

Full Length Research Paper

Physical and mechanical properties of decomposed granite soils sampled in Cheongju, Korea

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Decomposed granite soils are in a wide range of conditions depending on the degrees of weathering. This paper is intended to examine laboratory tests, such as consolidation tests and conventional triaxial compression (CTC) tests conducted in order to find out the mechanical properties of Cheongju granite soil. Along with the foregoing, the results of basic physical tests conducted in order to grasp the physical properties of Cheongju granite soil were described and based on the results, methods to calculate the mechanical parameters of numerical approaches using Lade's single and double surface work-hardening constitutive model were examined. Finally, this study is intended to explain the stress properties of Cheongju granite soil used as a geotechnical material based on its shear behavior and critical state concept using the results of isotropic consolidation tests and triaxial compression tests. As a conclusion, it can be seen that in the relationship between confining stress and maximum deviator stress, the slope is maintained at a constant value of 2.95. In the drained CTC test, maximum deviator stress generally existed in a range of axial strain of 6 to 8% and larger dilatancy phenomena appeared when confining stress was smaller. Finally, based on the results of the CTC tests on Cheongju granite soil, although axial strain, deviator stress and pore water pressure showed mechanical properties similar to those of over consolidated soil, Cheongju granite soil showed behavior similar to that of normally consolidated soil in terms of volumetric strain.

Key words: Granite soil, stress-strain relation, single and double surface work-hardening model, soil parameter.

INTRODUCTION

Decomposed granite soil is a generic term of soils generated through the weathering of granite, schistose granite and granite gneiss on the spot, that is, residual soils remaining on the spot and transported soils originated in the residual soils. The soils are in a wide range of conditions depending on the degrees of weathering from those close to rock to those that contain fine grains, such as silt or clay. Weathering can be divided into three types; mechanical weathering that refers to processes for bed rock to be crushed into small pieces or to be worn due to climatic causes such as rain-fall or winds, chemical weathering that creates completely different minerals from those of bed rock by completely changing chemical properties and solution that refers to processes through which soluble minerals are dissolved

from rock while insoluble minerals remain as residues. Based on the degrees of weathering, rock and soils are divided into hard rock, soft rock, weathered rock, under weathering, slightly weathered, highly weathered and completely weathered soil.

Decomposed granite soil's characteristics to be distinguished from rock include the facts that it can be generally machine-excavated and that although, it is apparently rock, it is decomposed while being handled with a machine or other way to become sand or gravel. The mechanical properties of decomposed granite soil after being weathered are affected by the contents of primary minerals, such as quartz, mica and feldspar and bed rock structures as well as natural conditions, such as climates and drain conditions.

This paper is intended to examine laboratory tests, such as consolidation tests and conventional triaxial compression (CTC) tests conducted in order to find out the mechanical properties of decomposed granite soil

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sampled in Cheongju (Cheongju granite soil) such as shear and stress-strain behavior. Along with the foregoing, the results of basic physical tests conducted in order to grasp the physical properties of Cheongju granite soil were described and based on the results, methods to calculate the mechanical parameters of numerical approaches using Lade's single and double surface work-hardening constitutive model (Lade and Musante, 1977; Lade, 1978; Lade and Hong, 1989) were examined. Finally, this study is intended to explain the stress properties of Cheongju granite soil used as a geotechnical material based on its shear behavior and critical state concept using the results of isotropic consolidation tests and triaxial compression tests.

PHYSICAL PROPERTIES OF DECOMPOSED GRANITE SOIL IN CHEONGJU REGION

General properties of decomposed granite soil

General properties of the decomposed granite soil will roughly be described in terms of its physical properties, compaction and crushing, permeability and strength. The physical and mechanical properties of weathered residual soil are greatly affected by weathering environments and bed rock structures, and depending on the degrees of weathering, the sizes of soil particles are in diverse, ranging from particles close to rock to fine grains such as silt or clay.

The degrees of weathering are determined by the natures of bed rock, climates and drain condition and decrease along with the depth from the ground surface. General tendencies of decomposed granite soil are that natural water contents and void ratios decrease with depths while unit weights increase. As the depth increases, soil particles become coarser and angular particles increase. This can be considered as a result of the fact that weathering begins from the ground surface. Soil classification is based on particle size distribution curves and Atterberg limits tests, and decomposed granite soils are generally non-plastic or with a very low plasticity index. However, some highly weathered soils have almost the same properties as those of heavy clayey soil and very large plasticity indexes. In the case of soils with 15% or higher rates of passing, the No. 200 sieve, fine grained soil determines the behavior of the entire soils, and their mechanical behavior cannot be considered as being similar to those of sandy soil even if they are non-plastic or have low plasticity indexes. According to Unified Soil Classification System (USCS), soil are mostly classified into sand with silty fines SM or sand with clayey fines SC and treated as sandy soil. However, when they have been completely weathered near the ground surface, they change into moisture content (MC) or mixed lining (ML) which is clayey soil (Lan et al., 2003).

As for decomposed granite soil's compaction properties, as with other soils, when compaction energy increases, its maximum dry density increases while its optimum moisture content (OMC) decreases. Among decomposed granite soil's characteristics, one of the differences from other soils is particle crushing. Particle crushing refers to changes in the conditions of particles by external forces, such as shear, compression and compaction as compared to soils in natural conditions. Although, the particles are assumed not to be crushed in basic assumptions in soil mechanics as they are considered as incompressible rigid bodies, in reality, they are partially crushed depending on confining stress or failure loading (Wang et al., 2010). There are views that consider micro-cracks and intra-voids developed in the particles as major causes of the crushing and other views that consider the processes for mica or feldspar other than quartz to become fine-grained as internal causes of the crushing (Karimpour and Lade, 2010). Grain size distribution, shapes, strength, confining stress and the existence of pore water are considered as external causes that determine particle crushing. If confining stress is larger, the particles are more angular and the particles are larger, contact faces will increase so that contact stress is intensified and thus, larger particle crushing appears (Ham et al., 2010).

It is well known that soil permeability is determined by particle sizes and soil structures, and in general, the permeability of decomposed granite soil is in a range of 10^{-3} to 10^{-5} cm/s. However, if decomposed granite soil is compacted, particle crushing occurs and the soil will become denser and thus the permeability will decrease down to even below 10^{-5} cm/s. Unlike sand, decomposed granite soil is characterized by the fact that Darcy's law is not applicable. Since water flows in compacted soil are viscous flows, the viscosity of absorbed water increases so that the water behaves like water of crystallization. Because of this, a certain amount of threshold hydraulic gradient is necessary for water flows to occur and particles do not stop at small pressure head differences, and thus Darcy's law is not applicable (Yin, 2009).

Finally, to review strength properties, decomposed granite soil is well-graded, has low plasticity, has larger strength when particles are coarser and more angular and is less affected by water contents as compared to in-situ soil. Since in-situ soil shows conditions of having been apparently pre-consolidated to some extent by geostatic stress that has acted on the bed rock, it shows a characteristic to swell at shearing within the range of engineering stress. While compacted granite soil has no cohesion between particles and well-structured particle alignment as crushing has occurred in the processes of compaction and shearing, in-situ soil has cohesion and quite irregular particle alignment due to the components of its bed rock. Therefore, even if decomposed granite soil before compaction has the same void ratio as the soil after compaction, it will have different ultimate strength

