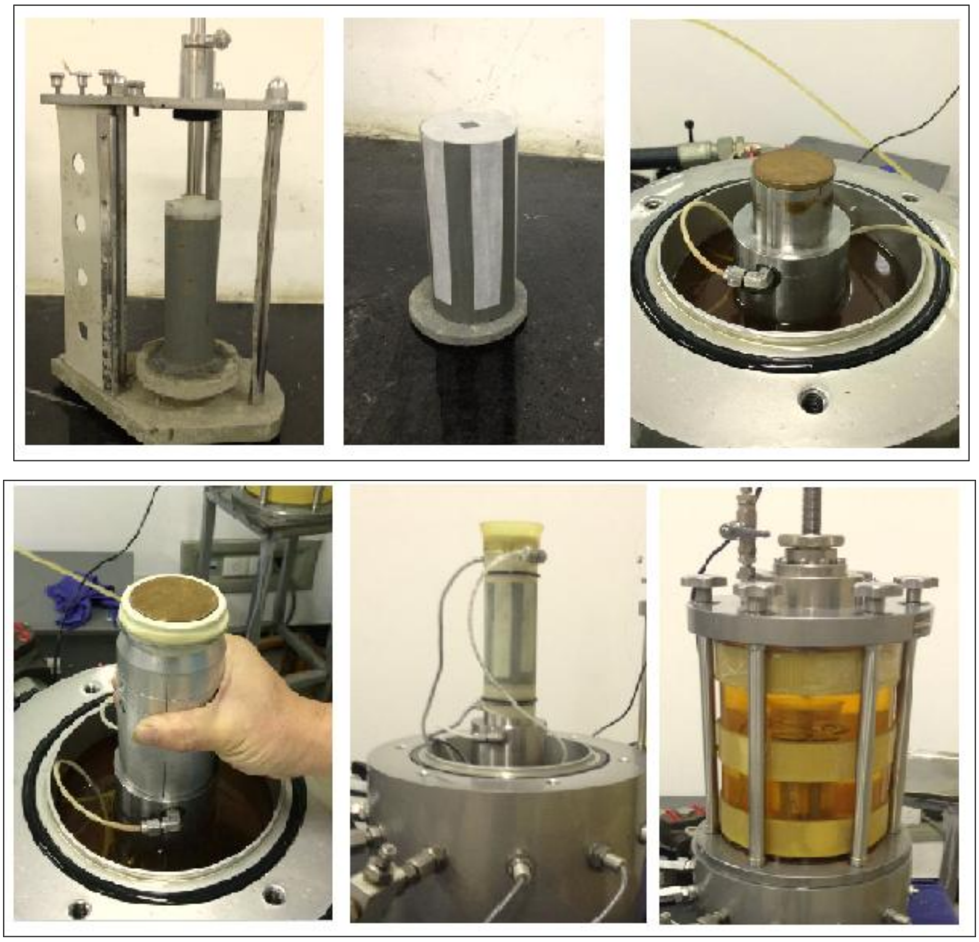


Figure 3. The process of typical test preparation

3. Test Results and Discussion

3.1. Effect of Cyclic Stress Path Inclinations η^{ampl} on Pore Water Pressure

The variation of normalized excess pore-water pressure r_u (u/p'_0) with time under different stress path inclinations η^{ampl}=3, 1 and 0.64, when the cyclic stress ratio is 0.189, 0.284 and 0.379 is illustrated in Figure 4 respectably. The curves in Figure 4 indicated that the development of cyclic pore water pressure is substantially affected by the stress path inclinations η^{ampl}. Comparison among different stress path inclinations shows that: the vibration amplitudes of normalized excess pore-water pressure increase with the decrease of η^{ampl}. However, compared with the instantaneous pore water pressure, the accumulative pore water pressure is more meaningful.

