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Soil Fabric and Transitional Behavior in Completely Decomposed Granite: An Example of Well-Graded Soil

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Abstract: Unlike sedimentary soils, limited studies have dealt with completely decomposed granite (CDG) soils, even though they are plentiful and used extensively in several engineering applications. In this paper, a set of triaxial compression tests have been conducted on well-graded intact and disturbed CDG soils to study the impact of the fabric on soil behavior. The soil behavior was robustly affected by the soil fabric and its mineral composition. The intact soil showed multiple parallel compression lines, while a unique isotropic compression line was present in the case of disturbed soil. Both the intact and disturbed soils showed unique critical state lines (CSL) in both the e-log p′ and q-p′ spaces. The intact soil showed behavior unlike other transitional soils that have both distinct isotropic compression lines ICLs and CSLs. The gradient of the unique ICL of the disturbed soil was much more than that of the parallel compression lines of the intact soil. In the intact soil, the slope of the unique CSL (M) in the q-p′ space was higher than that of the disturbed soil. The isotropic response was present for both the intact and disturbed soils after erasing the inherited anisotropy as the stress increased with irrecoverable volumetric change. Soil fabric is considered the dominant factor in the transitional behavior and such a mode of soil behavior is no longer restricted to gap-graded soil as previously thought.

Keywords: transitional mode; soil fabric; completely decomposed granite

1. Introduction

Saprolites triggered from different weathering processes on parent rocks are abundant in different regions around the world, in particular the tropical and subtropical regions. Completely decomposed granitic soil is abundant in southern China and extensively employed in many engineering applications as a construction or filling material. Although, the widespread and extensive use of such soils, limited studies have dealt with it, unlike sedimentary soils. Many engineering problems can be therefore occurred due to our limited knowledge about the fundamental characteristics of such soils; the weathered rocks also are in charge of several modes of landslides resulting in loss of lives and property [1–3].

Similar to clayey soils, in the e-log p′ space, at the high-stress levels, compression lines of clean sands converge in a unique compression line [4–6]. The volumetric strain of clean sands induced by the compression path is attributed to the particle crushing mechanism, unlike what happens in clays, which is associated with particle rearrangement [7]. In addition, clean sands reach the critical state irrespective of the initial states and drainage conditions, as a unique critical state line (CSL) is present in the e-log p′ space parallel to the unique compression line. Consequently, sandy soils can be depicted within the critical state framework in the same manner as clays [8].

As clean sands and clays are considered the extremes of the whole range of soils and they define unique compression lines, a transitional behavior has to be therefore present

in these extremes of soils [9]. The transitional mode can be defined as a behavior present between clean sands and clays [10]. This mode of soil behavior is essentially featured by the nonuniqueness of the critical state lines and/or individual compression lines [10–13]. Soil that shows this mode does not obey Rendulic's principle, also such that soil cannot be characterized with the framework of the critical state. The transitional mode was significantly present in the gap-graded residual soil triggered from the decomposition of Botucatu sandstone [9]. Therefore, it was thought that such behavior occurs only in the gap-graded soils. Recently, considerable studies indicated that the transitional mode is common more than the previous prevailing thought, which was restricted to gap-graded soil [9,11,14]. Besides the gap-graded soils, transitional behavior has been observed in well-graded soils, for example: in the coarse granular soil as reported by Xiao et al. [15] and in the silty clayey soil [10,12,16], in addition to sand [13,17].

In the e-log p' space, although the nonunique compression lines were present, Altuhafi et al. [13] observed that the soil reached the critical state with a unique CSL. The same behavior was also observed by Wang and Yan [18] in the intact completely decomposed granite (CDG). However, many studies that dealt with the disturbed CDG soils showed unique compression lines and CSLs [19–22]. The distinct critical state lines and compression lines were also present in other reported soils [10–12,14]. This can be explained as follows; in the case of a unique CSL, the soil fabric in charge of the nonuniqueness of the compression lines is not sturdy enough and totally damaged through the shearing path, resulting in a unique void ratio–stress relationship in the critical states. Meanwhile, in other situations, the soil fabric in charge of the nonuniqueness of the compression lines is strong enough to resist shearing, leading to nonunique critical state lines [11,14,23]. In other words, the transitional behavior extends to the critical state.