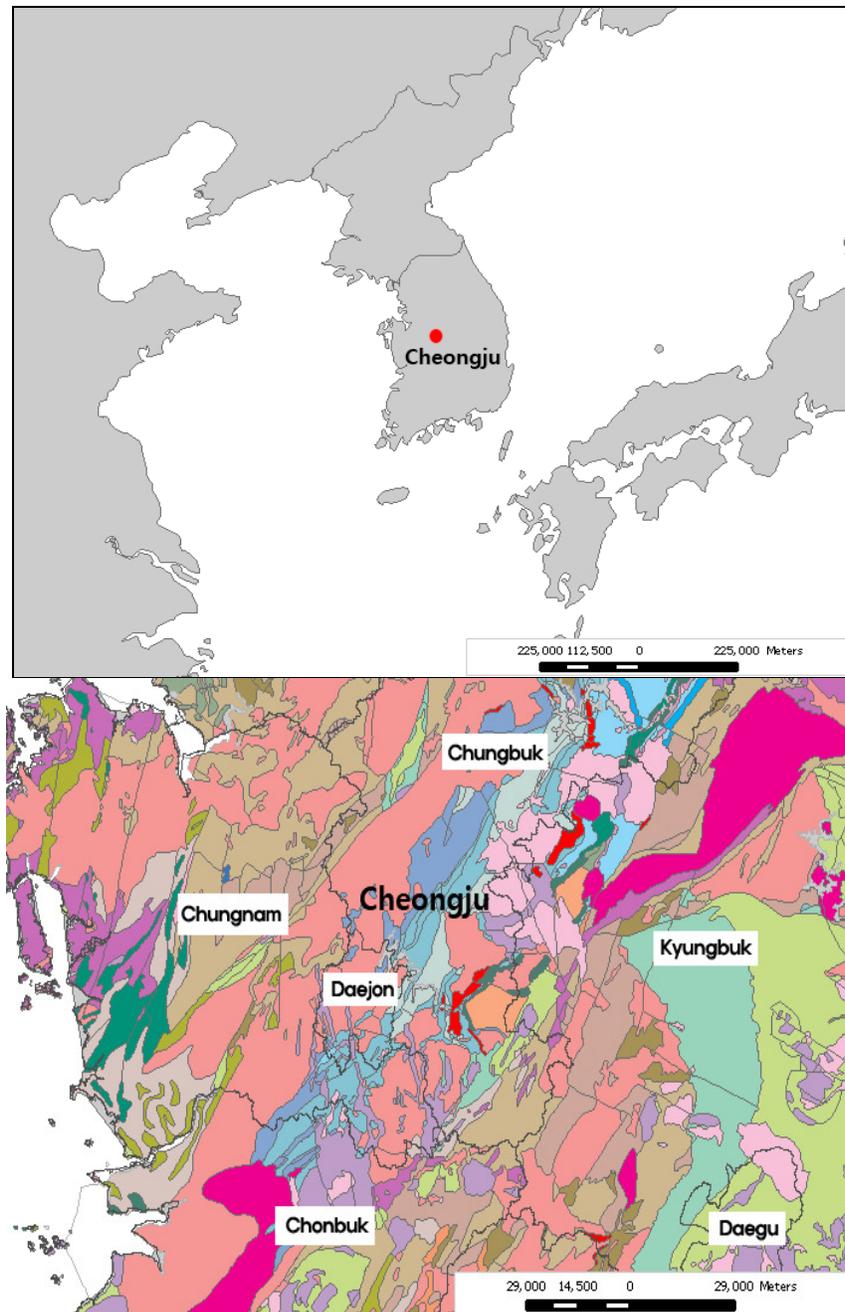


Figure 1. Location and geological features of Cheongju.

after being subjected to shearing stress due to the cohesion and irregularity of alignment and it will keep being deformed beyond the peak point to reach a certain residual strength while losing cohesion (Hossain and Yin, 2010a, b).

Physical properties of Cheongju granite soil

Cheongju is an inland city located 128 km from Southeast

of Seoul, Korea, and is a well-known place of wide range of decomposed granite soil as shown in Figure 1. The decomposed granite soil in Cheongju region that was selected for decomposed granite soil behavior analysis was received in a disturbed condition after being removed of surface soil. The condition of the specimens was microcrystalline granite soil with some remaining bed rock structures composed of mostly sandy soil and some soil in the process of argillization. Since decomposed granite soil has different mechanical properties depending on the

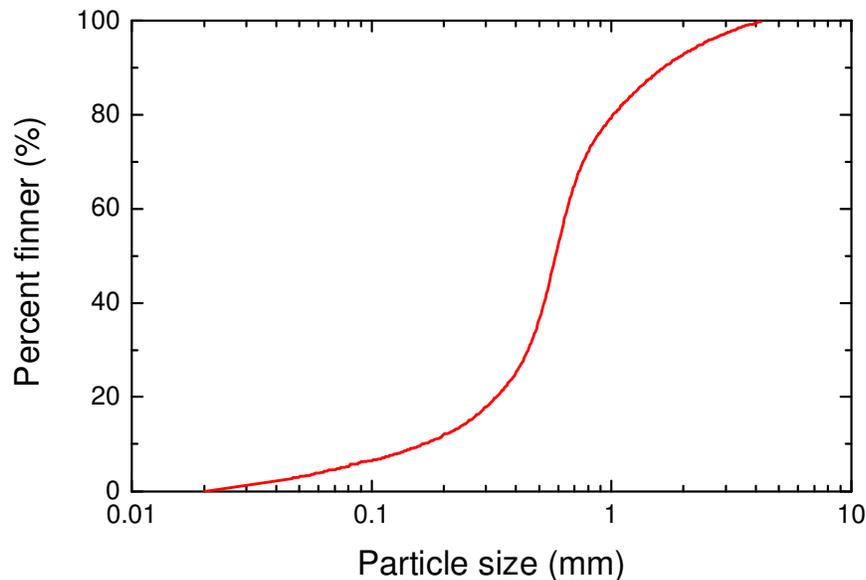


Figure 2. Particle size distribution curve of the sample.

Table 2. Energy-dispersive X-ray spectroscopy (EDS) analysis.

Element	Weight (%)	Atomic (%) [*]	Precision 2sigma	K-ratio ^{**}
N _a	0.00	0.00	0.00	0.0000
Al	20.32	22.79	0.13	0.2191
S _i	60.43	65.12	0.20	0.5528
K	6.56	5.08	0.08	0.0710
C _a	1.18	0.89	0.04	0.0135
T _i	0.71	0.45	0.04	0.0083
F _e	7.91	4.29	0.11	0.0998
C _u	2.89	1.38	0.10	0.0354

Total: 100; *: Atomic percent is normalized to 100; **: K-ratio = K-ratio × R, where R = reference (standard)/reference (sample).

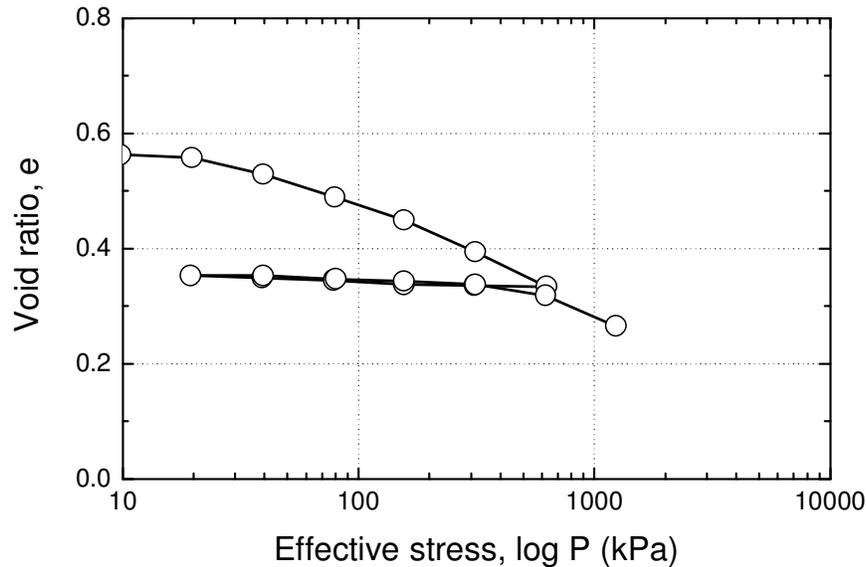
degrees of weathering, basic mineral components were identified through X-ray diffraction tests (XRD) which are a sort of mineral analysis as shown in Figure 2 and Table 2. At the same time, particle size distribution tests, compaction tests, Atterberg limit tests, permeability tests and consolidation tests were conducted in order to determine the physical properties. The physical properties of the soil specimens obtained through these tests are summarized in Table 1. Figure 2 shows a particle distribution curve for decomposed granite soil. It shows a ratio of passing the No. 200 sieve of 4.79%, a uniformity coefficient (C_u) of 3.35 and a coefficient of curvature (C_c) of 2.18.

A conventional incremental loading (IL) consolidation test was conducted in order to grasp the compressibility of the decomposed granite soil and the results are shown in Figure 3. The reason why clear yielding stress is not shown on the normal consolidation line is that the

specimens are not transported soil, but is residual soil, and thus no preconsolidation pressure exists. The e -log p relations in the processes of swelling and reconsolidation also show reversibility and thus, the test is showing the same tendency as general consolidation tests. On reviewing the compression curve, it can be seen that the specimens are similar to sandy soil in terms of drain rates and shows the same behavior as clayey soil in terms of settlement that occurs over a long time. At least 90% of the entire volumetric deformation occurred at the beginning (within 5 min) of the consolidation test, and although deformation after one hour was less than 1% of the entire deformation, it occurred continuously. It is considered that granite soil takes 350 to 750 h to complete consolidation and consolidation settlement continues even after complete dissipation of excess pore water pressure, because of the effect of gradual particle crushing (Miura and Ohara, 1979).

Table 1. Physical properties of the sample.