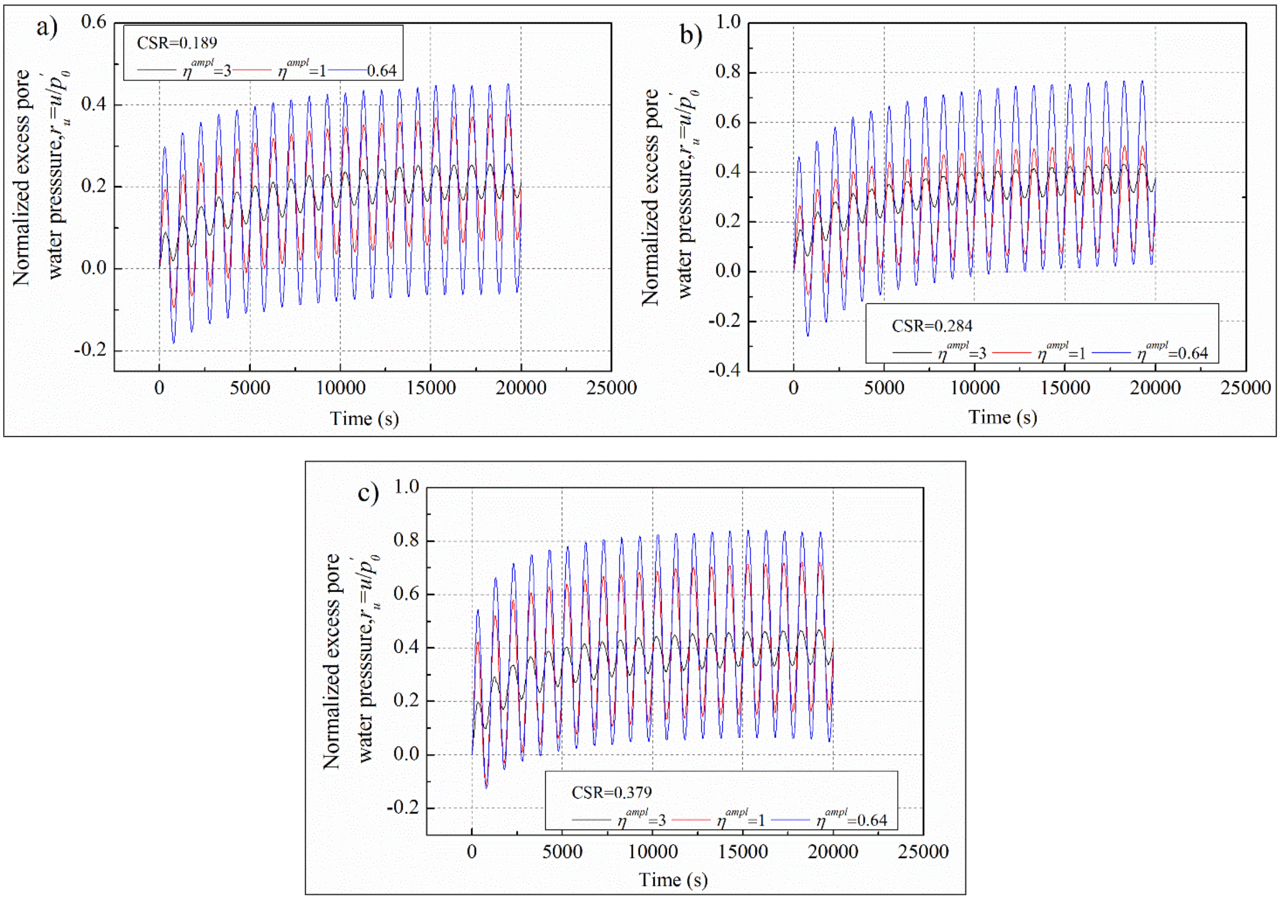


Figure 4. Variation of normalized excess pore water pressure, r_u (u/p'_0) with time in a cyclic stress-controlled test on K_0-consolidated soft clay under different stress path inclinations η^{ampl}=3, 1 and 0.64: (a) CSR=0.189; (b) CSR=0.284; (c) CSR=0.379.

The accumulative normalized excess pore water pressure r^q_u recorded at the end of each stress cycle are presented in Figure 5. It can be seen that the accumulative normalized excess pore water pressure rqu increase with the number of load cycles N and gradually stabilized in all tests. The results show that rqu is affected by the stress path inclinations η^{ampl}, r^q_u is not consistently higher or lower if the stress path inclination η^{ampl} is lower. For example, when CSR=0.189 r^q_u at N=20 in CCP1, VCP1-1 and VCP1-2 tests is 0.217%, 0.195%, and 0.174%, respectively. Recalling that the corresponding inclinations of stress paths, η^{ampl}, in CCP1, VCP1-1 and VCP1-2 tests are 3, 1, and 0.64, respectively, it is indicated that r^q_u decreases with the decrease in η^{ampl}. If η^{ampl} is decreased from 3 to 0.64, r^q_u may decrease by 20%. However, with the increase of CSR values, it is shown in Figure 5b and 5c that r^q_u at N=20 in VCP2-2 and VCP3-1 tests is almost equal to their corresponding CCP2 and CCP3 tests, i.e., 0.371%, 0.385% and 0.427%, 0.430%, respectively; While r^q_u at N=20 in VCP2-1 and VCP3-2 tests are much smaller, i.e., 0.261% and 0.372%, respectively. For example, if at CSR=0.284, η^{ampl} is decreased from 3 to 1, r^q_u may decrease by 32%. Similarly, if at CSR=0.379, η^{ampl} is decreased from 3 to 0.64, r^q_u may decrease by 13%. As a whole, it is concluded that, for K_0-consolidated samples, with an identical initial stress and average stress [i.e., identical values of p^{av} and q^{av}, as shown in Figure 2(b)], the extent of VCP effects and their trends is depended on the CSR values. This indicates that r^q_u is substantially affected by both of the deviatoric part of the stress amplitude q^{ampl} and isotropic amplitude p^{ampl}, only for some special stress path inclinations η^{ampl} both VCP and corresponding CCP test deliver similar r^q_u. For some other stress path inclinations η^{ampl} the VCP test may underestimate the r^q_u in comparison to the corresponding CCP test. However, Cai et al. (2013) found that the r^q_u of remold saturated soft clay were not influenced by VCP with identical initial stress but varied average stress in both undrained and partially drained conditions [20]. It can be concluded that the accumulation of pore water pressure r^q_u depends on the inclinations of stress path as well as soil structure properties and initial stress states.