Based on the convergence degree of the CSLs in the e-log p' space and compression lines, the transitional mode can be quantified through two parameters P and m, respectively proposed by Ponzoni et al. [12]. Figure 1 indicates the m and P parameters calculation from the changes of the specific volumes. The m parameter is the ratio between the final and initial specific volume change (i.e., $m = \Delta v2/\Delta v1$, as indicated in Figure 1b). While the P parameter is determined from the intersection of the CSLs with the axis of the specific volume at log p' = 1 and the change of the initial specific volumes (i.e., $P = \Delta \Gamma /\Delta v1$, as illustrated in Figure 1d). The m and P parameters values range from 0 to 1, the soil behavior is totally transitional when the m and P values equal 1, in which distinct lines were significantly present. Soil exhibits a unique compression line and CSL when the m and P values are 0 and the in-between values denote to different degrees of the transitional mode.

Transitional behavior was found in different types of soils as mentioned previously, thus different factors govern this behavior, and the fine plastic content was one of these factors [10,14]. As well, the soil minerals and grading are considered crucial factors [24]. While the methodology of the specimen's preparation, stress level, and stress history have no effects on the transitional behavior [9,14]. The transitional behavior was apparent in undisturbed samples (i.e., intact samples) more than that in disturbed samples (i.e., reconstituted samples) [16].

The key purpose of this research is to investigate the transitional behavior in a well-graded CDG soil, particularly investigating the normal compression lines and CSLs patterns as well as evaluating the transitional behavior degree through the m and P parameters. In addition to answering the risen question of whether the transitional behavior is more pronounced in the intact CDG soil.
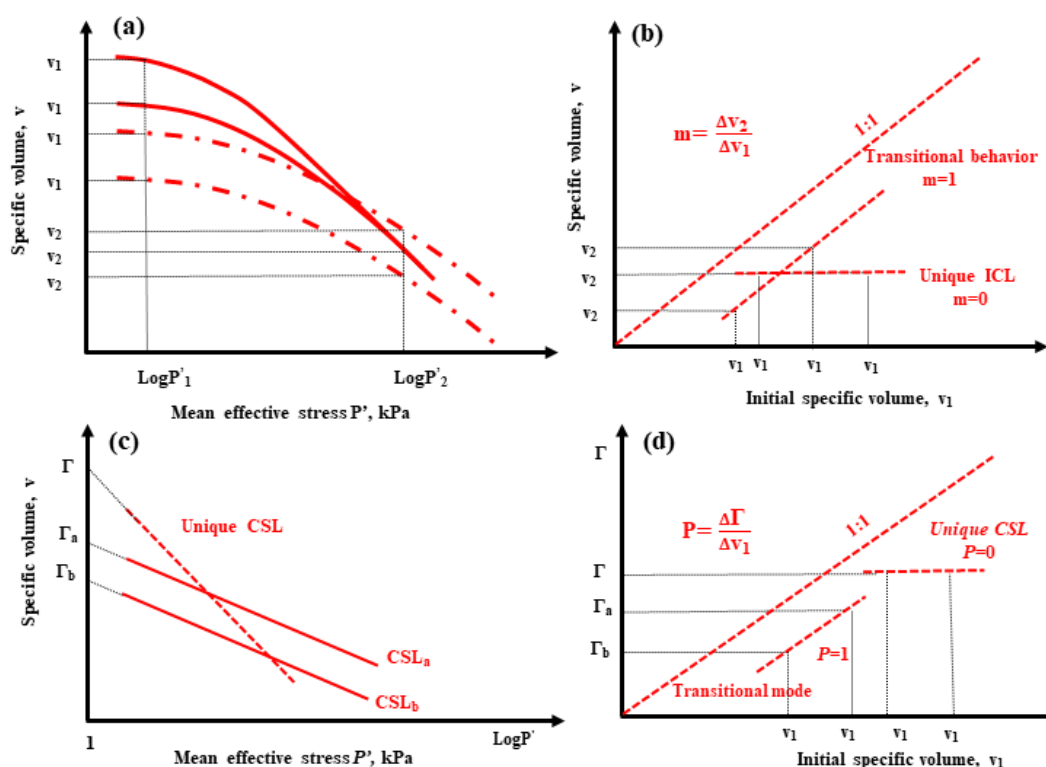
Figure 1. Calculation of *m* and *P* parameters; (**a**) compression lines, (**b**) quantifying the *m* parameter, (**c**) critical state lines, (**d**) quantifying the *P* parameter [12].