Special gravity (G_s)	Natural water content ω_n (%)	Plasticity index (PI)	USGS	γ_{dmax} (tf/m^3)	OMC (%)	Permeability (cm/s)
2.661	9.23	NP	SM	1.79	16	9.62×10^{-5}

**Figure 3.** Compression curve in loading and unloading procedure by consolidation test.

MECHANICAL PROPERTIES OF CHEONGJU GRANITE SOIL UNDER SHEAR STRESSES

CTC tests are essentially conducted not only to obtain soil parameters for various mechanical models, but also for stress-strain behavior analysis. In this study, in order to examine the mechanical behavior of Cheongju granite soil under shear forces, triaxial compression tests were conducted under consolidated-undrained (CU) test conditions and consolidated-drained (CD) test conditions. In the case of granite soil, shear strength or deformation behavior vary greatly with specimen preparation methods. Therefore, the specimen was made to have at least 95% compaction energy based on Proctor compaction test, method A in order to maintain compaction, permeability and crushing properties in the process of specimen remolding and thereby secure the mechanical uniformity of the specimen and minimize particle separation effects. The size of the specimen was determined to be a diameter of 35 mm and a height of 89 mm, and in order to enhance the degree of saturation of the specimen, the specimen was left unattended for 72 h while being applied with the flowing water method (Lowe and Johnson, 1960), the backpressure method (Black and Lee, 1973), the CO₂ method (Lade and Duncan, 1977) and vacuum method (Lowe and Johnson, 1960)

simultaneously. All test series could be started when pore water pressure coefficient B is over 0.98 to prevent low degree of saturation effect. When the saturation has been completed, confining stress, σ_c was loaded to conduct consolidation for 24 h. In this study, four levels of confining stress-49, 98, 196 and 294 kPa were applied to compose test series. Under undrained conditions, a strain rate of 0.4 mm/min was applied and in test series where drained conditions were applied, shear stress was loaded at a strain rate of 0.1 mm/min. As loading time during consolidation, although the time during which the primary consolidation is completed or 100 min for specimens with a diameter of 35 mm and 150 min or longer time for specimens with a diameter of 50 mm are generally used, in this study, 24 h was determined to be one cycle considering that many things were yet to be specified in relation to loading time. Based on the compression curve of the 35 mm specimens used in this study, secondary consolidation curves were generally drawn when 30 min had passed. In order to verify the data reproducibility, experiments were repeated at least twice. Figure 4 shows a schematic diagram of the CTC test used in this study. Volumetric strain control device (VSCD) is the most important experimental apparatus to control the pore water movement precisely. It consists of stepping motor, piston and pore water cell as shown in Figure 5, and the

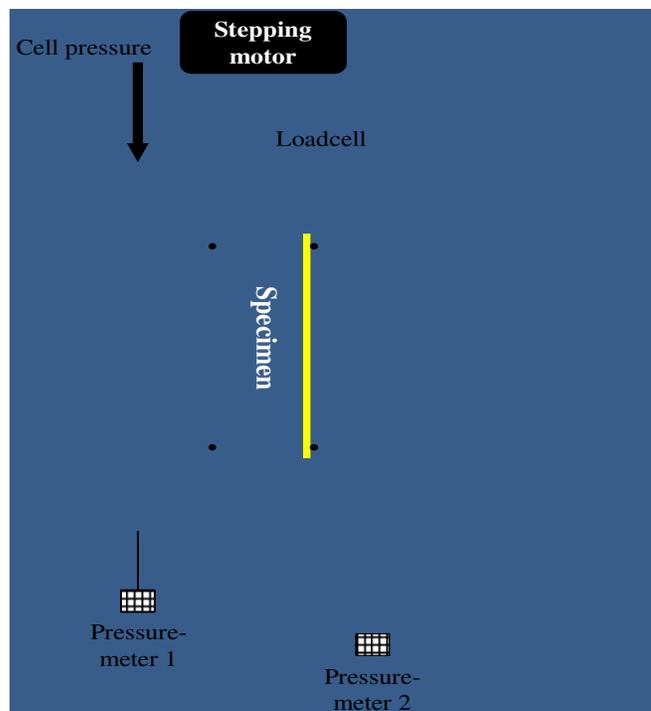


Figure 4. Schematic diagram of CTC test equipment.

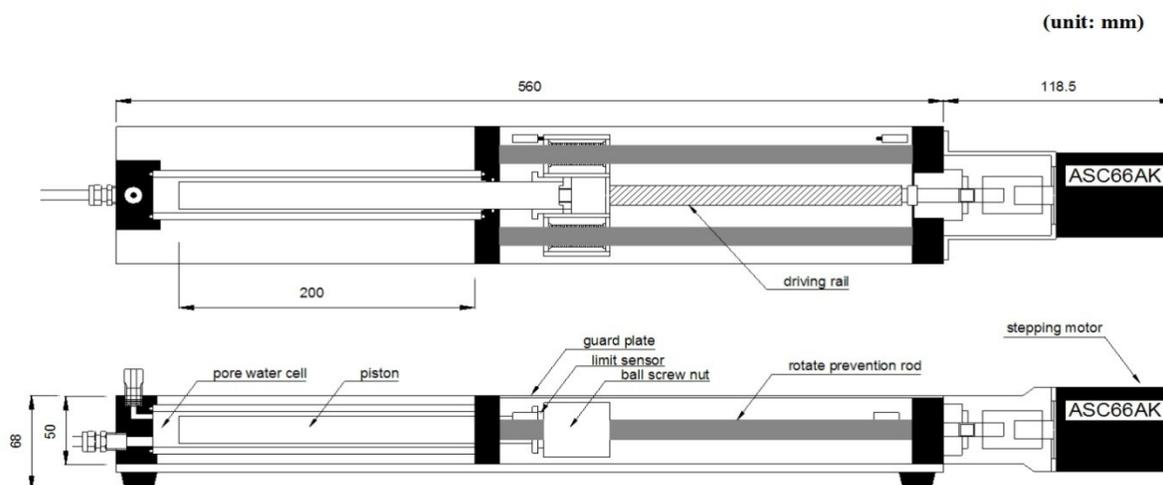


Figure 5. Scheme of the volumetric strain control device, VSCD (Kwon et al., 2007).

resolution of VSCD was $6.3 \times 10^{-5} \text{ cm}^3/\text{pulse}$.

Isotropic compression test

The test was conducted by confining stress increasing from 49 to 98, 196 and 294 kPa isotropically at intervals of 24 h on specimens made to have at least 95% compaction energy by proctor compaction test, method A and then, unloading the confining stress in reverse order. Figure 6 shows the confining stress-volumetric strain

relation obtained from the isotropic loading-unloading compression test. Since plastic expansive failure does not occur in isotropic compression tests because shear deformation does not occur in these tests, isotropic compression tests are used to calculate the plastic collapse strain of double surface work-hardening constitutive model (Lade, 2007).

CTC test under consolidated-undrained condition

In CU tests, constant isotropic pressure is applied to

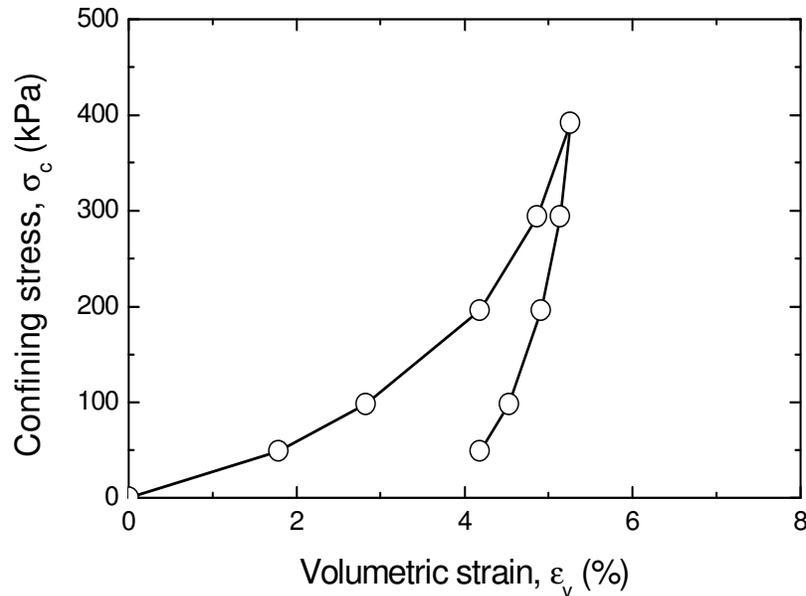


Figure 6. Confining stress-volumetric strain relation from isotropic loading-unloading compression test.

specimens while drain is allowed to consolidate the specimens until volume changes are completed and then shear tests are conducted under undrained conditions to measure deviator stress against axial strain and related pore water pressure. The reason that we selected the testing method, CU test is that Cheongju granite soil is isotropically consolidated in natural state, and then experienced shear external stresses in general condition. Therefore, CU test is most meaningful approach in engineering viewpoint for Cheongju granite soil.

Figures 7 and 8, respectively show the relation between the deviator stress and axial strain and the pattern of occurrence of pore water pressure in relation to axial strain. First, to review the stress-strain relation of the Cheongju granite soil illustrated in Figure 7a, strength development varies with confining stress. As confining stress increases, the maximum deviator stress obtained from each specimen linearly increases and this is considered as indicating great changes in the mechanical behavior of the specimen. In case confining stress is relatively small, yielding stress in a range of 2 to 4% appears to show hardening-softening behavior, while in case confining stress is large, yielding stress in a range of 5 to 7% to show hardening-consistent behavior. In Figure 8a, under undrained conditions, as axial strain increased, pore water pressure increased continuously and in case confining stress was large, increasing tendencies clearly appeared. In all the test cases, pore water pressure had its maximum value at the level of 50% of the confining stress.