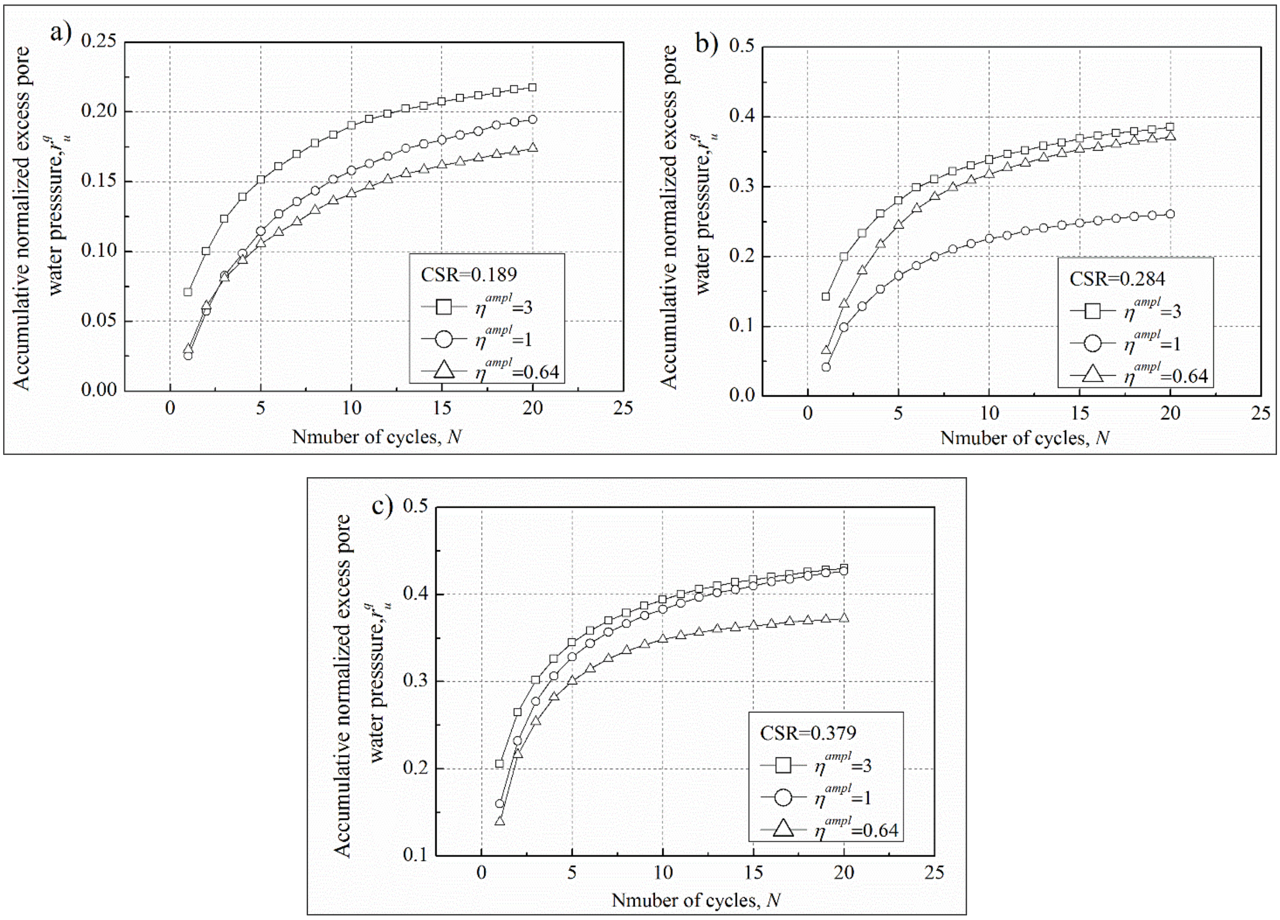
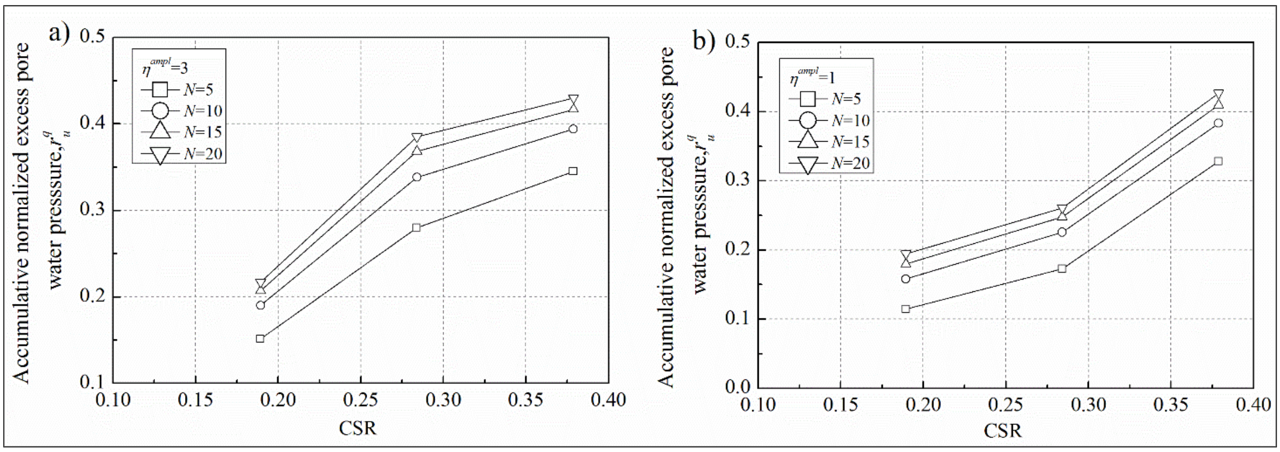