2. Materials and Methods

A set of isotropic compression and triaxial compression shearing tests have been performed on a well-graded CDG soil originated from monzonitic granite, Guangdong province, China. Figure 2 depicts the grain-size distribution of the studied CDG soil and other reported soils [18,21] for comparison. Based on the unified soil classification system (USCS), the studied soil classification is highly plastic clay. The maximum dry density was 1.51 kN/m^3 at optimum moisture content of 25.25%. Liquid and plastic limits were 55.84% and 27.85%, respectively. The X-ray diffraction (XRD) analysis revealed that the soil minerals were 58.25% quartz and 38.25% kaolinite with 3.5% illite and montmorillonite. The most abundant minerals were quartz and clay minerals particularly kaolinite which was strongly present. This can be attributed to the intense environment of weathering and the unimpeded drainage conditions such as well-drained hill slopes and joints as well as fractures of the parent rock, therefore kaolin clay mineral was clearly noticed. The other clay minerals such as montmorillonite and illite were also observed but in small fractions, this can be imputed to the removal of the limited amount of cations in a stagnant condition near to the bottom of the hill slope [25,26].

The triaxial tests were carried on intact specimens trimmed as shown in Figure 3 brought from Bozhi City, Guangdong province, People's Republic of China. After trimming the soil specimens, they were enclosed by a rubber membrane and then immersed in a water tank subjected to a negative pressure for 4 to 5 days to improve their saturation degree. The soil specimens were then placed in the automated triaxial device and saturated to over 0.95 B-value proposed by Skempton [27]. During the isotropic compression, the rate of loading and unloading of 7 and 15 kPa/h were respectively used. A strain-controlled mode with adequate axial strain rates to limit the induced excess pore water pressure, based on Head [28], was applied. The isotropic compression and drained and undrained triaxial shearing tests on both the intact and disturbed soils were performed as summarized in Tables 1–4, respectively. The void ratios of the soil samples were back-calculated from the volume change and the final void ratios as indicated by Verdugo and Ishihara [29].
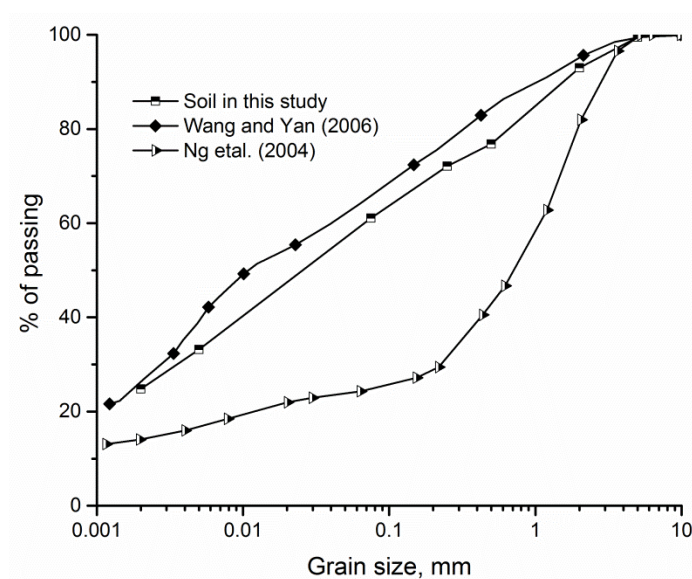
Figure 2. Grain-size distribution curves.

Figure 3. Trimming of the intact samples.

Table 1. Testing conditions of the isotropic compression path of the intact soil.

Test ID	Loading Path, P′ kPa	Initial Void Ratio	Final Void Ratio	Slope of Parallel ICLs (ξ)
ISO-1	20–450	0.8198	0.7583	
ISO-2	20–450–275–600	0.6988	0.6442	
ISO-3	20–600	0.7833	0.7194	0.027
ISO-4	20–250–50–600	0.7312	0.6733	
ISO-5	20–600–200	0.7490	0.7057	

ISO refers to the isotropic compression test.