In particular, Figure 7b and Figure 8b, respectively show deviator stress and pore water pressure normalized by the confining stress applied as individual test

conditions. When the effect of confining stress had been excluded, both deviator stress and pore water pressure were shown to exist on a single trend line thereby indicating that the deformation characteristics and stress distribution of the Cheongju granite soil could sufficiently be predicted based on these results. The degree of pore water pressure development in relation to confining stress was being affected by the degree of saturation and it can be assumed that this is because the degree of pore water pressure development was being affected by particle crushing effects in the consolidation and shear processes.

CTC tests under consolidated-drained condition

In the CD-test, it was intended to examine the strength and volume change characteristics of the Cheongju granite soil. When shear stress is applied, strain rates where pore water pressure would not develop should be applied and in this study, $0.1\% \text{ min}^{-1}$ was selected as an appropriate strain rate. Basically, we used the stress-controlled method to develop shear stresses to specimen.

Figures 9 and 10, respectively show the relation between deviator stress and axial strain and the relation between volumetric strain and axial strain in the CTC test conducted under drained conditions. As shown in Figure 9a, it could be identified that as with the drain test, the stress-strain relation greatly relied on confining stress and maximum deviator stress was linearly increasing. Along with it, it could be seen that maximum deviator stress existed at 4% based on axial strain. In the case of volumetric strain, when confining stress was smaller,

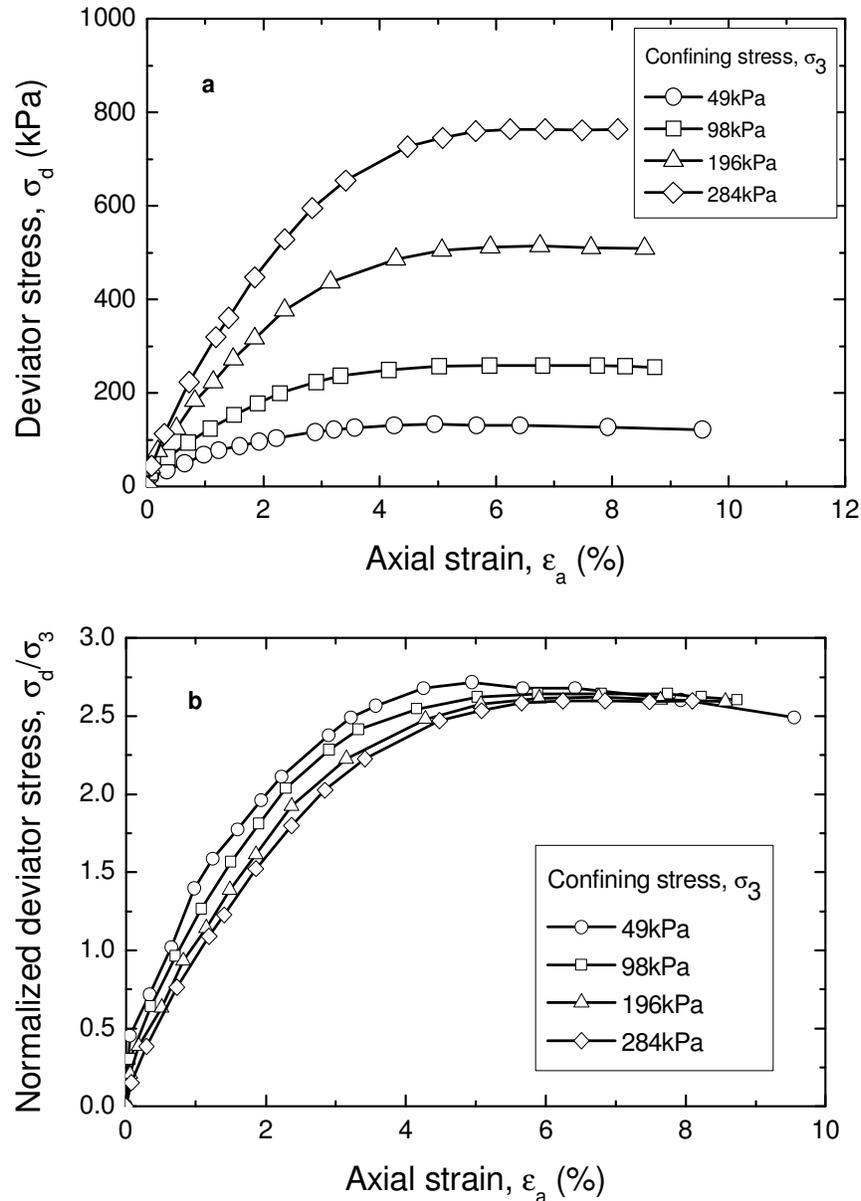


Figure 7. Relation between deviator stress and axial strain (a) and normalized relation with confining stress (b) by CTC test under undrained condition.

larger dilatancy effects of switching from compression to swelling were occurring.

Figures 9b and 10b, respectively show the results of normalization of deviator stress and volumetric strain. Similarly, to undrained conditions, the deviator stress could be almost expressed by one trend line when the effect of confining stress was removed. Figures 9b and 10b show a uniqueness of stress-strain relationship of granite soil, and it is a general mechanical behavior of soil, not only granite soil. In other words, stress-strain behavior of soil is completely independent of confining pressure. Actually, if it depends on confining pressure, there are many problems in generalizing soil behavior

particularly in stress-strain relationship. We also concluded that a uniqueness of stress-strain relationship of granite soil is widely recognized by the test results as in Figures 9b and 10b, and it means at the same time experimental series are well-established. It could be seen that although, volumetric strain maintained the same relation at the initial compression stage in all cases, dilatancy occurred more rapidly when confining stress was smaller after 3% axial strain.

The distributions of confining stress and maximum deviator stress using the results of CTC tests under CU and CD conditions are as shown in Figure 11. As shown in the figure, confining stress has almost linear relations

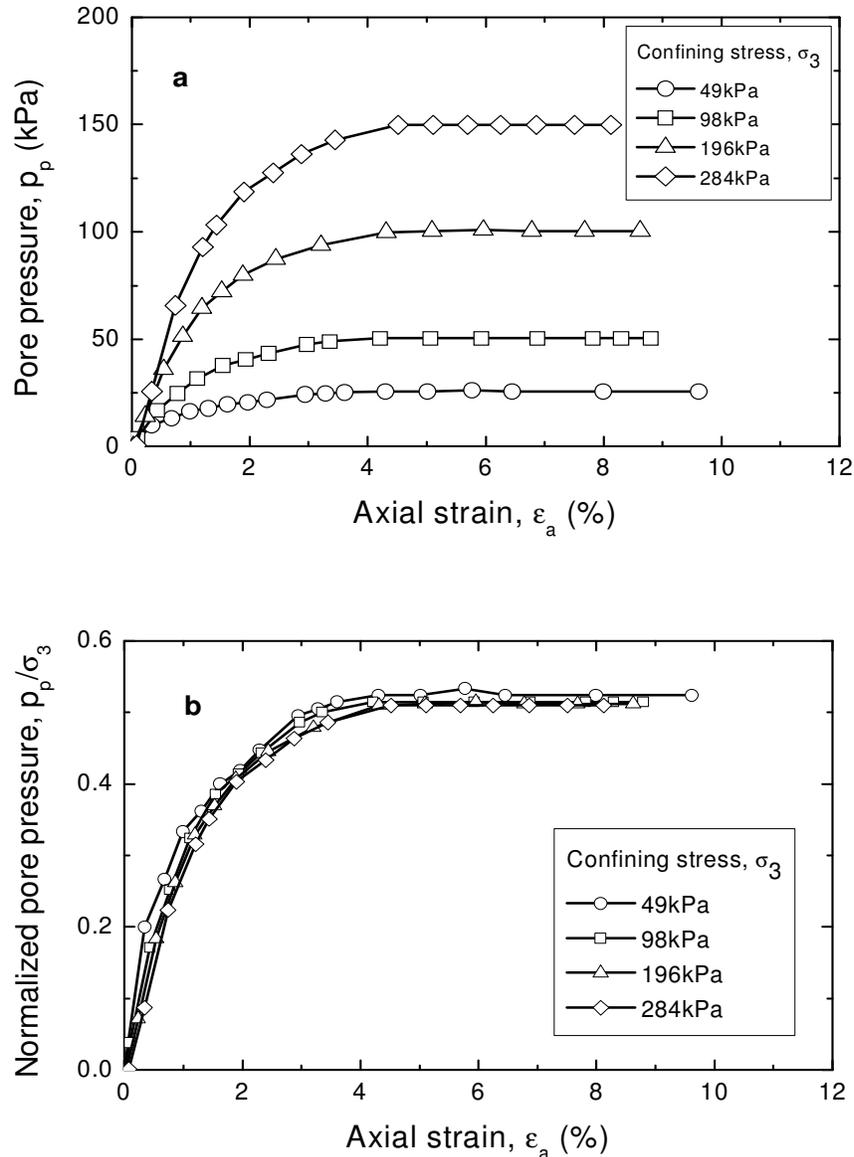


Figure 8. Relation between pore water pressure and axial strain (a) and normalized relation with confining stress (b) by CTC test under undrained condition.

with maximum deviator stress regardless of whether drain conditions are undrain or drain and the slope of this linear relation obtained through experimental data fitting while disregarding these drain conditions was 2.95. This slope of linear relations is one of the essential parameters in implementing numerical approaches to the Cheongju granite soil. On comparison of the drained or undrained shear strength of the decomposed granite soil under the concept of effective stress separately from discussion on quantitative estimations, it can be seen that failure lines expressed with effective stress in the results of these tests can be uniquely determined, although differences made by drain conditions or stress paths may be acknowledged to some extent.