Figure 5. Variation of the accumulative normalized excess pore water pressure r^q_u with number of cycles N under different stress path inclinations η^{ampl}=3, 1 and 0.64: (a) CSR=0.189; (b) CSR=0.284; (c) CSR=0.379.

The effect of the CSR on the pore water pressure, r^q_u measured at selected cycle numbers (i.e., N=5, 10, 15 and 20) for different stress path inclinations η^{ampl} is presented in Figure 6. It is shown that, for a given stress path inclination η^{ampl}, the development of r^q_u are highly dependent on the magnitude of the applied CSR. For a given number of cycles, larger pore water pressures r^q_u are generated at a higher CSR value as a general trend. However, it is interesting that the stress path inclination η^{ampl} seem to have a distinct influence on the growth model of r^q_u with the CSR values. This behavior again indicated that the accumulation of pore water pressure r^q_u depends on both of the cyclic deviator stress and variable confining pressure.



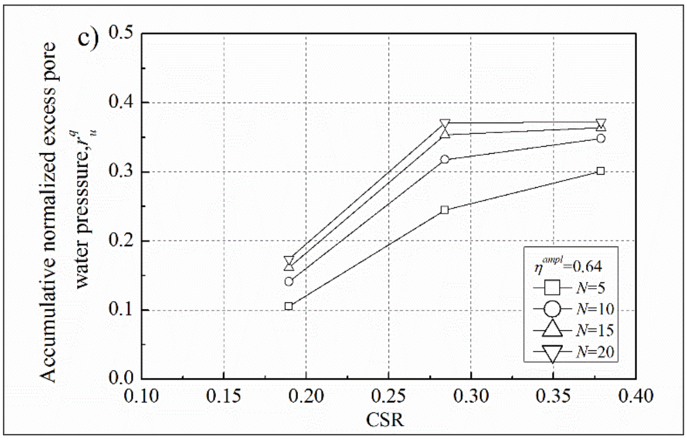
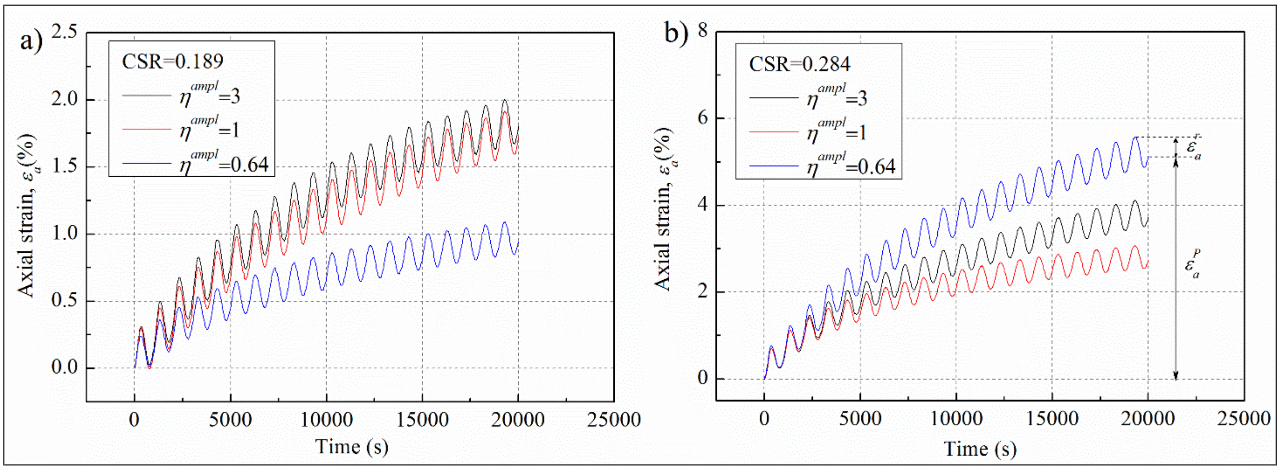