Table 2. Testing conditions of the isotropic compression path of the disturbed soil.

Test ID	Compaction Ratio	Load Path, P′ kPa	Initial Void Ratio	Final Void Ratio	Slope of the Unique ICL (ξ)
	85%	20–600	0.982	0.686	
Disturbed soil	87.5%	20–250–150–450	0.935	0.709	0.1
	90%	20–200–50–400–50	0.916	0.741	

Table 3. Testing conditions of the drained and undrained shearing tests of the intact soil.

Test Type	Consolidation Stress p′, kPa	Void Ratio before Shearing	Void Ratio after Shearing	Slope of the CSL (λ) in the e-log p′ Space	Slope of the CSL (M) in the q-p′ Space
	100	0.8882	0.9016		
CD	200	0.7841	0.7810		
	300	0.7060	0.6684		
	400	0.6791	0.6493	0.18	1.17
	100	0.8096	0.8096		
CU	200	0.7626	0.7626		
	300	0.7370	0.7370		
	400	0.6797	0.6797		

CD and CU denote the drained and undrained shearing tests, respectively.

Table 4. Testing conditions of the drained and undrained shearing tests of the disturbed soil.

| Test ID	Consolidated Stress p′(kPa)	Void Ratio before Shearing	Void Ratio after Shearing	Slope of the CSL (λ) in the e-log p′ Space	Slope of the CSL (M) in the q-p′ Space
	100	0.8140	0.7578		
CD	200	0.7818	0.6981		
	300	0.7451	0.6635		
	400	0.7069	0.6272	0.083	1.03
	100	0.8164	0.8164		
CU	200	0.7808	0.7808		
	300	0.7447	0.7447		
	400	0.7066	0.7066		