STRESS PATHS OF CHEONGJU GRANITE SOIL

Shear properties of soil are determined by shear strength relative to stress conditions at the times of failure. To accurately grasp shear properties of soil, stress-strain relations should be examined in detail. To this end, in this study, stress paths will be tracked based on the concept of critical states in order to explain the stress properties of the Cheongju granite soil.

Large streams of theoretical soil mechanics, thus far can be divided into schools centering on Roscoe of Cambridge and schools centering on Rowe of Manchester University, whereas the former deals with soil as a continuum material, the latter is based on granular

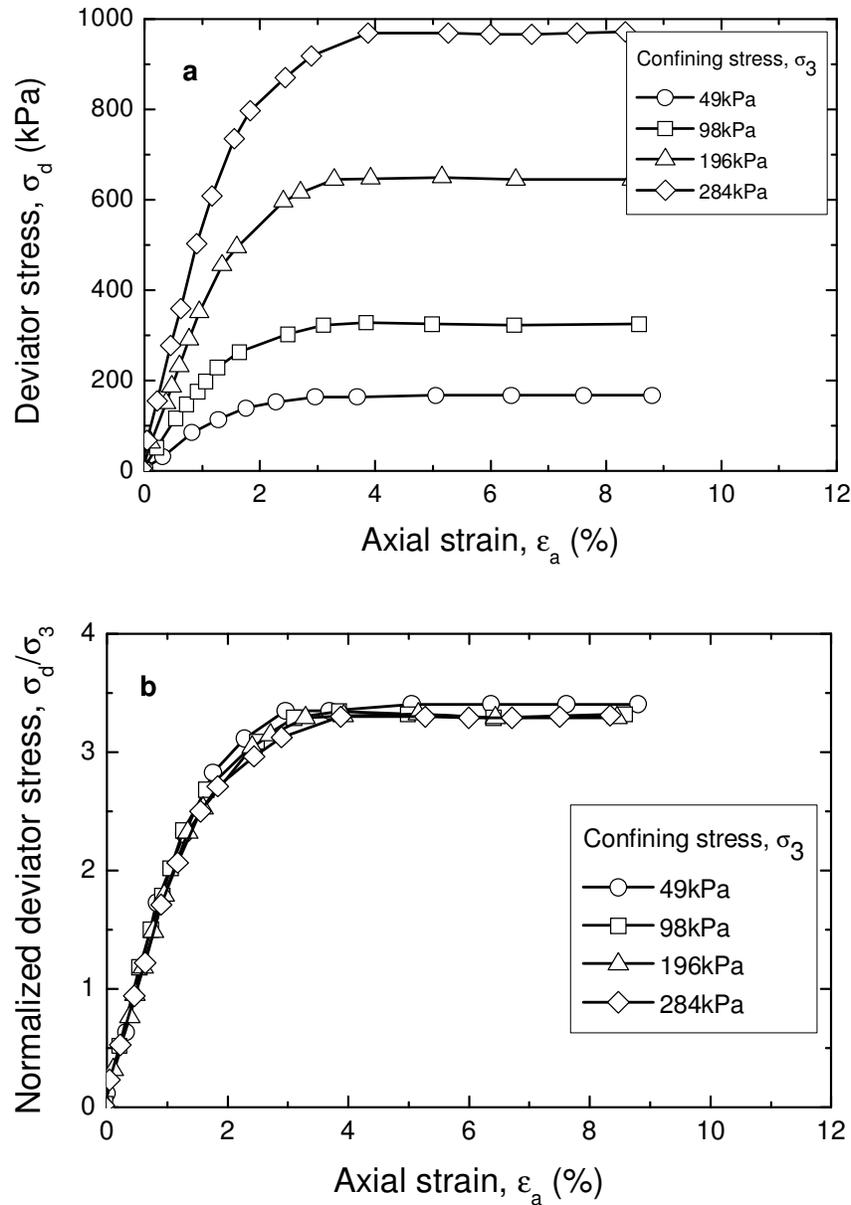


Figure 9. Relation between deviator stress and axial strain (a) and normalized relation with confining stress (b) by CTC test under drained condition.

granular mechanics focusing on the mechanical behavior of individual particles. When soil is dealt with as a continuum material, great difficulties are experienced in predicting the mechanical behavior of actual soil when extremely large soil deformation beyond a certain allowable limit occurs. In this study, the soil is examined from the viewpoint of the critical state theory that pays attention to the size of heterogeneity in heterogeneity.

In general, if soil experiences shear failure when it is under external forces, the principal stress ratio ($\eta = q/p$) will become a constant value and in this state, a state where only infinite shear deformation occurs without any

changes in the effective stress, and volume is defined as the critical state (Ng et al., 2004; Kumruzzaman and Yin, 2010), that is, when shear deformation of soil is large, the mean principal stress (p), shear stress (q) and void ratio (e) of the soil can be expressed as a line regardless of the stress history of the soil, drain conditions and test methods. This straight boundary line is called critical stress line (CSL) and the trajectory of stress changes that act along with increases in stress until the soil reach failure in the stress-strain relation in this case, which is called a stress path. In general, the stress path is expressed by connecting points that represent the sizes

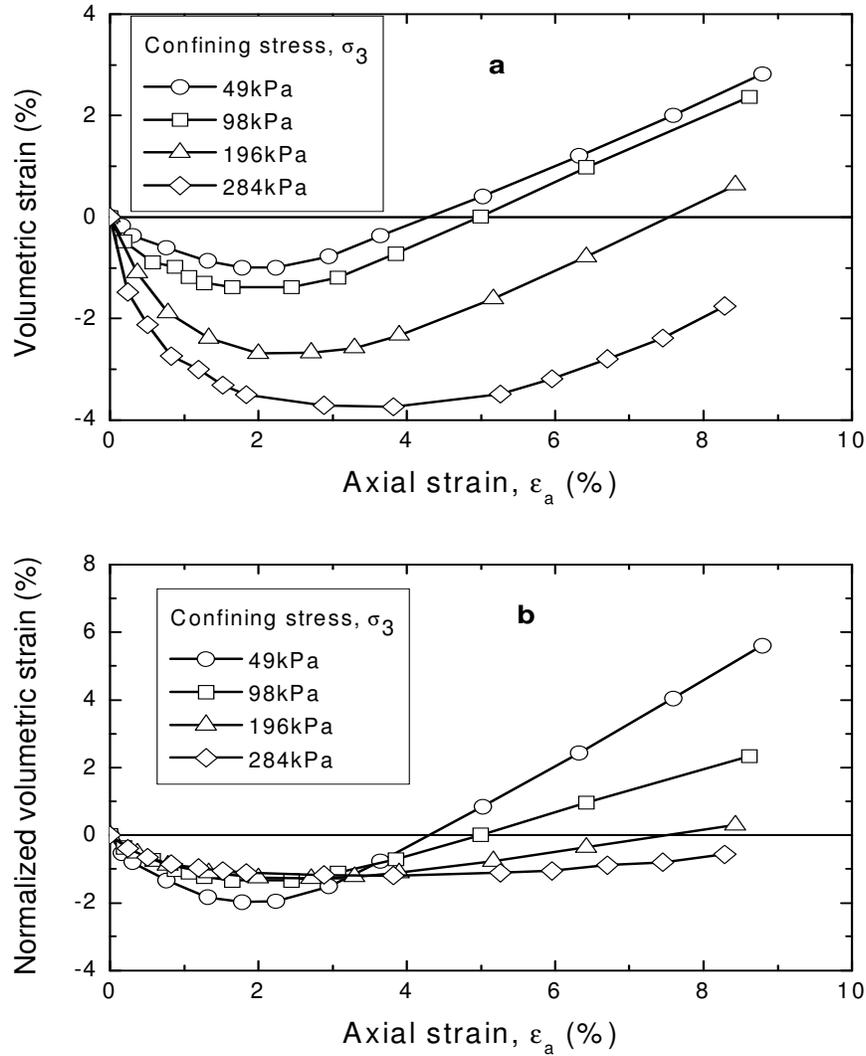


Figure 10. Relation between volumetric strain and axial strain (a) and normalized relation with confining stress (b) by CTC test under drained condition.