Figure 6. Relationships between CSR and r^q_u:(a) η^{ampl}=3; (b) η^{ampl}=1; (c) η^{ampl}=0.64

3.2. Effect of Cyclic Stress Path Inclinations η^{ampl} on Cyclic Deformation

One of the main purposes of this study was to compare the permanent deformation and resilient performance in CCP and VCP tests.The effect of stress path inclination η^{ampl} on the axial deformation behavior of K_0-consolidated soft clay is illustrated in Figure 7. As shown in Figure 7b the total axial strain consists of two components: the resilient axial strain $\varepsilon^r{}_a$ and the permanent axial strain $\varepsilon^p{}_a$. For a given CSR value, the axial strain development trends are similar at different η^{ampl} values and exhibits a significantly cyclic response during cycling, but the strain values for the number of cycles are different. While in Figure 7 the difference in the variation of resilient axial strain $\varepsilon^r{}_a$ with N is relatively small and it cannot be said how η^{ampl} affect it, the difference in the permanent axial strain $\varepsilon^p{}_a$ with N is significant.The curves reveal that stress path inclination η^{ampl} have a significant influence on the axial strain of K_0-consolidated soft clay and the influence rules are not consistent and even opposite under different CSR values. For example, as shown in Figure 7c for K_0-consolidated specimens with identical initial stress and average stress ratios η^{av} at larger cyclic stress ratio (CSR=0.379), specimen at η^{ampl}=0.64 is failed after 7 cycles according to the failure criteria of $\varepsilon^p{}_a$=10% suggested by Sakai et al (2003) [26]. While the specimens at η^{ampl}=3 and 1 are not failed until the end and VCP test with η^{ampl}=1 delivered larger values of $\varepsilon^p{}_a$ than the corresponding CCP tests (η^{ampl}=3). The behaviour is opposite for the smaller tested cyclic stress ratios, when CSR=0.189 the permanent axial $\varepsilon^p{}_a$ in VCP tests is smaller than that in the corresponding CCP tests. Furthermore, $\varepsilon^p{}_a$ decreases with the decrease of η^{ampl} values. The curves of $\varepsilon^p{}_a$ at intermediate value CSR=0.189 under different η^{ampl} values in Figure 5b again indicated the opposite behaviour. Similar phenomenon had also been reported by Rondon et al (2009), but their experimental study is related to unbound granular materials in free to drained conditions [19]. Sun et al (2017) conducted three types (identical maximum stress, identical average stress, and identical initial stress) of CCP and VCP tests on Toyoura sand in free to drained conditions [22]. Their study indicated that VCP tests generate more permanent strains (axial strain and volumetric strain) than CCP tests in most cases, especially with identical average stress, they attributed the result to the difference of confining pressure, σ_3. Actually, the difference of initial consolidation pressure is the ultimate cause.Thus, a more meaningful comparison of CCP and VCP tests should not only keep the identical average stress but also the identical initial stress. This study through K_0-consolidation and adopt sine wave could realize the above request. Based on the experimental results in this study it can be concluded that in undarined conditions when CSR⩽0.189, the effect of VCP on the permanent axial deformation of Wenzhou soft clay can be ignored, once the CSR above this level, the effect of VCP should not be ignored, otherwise, the settlements of the pavement may be substantially underestimated in some stress path inclinations η^{ampl}.



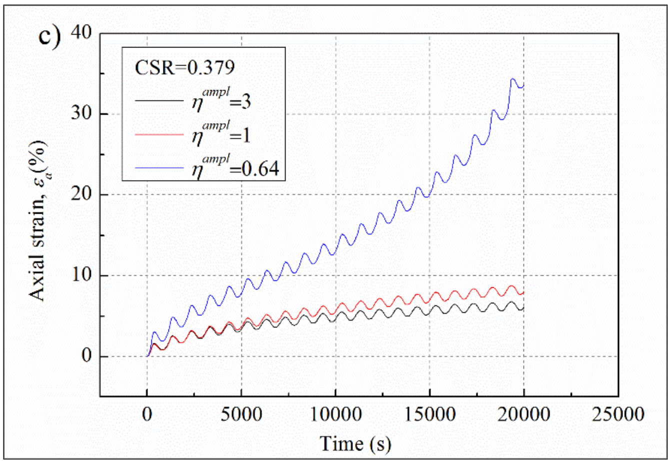


Figure 7. Variation of the axial strain with number of cycles *N* under different stress path inclinations η^{ampl}=3, 1 and 0.64: (a) CSR=0.189; (b) CSR=0.284; (c) CSR=0.379