3. Results and Discussion

3.1. Compression Response

Figure 4 shows the isotropic compression responses during the loading and unloading stages of the intact samples. It was found that the isotropic compression lines (ICLs) did not merge in unique line and they were quite parallel. However, a unique ICL was significantly present for the disturbed soil remolded by the moist tamping method, as shown in Figure 5. The ICLs of the disturbed soil merged in a unique line, at mean effective stress of 400 kPa, while in the intact samples, the ICLs still exhibit parallelism up to 600 kPa without any tendency of convergence. Unique ICLs were clearly found in the re-compacted CDG soils irrespective of their initial state [19,21,22,30,31]. The slope of the ICL (ξ) of the intact CDG soil was 0.027 lower than that of the disturbed soil which was 0.1. This can be imputed to the inherited fabric from the parent rock of the intact specimen with a contribution of remnant bonds, in addition to the loose structure induced from the moist tamping manner employed in the disturbed specimens′ preparation. To investigate the effect of the remnant bonds, the tangential stiffness (E_{tan}) and the axial strain during the drained shearing were plotted in Figure 6. The intact soil stiffness was higher than that of the disturbed soil. The intact soil also showed a sudden reduction; however, the disturbed soil stiffness curve was smooth without a sudden reduction, which meaning that the soil bonds were totally destroyed through the isotropic compression stage. This behavior matched with the results of Liu et al. [3] and Elkamhawy et al. [32].
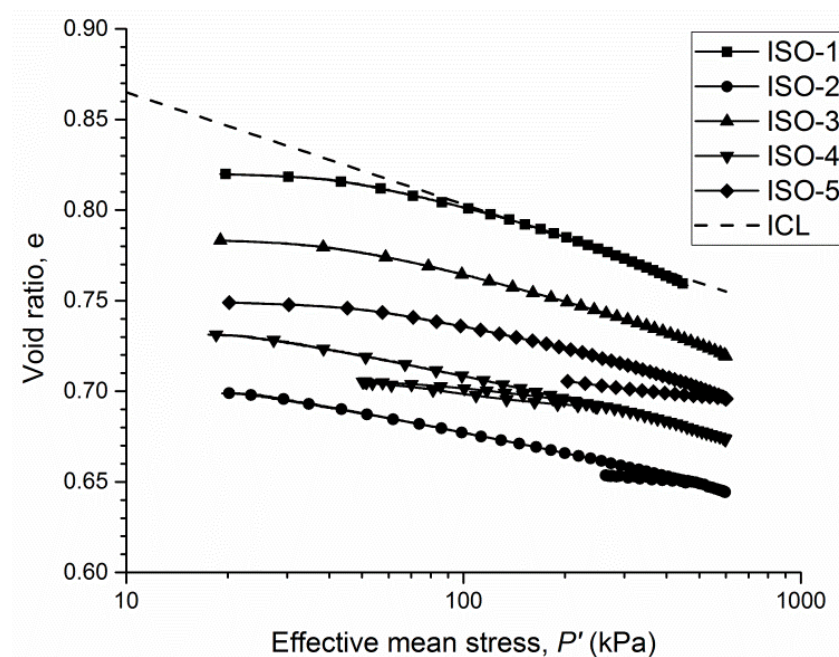
Figure 4. Isotropic compression response of the intact soil.
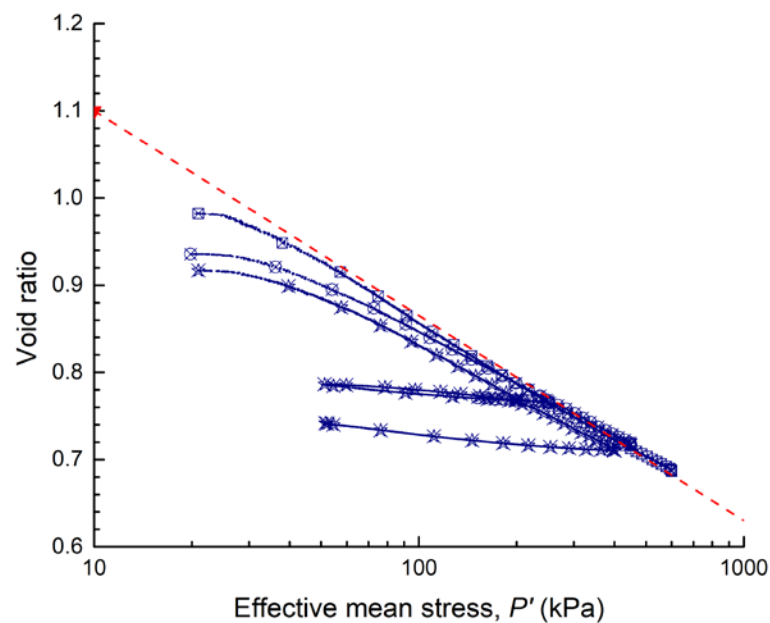
Figure 5. Isotropic compression response of the disturbed soil [30].

Xiao et al. [15] and Nocilla et al. [10] imputed the nonuniqueness of the ICLs to the debility of the isotropic compression stage to eradicate the variation in the initial fabric of the soil specimens to reach a unique relationship of effective stress-void ratio, thus nonunique ICLs were present. This finding matches closely with observations of Xu et al. [33], where the transitional behavior is clearly apparent in the undisturbed soil specimens than that in the disturbed soils, showing that fabric is the vigorous factor controlling the transitional behavior. Altuhafi et al. [13] concluded that the reported subglacial Langjokull sediments reached the ultimate grading during deposition and cannot evolve more under high stresses and strains. Since in granular materials, yield is associated with particle crushing, thus the subglacial soil showed distinct compression lines in both the intact and remolded states [13]. Although the undisturbed CDG soil