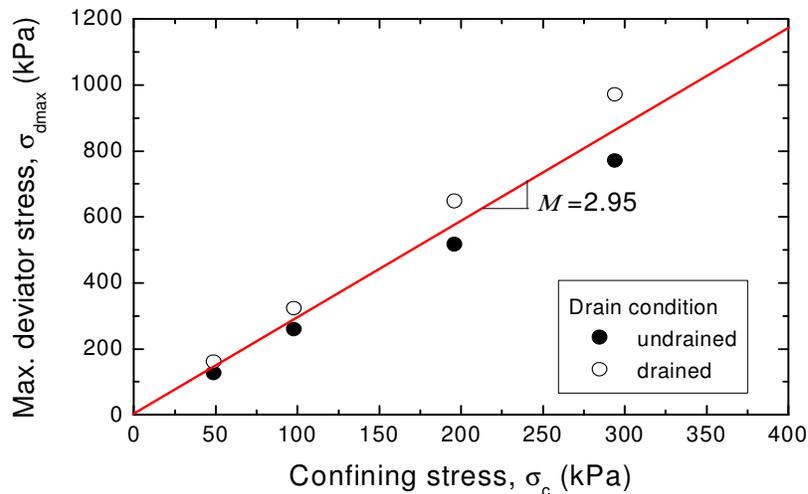


Figure 11. Relation between maximum deviator stress and confining stress.

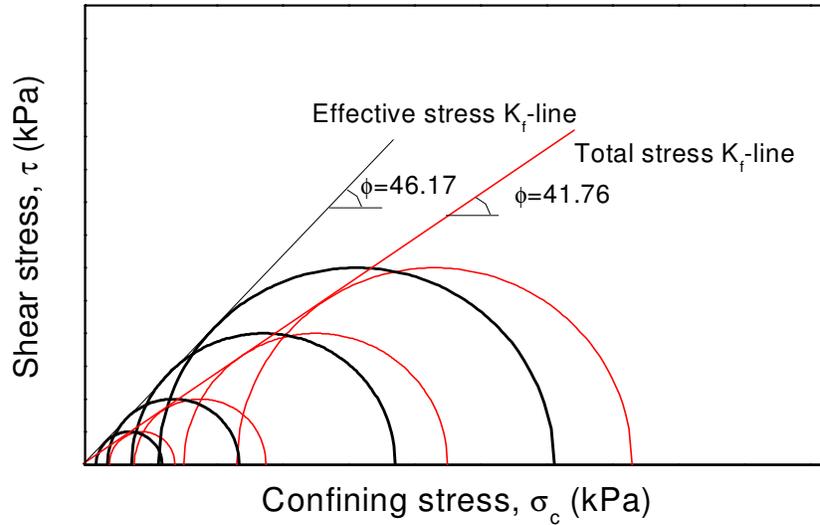


Figure 12. Mohr's circle of the specimen under CU condition.

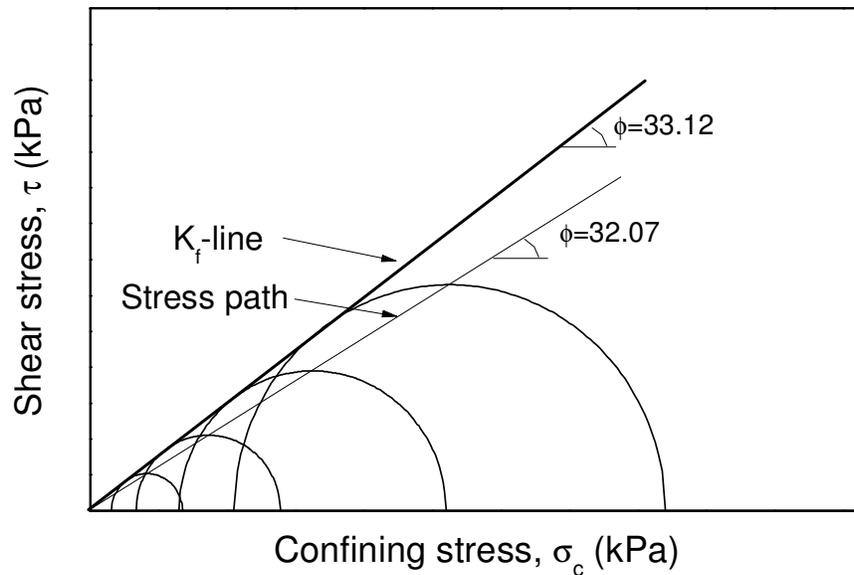


Figure 13. Mohr's circle of the specimen under CD condition.

of principal stress corresponding to strain in two dimensional coordination systems. This stress path was established as an axis of the critical state theory of soil stress-strain and there are two methods of illustrating the stress path in the three dimensional space used to trace the state of progressive shear behavior with the size and location of stress on Mohr's circles. In this paper, these parameters will be used to analyze the shear properties.

Figure 12 shows Mohr's circles obtained as a result of a CTC test conducted under undrained conditions and the one drawn in bold line is a Mohr's circle expressed using effective stress and the other one is a Mohr's circle

expressed using total stress. The internal friction angle determined using effective stress is 46.17° and the internal friction angle evaluated using total stress is 41.76° . From the results of tests conducted under drained conditions, an internal friction angle of 33.12° was obtained as shown in Figure 13. As can be seen from the Mohr's circles obtained through tests under the two drain conditions, cohesion is shown to 0 kPa in all cases. It is considered that this is because the Cheongju granite soil is a microcrystalline soil that is not completely weathered.

In Figure 14 where the results of isotropic compression tests were idealized, the λ -line is a normal consolidation

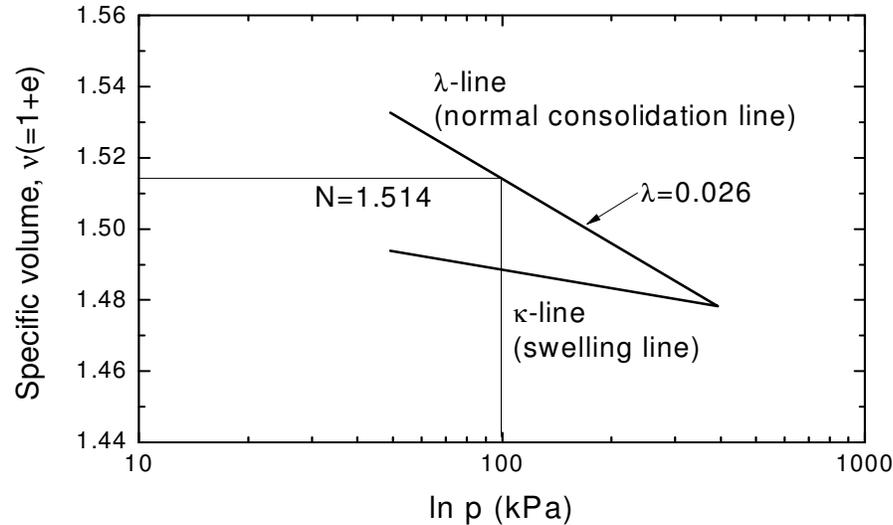


Figure 14. Idealization of isotropic compression curve.

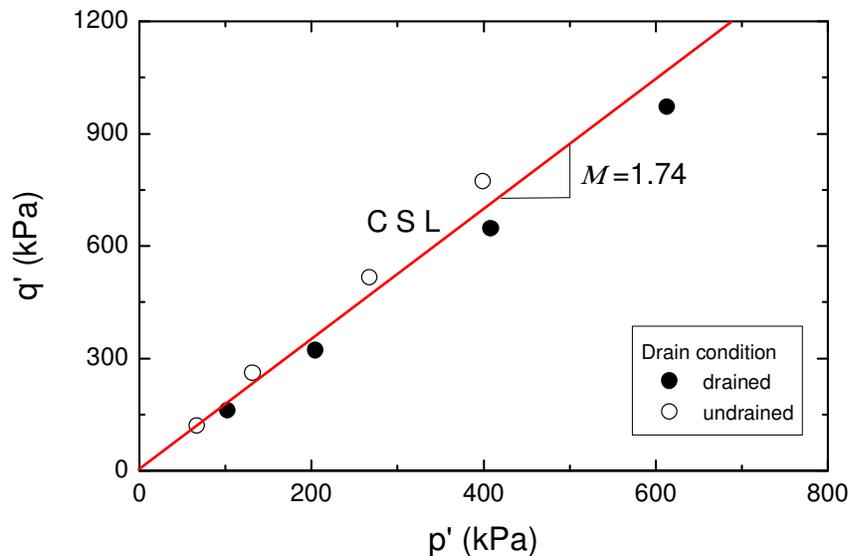


Figure 15. Failure points in p' - q' plane by drained and undrained CTC test.

line and λ is a slope which was evaluated to be 0.026 in the Cheongju granite soil. The κ -line is a swelling line and the soil parameter related with the specific volume was evaluated to be 1.514. Figures 15 and 16 show failure points obtained through CTC tests conducted on the q' - p' plane and the v - p' plane, respectively and along with these lines, CSL are illustrated. The slope of the CSL obtained from this q' - p' plane was shown to be 1.74.