Figure 8 shows that the deviatoric stress changes with the axial strain of the K_0-consolidated soft clay with a same CSR=0.284 under different stress path inclinations η^{ampl}=3, 1 and 0.64. As a whole, it is noted that irreversible axial plastic strain develops with the loading cycles, but the accumulation rate decreases with the loading cycles. It is also observed that the shape of the stress-strain hysteresis loops under the VCP and CCP stress paths are similar except for the areas encompassed by the hysteresis loop, which corresponds to the dissipated energy of the specimen under cyclic loading. The energy dissipated into the soil brings changes to the soil's properties through microstructural adjustment and leads to deformation to the soil depending on cyclic stress levels and the soil's initial conditions.

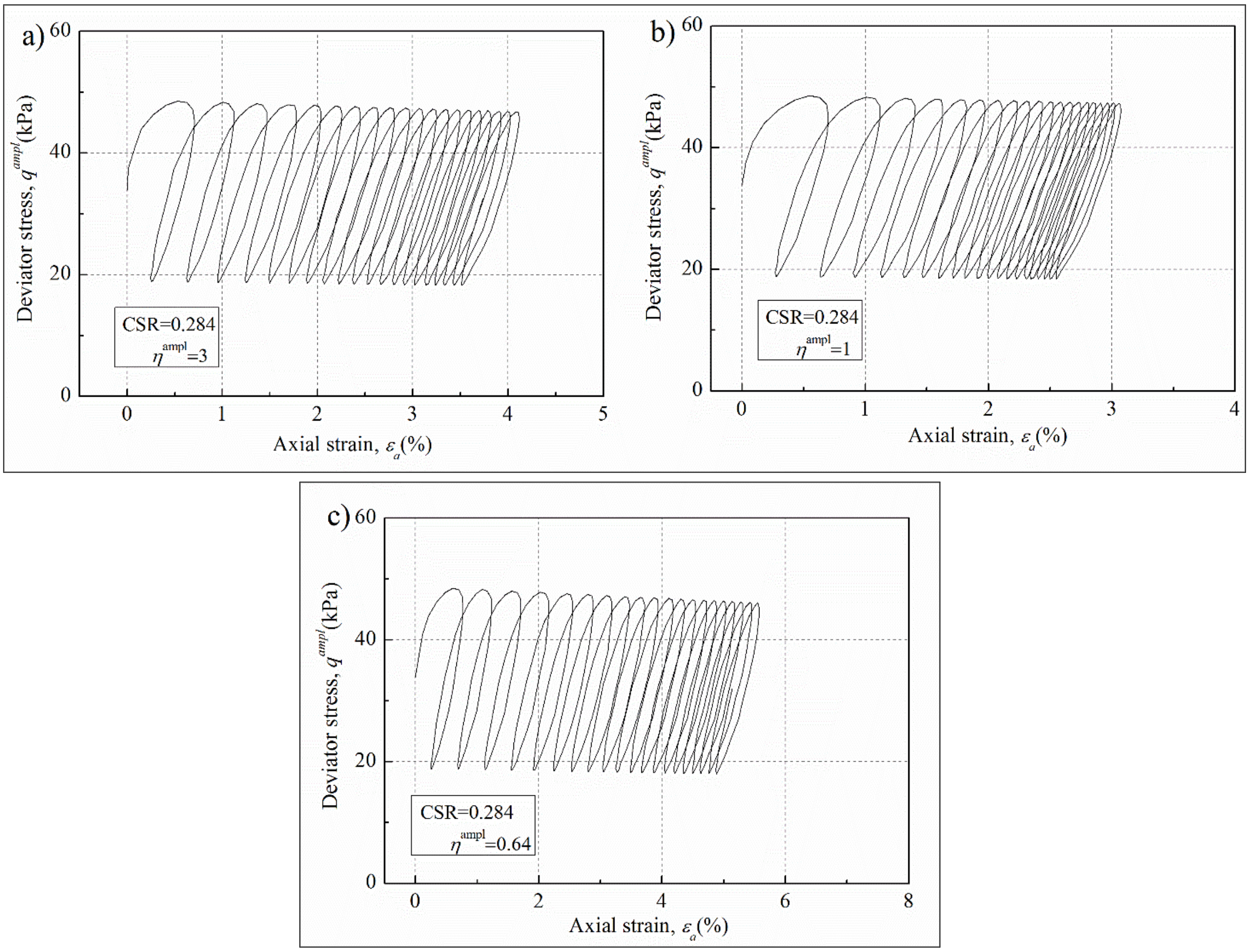


Figure 8. Stress-strain loops of a cyclic stress-controlled test on K_0-consolidated soft clay with a same CSR=0.284 under different stress path inclinations (a) η^{ampl}=3; (b) η^{ampl}=1; (c) η^{ampl}=0.64

Resilient modulus (Mr) is a key parameter for the analysis and design of a flexible pavement system under cyclic loading, apart from characterizing the stiffness and corresponding axial deformation of subgrade soils. In this study, resilient modulus Mr was defined as:

$$M_r = \frac{q^{ampl}}{\varepsilon_a^r} \tag{1}$$

Where q^{ampl} =amplitude of cyclic deviator stress and ε^r_a= recoverable portion of cyclic axial strain.