reported by Wang and Yan [18] revealed yield stress, the ICLs did not converge in a unique line and showed parallelism after the yield point.
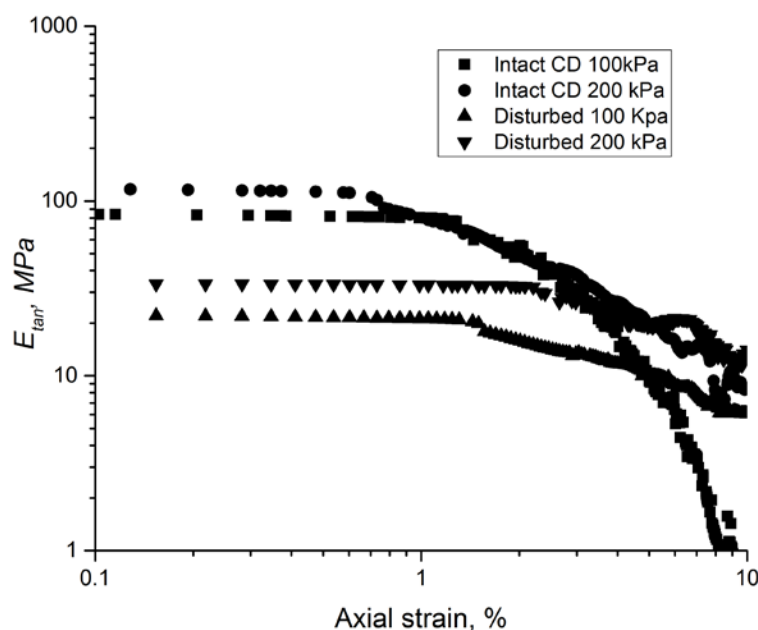

**Figure 6.** The drained tangential stiffness of both the disturbed and intact soils.

In this study, the intact soil showed distinct ICLs, while the disturbed soil showed unique ICL. It can be therefore said that the soil fabric is the governing factor of such behavior. Unfortunately, the available device is restricted to low stresses, thus it will be interesting to retest intact soil at high stresses to verify whether the intact soil would show a unique ICL in future studies. It would be also interesting to perform these tests in a triaxial device connected with a computed tomographic (CT) scanner to shed more light on the soil fabric deformation and particle crushing.

The value of the m parameter was 0 in the case of the disturbed sample, while it almost was 1 in the intact samples. Elkamhawy et al. [30] found transitional behavior in reconstituted CDG samples with increasing sand contents by 5% and 10%, as the values of m were 0.3 and 0.73, respectively. Nocilla et al. [10] also found the transitional mode in the reconstituted Italian silt with 3.5% clay content. These observations refute the previously prevailing thought that such mode of soil behavior is restricted to the gap-graded soil. Wang and Yan [18] also observed nonunique ICLs in the intact well-graded CDG soil. However, the CDG soil reported with Wang and Yan [18] exhibited high compressibility more than that in the studied soil, where ξ was 0.076, even though the closely grading as depicted in Figure 2. This can be explained from the XRD analysis, as the soil minerals reported by Wang and Yan [18] were quartz, feldspar, muscovite mica, and kaolinite/halloysite. While there is no fragile and crushable mineral such as feldspar that leads to an increase in the compressibility in the studied soil, the presence of mica also increases the compressibility of the soil even if present with slight portions [34].

3.2. Volumetric Change

The specimen deformations were carefully monitored during the isotropic compression stage, as an external linear variable displacement transducer (LVDT) was provided with the triaxial device for the axial displacement measurement. The specimen volumetric change is examined via the transducer of the back-pressure. Figure 7 depicts the radial and axial strains versus the mean effective stresses of both the intact and disturbed CDG soils. It was observed that the axial strain was smaller than the radial strain for both the intact and disturbed soils. Both the intact and disturbed soils revealed the same response, however, this mutual response is attributed to two different reasons. For the intact soil,