Figure 17 shows an effective stress path obtained under undrained conditions, whereas the effective stress path tended to bend at the beginning of loading as with normally consolidated clay, effective stress linearly increased in the opposite direction to the curve thereafter

as pore water pressure development was suppressed. This is a characteristic of decomposed granite soil as well as being a characteristic of sediment soil. The end of the effective stress path went over the CSL to reach failure and then came down to below the CSL along the same path and this is similar to the behavior of over consolidated clay. This eventually means that the state boundary surface of the decomposed granite soil is staying in the space formed on the CSL.

Figure 18 shows a result of normalization of q' and p' obtained based on the test result in Figure 17. The reason why the data were normalized as such is to prove that the Roscoe surface projected on the surface to reach the CSL

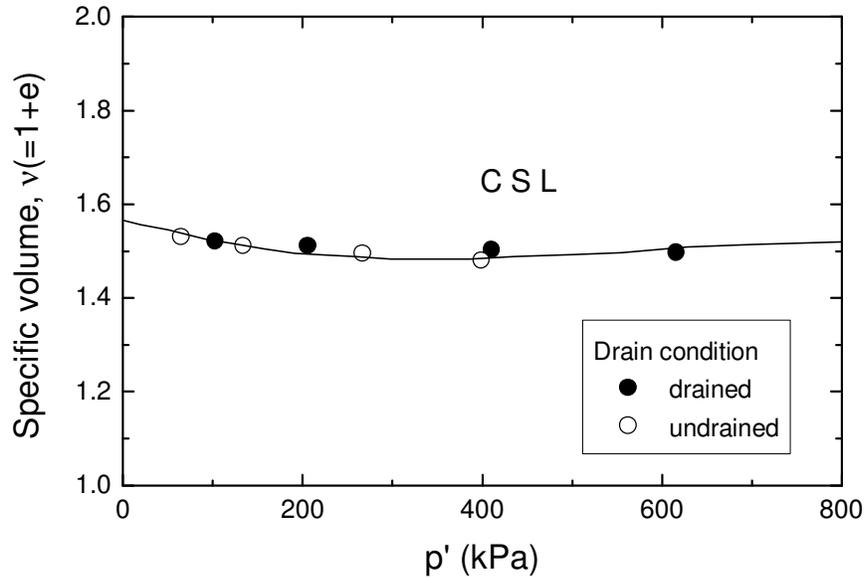


Figure 16. Failure points in v - q' plane by drained and undrained CTC test.

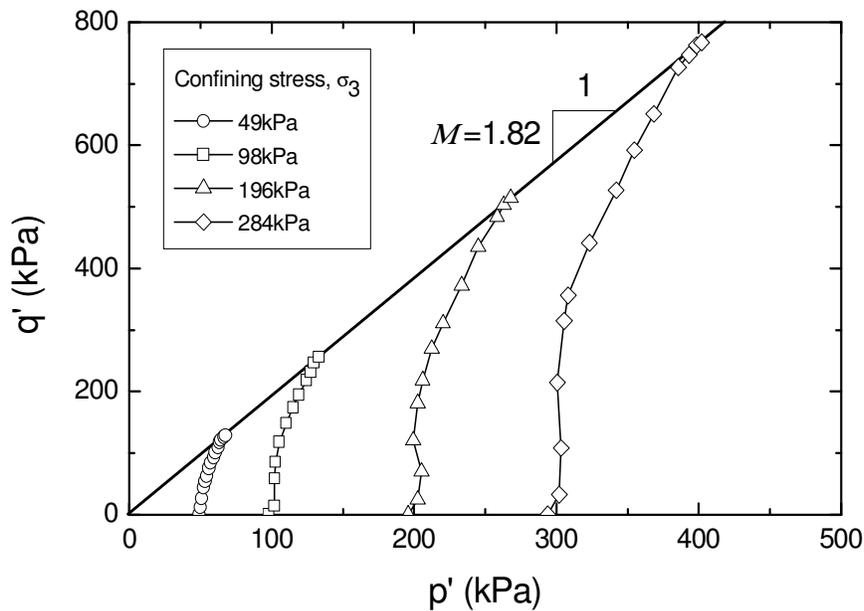


Figure 17. Stress paths in p' - q' plane by undrained CTC test.

through different stress paths under different confining stresses is unique, that is, when there are stress paths with similar shapes at each stress level on the same stress plane, this is a method to determine whether their trajectories projected on a certain reference projection surface are the same. Based on the results of the test, it could be seen that, on the Roscoe surface, if soils receive shear forces, they will reach the CSL through their own stress paths regardless of drain conditions, that this, stress face can be expressed as one curve and that the yielding surfaces are independent from

preconsolidation pressure. The criterion for normalization was the mean equivalent stress of each test case, P_e and mean effective normal stress p' and deviator stress q' were separately expressed using this value. The mean equivalent stress, P_e is expressed as per the following Equation 1:

$$P_e = \exp \left[\frac{(N - \nu)}{\lambda} \right] \quad (1)$$

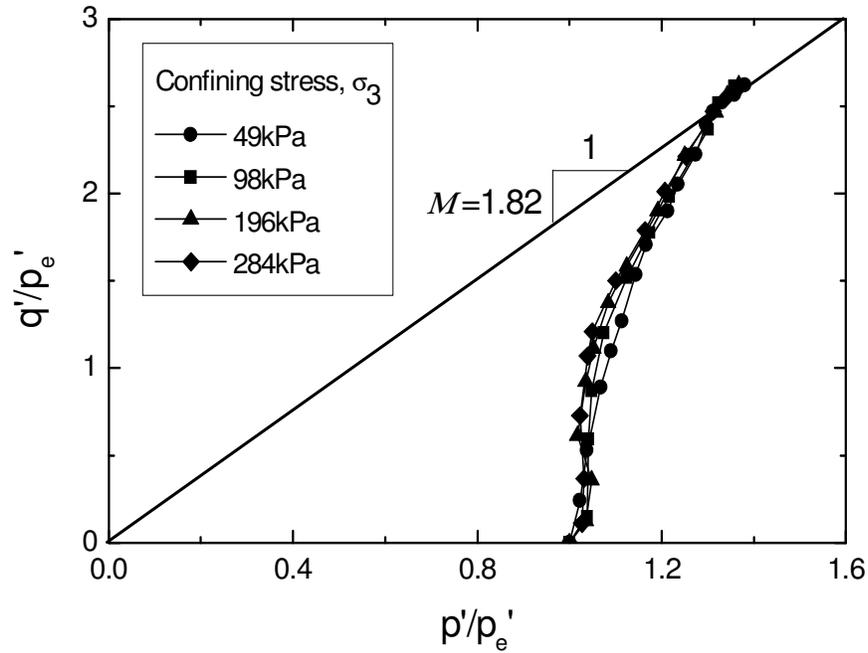


Figure 18. Normalized stress paths by undrained CTC test.

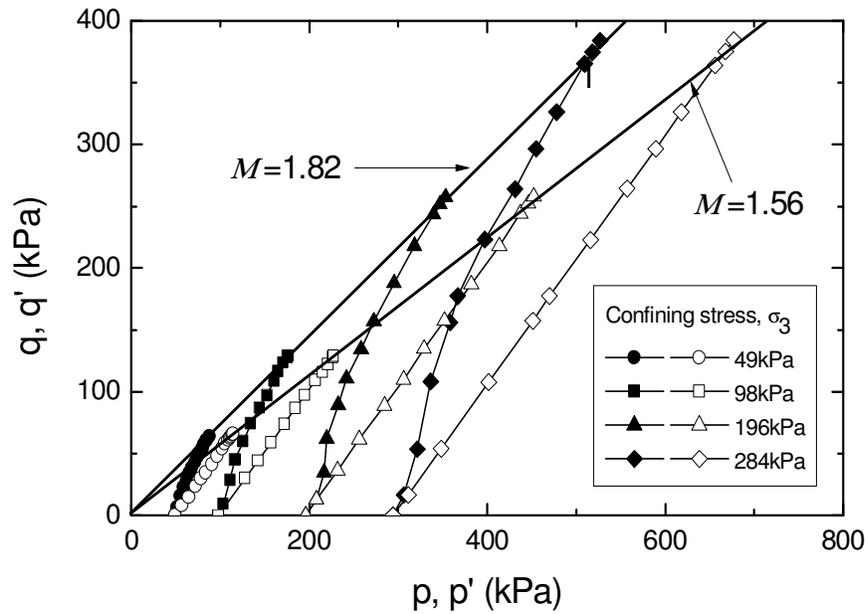


Figure 19. Effective stress path and total stress path.

where, parameter N represents the specific volume when the value of P_e is 1 in the normal consolidation line and λ is the slope of the normal consolidation line as shown in Figure 14. Since $e = e_0$ in the tests under undrained conditions, confining stress becomes the same value as mean equivalent stress in those tests. By using mean equivalent stress as a reference value for normalization, it becomes possible to express the accuracy of the test

and the state boundary surface.