Figure 9 illustrates the changes in resilient modulus Mr with the number of loading cycles, N for CCP and VCP tests with three values of CSR. It can be noted that Mr typically decreased in the stage with the eraly loading cycles and then remained reasonably steady.The curves also indicated that the influenced trend of VCP on the resilient modulus Mr depended on the values of CSR. For small cyclic stress ratio CSR=0.189 (Figure 9a), If VCP cycles were applied along an intermediate inclination η^{ampl}=1 the resilient modulus Mr in VCP and CCP tests were similar. The behaviour is identical for the larger tested cyclic stress ratio CSR=0.379 (Figure 9c). While for η^{ampl}=0.64 and smaller amplitudes (CSR=0.189) the resilient modulus Mr in the VCP test exceeded the one of the corresponding CCP test. After N= 20 cycles the resilient modulus Mr in the test VCP1-1 was 1.4 times larger than that in the corresponding CCP1 test. The behaviour is opposite for the larger tested cyclic stress ratio CSR=0.379. When CSR=0.284, Mr in CCP2 test is larger than in the VCP2-2 test with η^{ampl}=0.64 but smaller than in the in the VCP2-1 test with η^{ampl}=1. Based on the results of resilient modulus Mr in this study, it shows that for larger values of the VCP stress path inclination $\eta^{ampl}=q^{ampl}/p^{ampl}=1$ the resilient modulus Mr in the VCP tests was similar or larger than in the corresponding CCP test, independently of the applied amplitude of cyclic deviator stress. In this case the relative influence of VCP on the resilient modulus Mr can be neglected. While the same applies to small VCP stress path inclinations η^{ampl}=0.64 in combination with larger cyclic deviator stress amplitudes the resilient modulus Mr in the in the CCP test exceeded the one of the corresponding VCP test. CCP tests seem to overestimate the resilient modulus Mr in comparison to VCP tests and thus underestimate the axial deformation. The results indicated that the magnitudes of the resilient moduli computed from CCP or VCP tests are not in agreement. In turn, this demonstrates that the resilient behavior depends not only on the maximum values of deviatorand mean normal stress, but also on the stress path. It has previously been suggested that compatible values of resilient modulus from CCP and VCP tests could be obtained, provided that an equal mean value of the mean deviator stress (qm) and mean of mean normal stress(pm) is equal in both tests [27]. However, our test procedure was not specifically designed to meet this condition. Furthermore, the samples generally display a significant dependence on deviator stress, this effect may in turn influence the computed resilient modulus.