the in-situ soil stress state is anisotropic with K_0 values less than 1, thus the changes that occurred in stress in the radial direction are higher than that occurred in the axial direction [18]. Consequently, the change in radial strain is larger than that in axial strain. On the other hand, for the disturbed soil, this response can be imputed to the moist tamping preparation technique [22]. As, when a soil specimen is compacted in a specified direction, this direction consequently becomes stiffer than the other one. Thus, during the isotropic compression, the contraction response of the axial direction is less than that occurred in the radial direction [35]. As the stress increases, the inherited anisotropy in the soil's memory is gradually erased; hence the soil response becomes isotropic. Axial and radial strains, in the case of intact soil, were illustrated in Figure 7a,b, they were much less than those in disturbed soil shown in Figure 7c. This clearly reveals the impacts of the inherited fabric and the remnant bonds of the parent rock on the soil response. However, during the unloading path, the changes in strains were almost insignificant; indicating that the volumetric changes were irrecoverable.
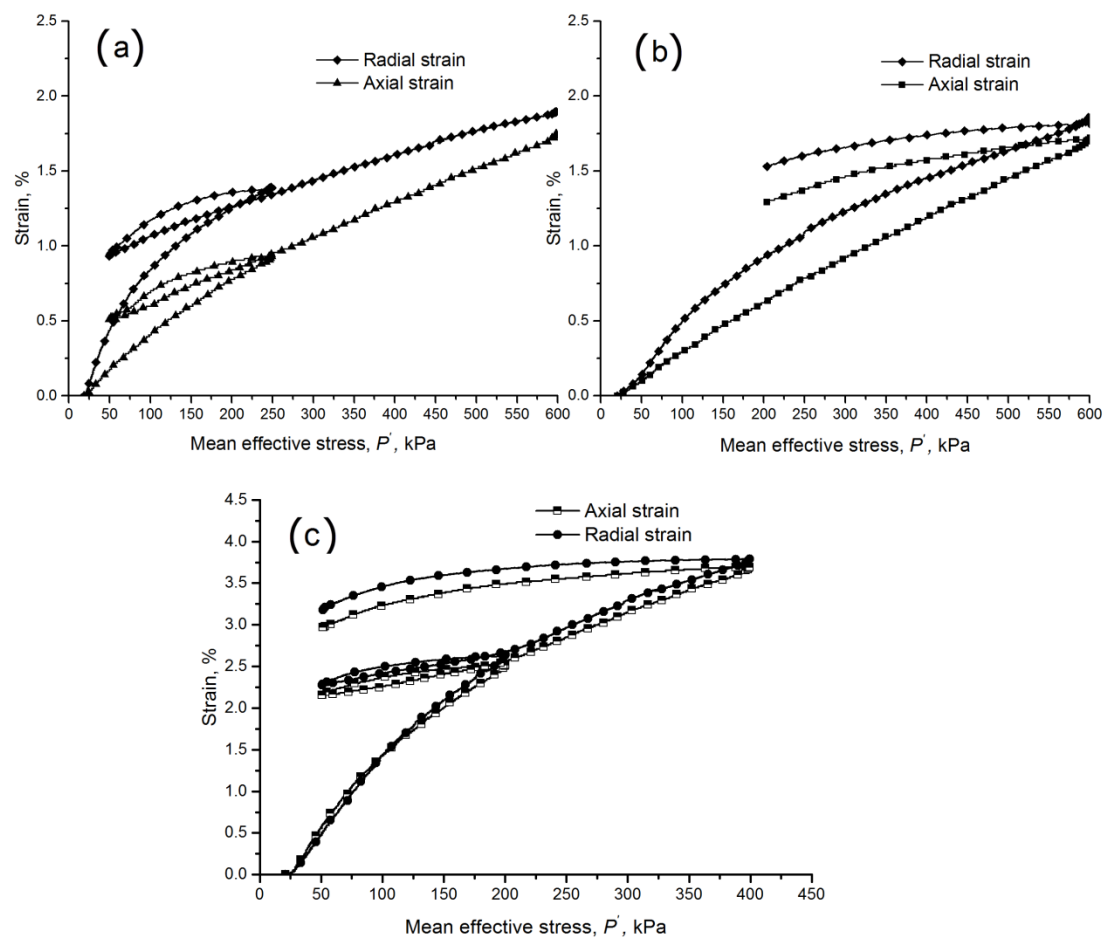

Figure 7. Radial and axial strains with mean effective stress of intact and disturbed soils during an isotropic compression; (**a**) ISO-4, (**b**) ISO-5, and (**c**) disturbed soil.

3.3. The Critical State

Although the nonuniqueness of the ICLs was present, a unique CSL was present in the e-log p' and q-p' domains, as shown in Figure 8a,b, respectively. A unique CSL was present irrespective of the initial state of the soil specimens and drainage conditions. This response matches closely with the intact CDG reported by Wang and Yan [18]. Altuhafi et al. [13] proposed that the soil fabrics that caused non-converging of the ICLs were not sufficiently forceful and totally damaged during the shear path. Therefore, the uniqueness of the critical states in a single line is related to the whole destruction of the soil fabrics