Figure 19 shows a comparison between an effective stress path and a total stress path that shows the effect of pore water pressure on stress paths. The effective stress path shows a path where pore water pressure develops, and thus the path bends similarly to typical normally consolidated clay at the beginning of which the occurrence of pore water pressure is very scarce thereafter and

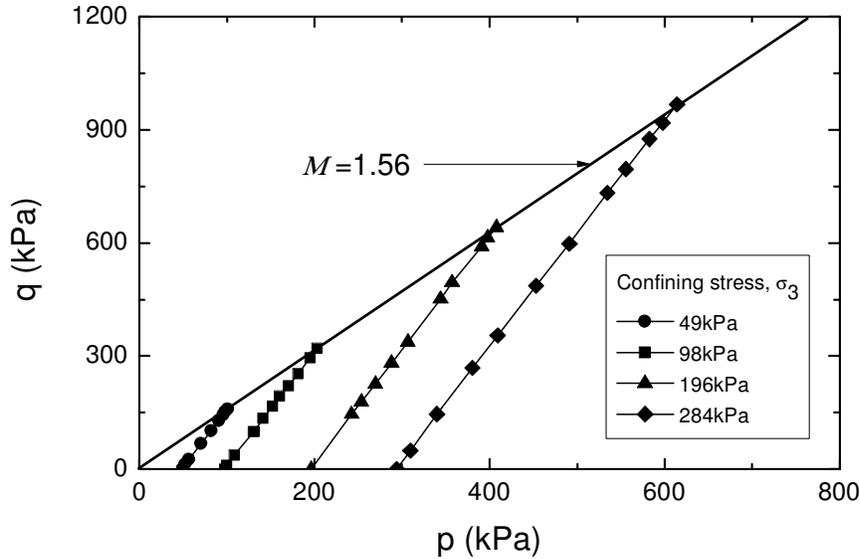


Figure 20. Stress paths p - q plane by drained condition.

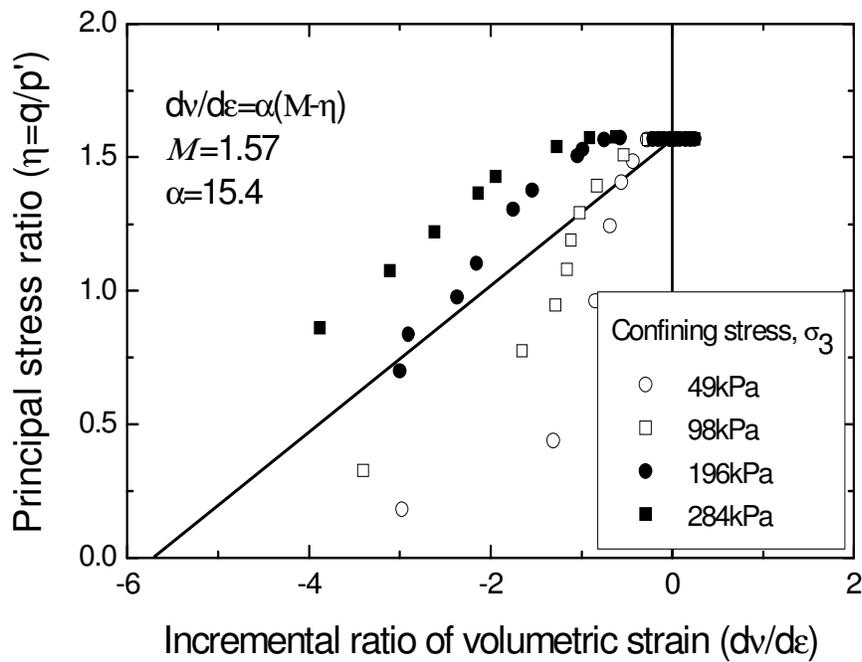


Figure 21. Relation between principle stress ratio and volumetric strain.

thus, the effective stress path shows a tendency to become straight or to turn over.

Figure 20 illustrates the stress path in a drained CTC test drawn on a p - q plane and Figure 21 shows the results of the drained CTC test as the relationship between principal stress ratio, $\eta (= q/p')$ and incremental ratio volumetric strain ($dv/d\epsilon$). In this case, strain increment $d\epsilon$ was determined using the following equation:

$$d\epsilon = \frac{2}{3}(d\epsilon_1 - d\epsilon_3) = \epsilon_a - \frac{1}{3}(\epsilon_v). \tag{2}$$

As shown in Figure 20, if it is assumed that volumetric strain ($dv/d\epsilon$) in decomposed granite soil can be expressed as being in a linear relationship with principal stress ratios, the volumetric strain can be roughly approximated with the following formula regardless of

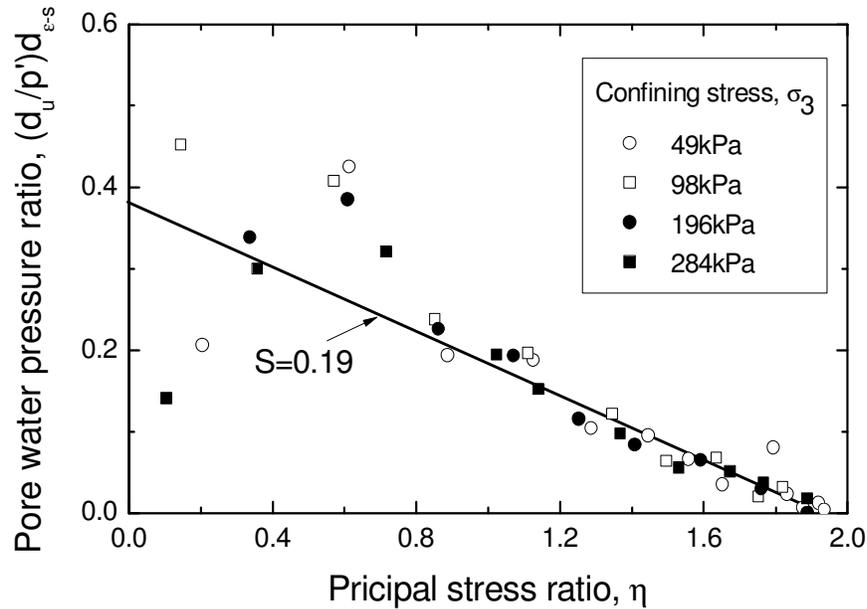


Figure 22. Relation between principle stress ratio and pore water pressure ratio.

water contents, degrees of weathering and confining stress:

$$dv/d\varepsilon = \alpha(M - \eta). \quad (3)$$

where, M is the stress ratio when $dv/d\varepsilon = 0$, that is, when volumetric strain is converted into compression during shearing and has a value of around 1.57, α represents the slope of this linear relationship.

In undrained CTC tests, since volume changes do not occur and thus specimen volume has a constant value, the stress should be expressed using pore water pressure instead of $dv/d\varepsilon$. Figure 22 shows the relationship between principle stress ratios, η and pore water pressure ratios, and indicates that maximum pore water pressure occurs when η is 1.92. The point where the pore water pressure ratio is 0 becomes the point where failure on the CSL and since pore water pressure did not show softening in the test where decomposed granite soil was used, negative values were not observed in pore water pressure in the test.

CONCLUSION

The results obtained through basic physical properties and CTC tests using decomposed granite soil sampled in Cheongju and examined in terms of weathering, shear and stress properties are summarized as follows.

The drained and undrained CTC tests conducted with different confining stresses after performing isotropic consolidation are basic experiments in interpreting

decomposed granite soil's stress-strain behavior using Lade's single and double surface work-hardening constitutive model. Though, these tests, stable results were obtained in terms of deviator stress against axial strain and the relationship between pore water pressure and volumetric strain, and it can be seen that in the relationship between confining stress and maximum deviator stress, the slope is maintained at a constant value of 2.95.

As a result of an undrained CTC test, when confining stress as yielding stress was relatively smaller, axial strain was also smaller and showed hardening-softening, but showed general consistency when normalized. In the drained CTC test, maximum deviator stress generally existed in a range of axial strain of 6 to 8% and larger dilatancy phenomena appeared when confining stress was smaller. When normalized, deformation showed differences up to around 7%.

Cohesion could not be observed in Mohr's circles obtained through triaxial compression tests and the internal friction angle of the effective stress path was 46.17. In the 3D space of $p'-q'-v$, the slope (M) of the CSL was shown to be 1.74. The effective stress path in the $p'-q'$ plane was normalized and based on the results, the effective stress path showed behavior similar to that of normally consolidated clay at the beginning while showing a tendency of going upward-rightward in some part when it went over the CSL. The principal stress ratio that becomes the turning point between compression and swelling by shear force obtained by organizing the results of drained CTC tests as the relationship between principal stress ratios and the incremental ratios of volumetric strain was shown to be 1.65. Based on the

results of the CTC tests on Cheongju granite soil, although axial strain, deviator stress and pore water pressure showed mechanical properties similar to those of overconsolidated soil, Cheongju granite soil showed behavior similar to that of normally consolidated soil in terms of volumetric strain.

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