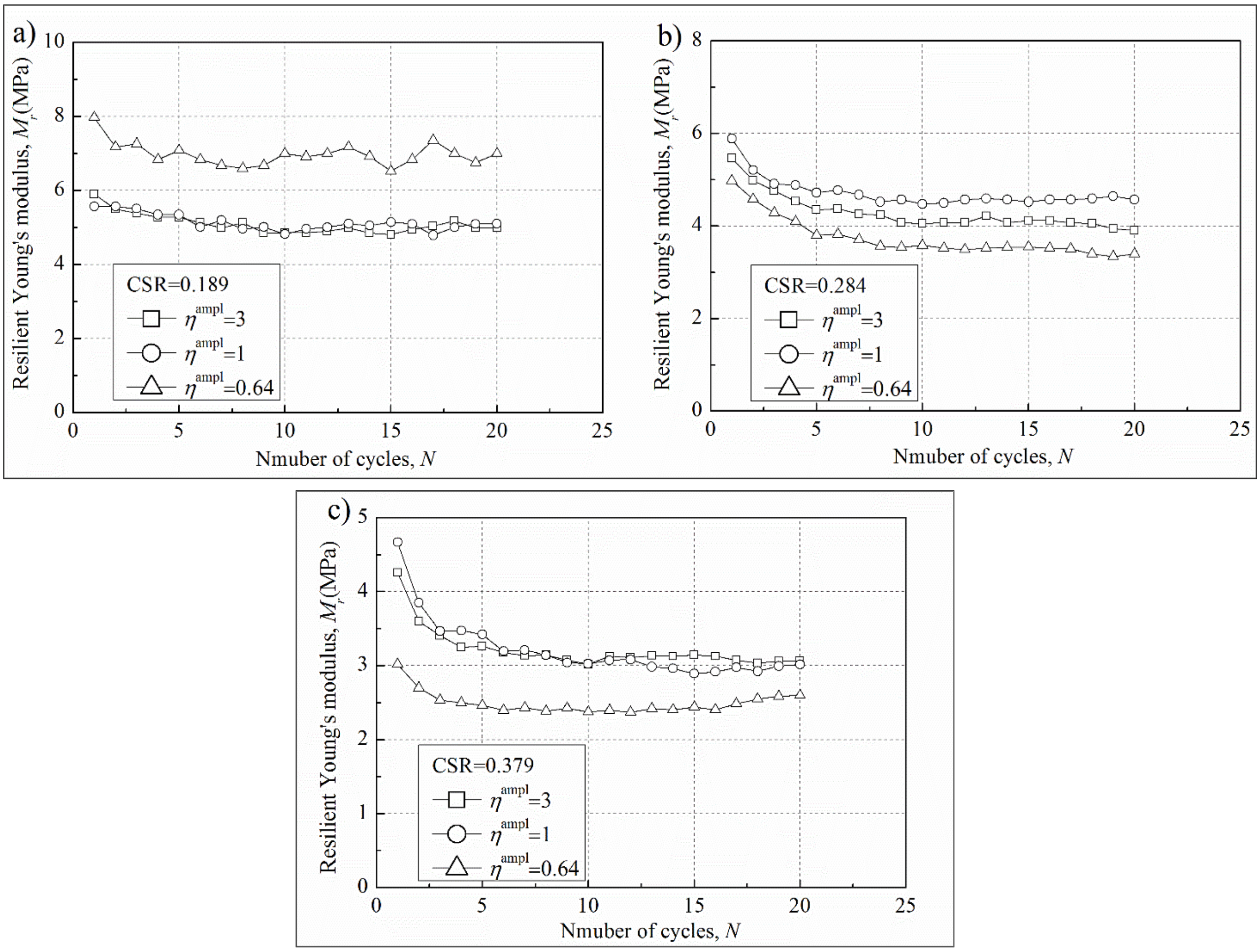


Figure 9. Resilient modulus (Mr) variations as a function of number of cycles for K_0-consolidated soft clay under different stress path inclinations η^{ampl}=3, 1 and 0.64: (a) CSR=0.189; (b) CSR=0.284; (c) CSR=0.379

4. Discussion and Conclusion

As most of the VCP studies were conducted on granular materials or sandy soils in free to drained conditions, the pore water pressure during cyclic loading is not considered. However, due to the low permeability of the soft clays, most of the cyclic tests were carried out in undrained or partially drained conditions. Reliable excess pore-water pressure in the specimen during cyclic loading is the key to assessment of effective stresses that control its deformation response. Recalling the accumulative normalized excess pore water pressure r^q_u in Figure 5, according to the effective stress theory, smaller pore water pressure r^q_u means larger effective stress and potentially stiffer soil, thus, the permanent axial strain should be smaller. However, not all of the permanent axial strain of the tests follow the trend as shown in Figure 7. In some stress paths the trend seems counterintuitive. For example, r^q_u in VCP2-2 test is slighter smaller than in the corresponding CCP2 test, while VCP2-2 test deliver larger permanent axial strain. This counterintuitive trend is more obvious in CCP and VCP tests at larger CSR value as shown in Figure 5c and 7c. All the pore water pressure and axial deformation trend of change demonstrates that the effect of cyclic deviator stress and variable confining pressure are coupled.

The excess pore-water pressure r^q_u may not be a dominant contributor to the cyclic degradation of normally K_0-consolidated soft clays. This trend may be caused by other factors, which may be the constraints of the role of cyclic confining pressure. This counterintuitive phenomenon is similar to overconsolidated clays. Many experts found that in overconsolidated clays of higher plasticity, with OCR higher than 2, a significant cyclic degradation takes place despite a decrease of pore water pressure with N and associated increases of effective stress [28-30]. The increase of the effective stress would suggest a strengthening of soil instead of its degradation, but that is apparently not the case. Mortezaie et.al. (2013) through cyclic strain-controlled Norwegian Geotechnical Institute (NGI) simple shear test on normally consolidated kaolinite clay having PI=28 also found a counterintuitive phenomenon, that higher frequency causes simultaneously larger degradation but smaller pore water pressures [12].

The results presented in this paper are applicable only to the K_0-consolidated saturated soft clay and similar clays tested in the uniform cyclic stress-controlled mode with an identical average stress. Test results clearly show that the cyclic deformation and cyclic pore water pressure can be affected substantially by the VCP, The trend and extent of the effect of VCP on the cyclic pore water pressure and cyclic deformation is significantly depend on CSR. It is suggested that at large CSR values the effect of VCP on the cyclic degradation of soils cannot be ignored; otherwise, the deformation of the soil may be significantly underestimated. In the end, it must be noted that due to the complexity of the coupling effect, further studies incorporate different initial stress, average stress ratio $\eta^{av}=q^{av}/p^{av}$, loading frequency f , OCR and PI are needed to better circumstantiate these conditions and to provide safe and rational rules for the analysis and design of a flexible pavement system.

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