triggered with the compression stage through the shearing path, leading finally to form a unique soil fabric in the critical states. As demonstrated in Figure 8b, the critical states lie on a unique critical state line in the q-p' space with a gradient (M) of 1.17 corresponding to a friction angle of 29.3°. However, in the case of disturbed soil, values of the gradient M and friction angles were 1.03 and 26.1°, respectively, this behavior agrees with Burland [36], Futai et al. [37], and Liu et al. [3]. This also reveals the impact of the inherited soil fabric on the soil strength parameters.
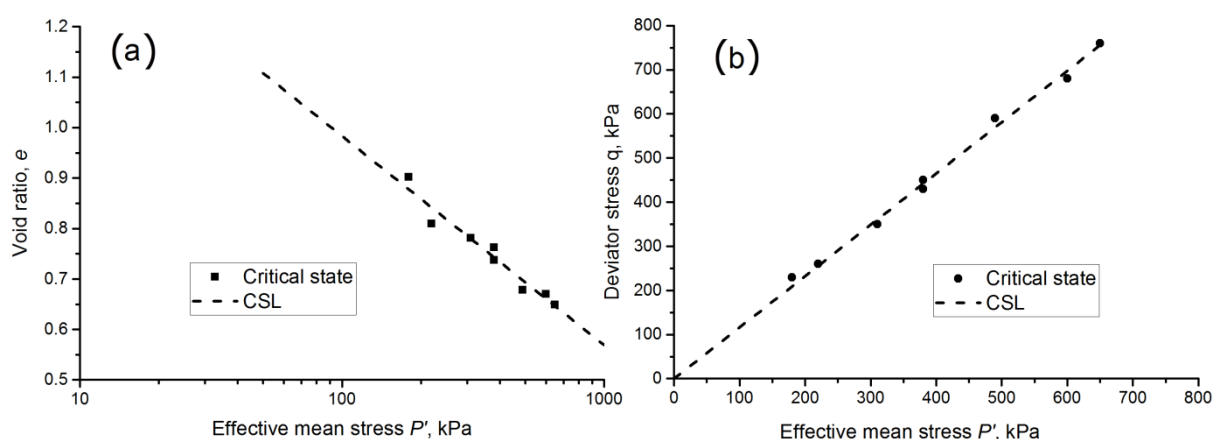
Figure 8. Critical states of the intact soil (**a**) the e-log p' space, (**b**) the deviator and mean effective stresses q-p' space.

4. Conclusions

To investigate the effect of disturbance on the behavior of the CDG soils, a set of triaxial compression in addition to drained and undrained triaxial shearing tests were conducted on an intact well-graded CDG soil and compared with the disturbed state. The intact samples showed parallel compression lines, while the disturbed soil revealed a unique ICL within the stress levels employed in this study. The soil behavior was unlike other "transitional" soils that have both distinct ICLs and CSLs. Soil fabric was found to have a paramount effective role in the compressibility of the CDG soil. The unique isotropic compression line gradient of the disturbed soil was 0.1 larger than 0.027 of the distinct lines of the intact soil.

In the case of intact CDG soil, fabric persists during the isotropic compression, nevertheless, it was not forceful enough to sustain through shearing, and the soil showed unique CSL irrespective of drainage conditions and initial states. The volume change was irrecoverable, regardless of soil state (i.e., intact or disturbed) and the disturbed soil strains were significantly larger than those in the intact soil. Both the intact and disturbed soils showed isotropic behavior after erasing the inherited anisotropy as the effective mean stress increased.

Results of this study proved that the transitional behavior is not constrained to gap-graded soils as thought previously and transitional behavior is more pronounced in intact soil. The intact soil revealed higher strength parameters than those of the disturbed soil, as the intact soil CSL gradient was 1.17 corresponding to a 29.3° friction angle and in the case of disturbed soil, the gradient M and friction angle were 1.03 and 26.1°, respectively. For soils that showed totally transitional behavior, a term such as "normal compression line" no longer signifies a state boundary surface. The concept of "state parameter" proposed by Been and Jefferies [8] also no longer has a clear meaning, thus new terminologies are urgently needed. Further research is necessary required to examine visually the impact of the soil fabrics on the non-convergence of the ICLs during the isotropic compression via a triaxial device connected with an industrial computed tomographic scanner (CT-scan) to monitor the fabric change step by step.

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