Anisotropic Behavior of Decomposed Granite Soils
화강풍화토의 비등방성 거동특성

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ABSTRACT

화강풍화토의 강도와 변형특성을 조사하기 위하여 불포화배수 삼축압축시험을 실시하였다. 0, 45, 90 , . 이차압축시의 변형거동에 관한 시간의존성은 다짐각도와 관계가 없는 것으로 판단되었다. 다짐각도가 압축강도와 변형에 미치는 영향은 특히 낮은 구속압력시에 크다. 다짐각도가 다르더라도 다일러턴시 비율은 다일러턴시로 인한 강도증가와 상관하여 변화한다. 따라서 다짐풍화토는 초기 비등방성 조직을 갖고 있는 모래와 같이 비등방성 역학적 성질을 갖는다고 할 수 있다.

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1. Introduction

A series of unsaturated-drained triaxial compression tests were performed on compacted materials. Decomposed granite soil was used as this type of material has generally been adopted previously as a construction material. The tests were planned to find not only the degree of anisotropy for the sedimentation angle of the compacted material, but also the influences of the confining stress, degree of saturation and specimen preparation method on the anisotropic properties.

2. Material Properties and Test Methods

2.1 Basic Characteristics

The physical properties of the sample are shown in Table 1.

The quartz, feldspar and colored mineral contents of this sample were about 23, 62 and 15%, respectively.

Table 1. Physical properties of the soil used

Sample	Grain size (mm)	Gs	e_{max}	e_{min}	d_{50}	Ignition loss(%)	Uc	Ip	W_{opt}	ρ_{dmax}
decomposed granite soil	–	2.685	1.116	0.613	0.555	1.83	7.14	N.P	13.24	1.783

2.2 Sample Preparation

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The rectangular sample was frozen at under -20°C followed by cylindrical triaxial compression of the specimens into a 50mm diameter and 100mm height using a core bit machine, as shown in Fig. 1. The sample had three different angles of the axial(major principal) direction to the sedimentation plane(compaction plane), 0, 45 and 90 degrees.

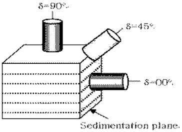

Fig 1. Schematic diagram of the sedimentation angle

2.3 Test

Unsaturated-drained triaxial compression tests were carried out on the specimens at a constant strain rate of 0.1mm/min. A double cell in the apparatus was equipped to measure the volume change of the specimen. The axial strain was measured using the gap sensor of up to 2.0% during the consolidation and initial part of the shear process.

The specimens were set up under a 20kPa confining pressure, with melting for about 6 hours. After this, confining pressures up to, 30, 60, 120 or 240 kPa were applied for times of 1 or 10hours, in order to find the time dependency of the mechanical behaviour of the unsaturated compacted materials.

3. Test results

3.1 Effect of the sedimentation angle on the compression settlement

Fig. 2 shows the relationship between the elapsed compression time and the axial strain for a sample subjected to a 60kPa confining stress. The amounts of axial strain during the primary and secondary compressions were referred to as S_1 and S_2, respectively, as shown in Fig. 2.

Fig. 3 shows the relationship between the axial strain and time under the conditions of 60kPa of confining pressure and 10 hours compressive stress. The arrow in the figure represents the moment the load ended. As δ of the specimen decreased from 90° to 0°, the axial strain S_1 increased.

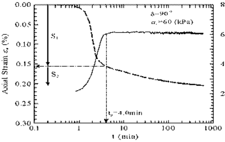

Fig 2. Relationship between axial strain, axial load and compression time

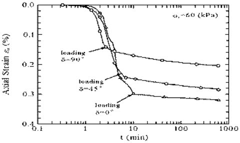

Fig 3. Relationship between axial strain and compression

Fig. 4 shows the effect of the confining pressure on the relationship between the axial strain and the duration of compressive stress for samples subjected to 30, 60, 120, 240kPa confining pressures for specimens with $\delta=90°$. As the confining stress increased, the amount of axial strain S_1 also increased. The arrows in the figure represent the moment the load ended. Then, the amount of axial strain, S_2, was approximately the same for all confining stresses. The behaviour of deformation during the secondary compression process was verified as not having any dependency on the confining stress.

Fig. 5 demonstrates the relationship between the duration and axial strain in the secondary compression process from Fig. 3. From Fig. 5, with longer duration, the rate of decrease became gentler. In addition, the behavior of deformation during the secondary compression process was dependent of δ. The influence of the sedimentation angle did not appear in the secondary compression process. S_2 was considered to be related to the particle rearrangement and decrease suction factors due to the compression.

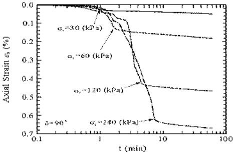

Fig 4. Relation between axial strain and compression

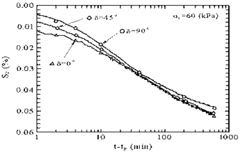

Fig 5. Relationship between S_2 and $t-t_p$

3.2 Effect of the sedimentation angle on the strength

In order to investigate the shear property of the compacted specimen with 90, 45 and 0 degree sedimentation angles, drained triaxial compression tests were performed. The deviator stress-axial strain and axial-volumetric strains diagrams for the specimen compressed at 120kPa are shown in Fig. 6. The deviator stress was defined by $q=(\sigma_1-\sigma_3)$.

In this result for the stress-strain relation, the deviator stress showed a clear maximum stress points, which then slowly deceased. As the value of δ increased from 0° to 90°, the maximum value of the deviator stress q_{max} also increased, but the axial strain at q_{max} decreased. In addition, the dilatancy rate at the point of failure increased with increasing δ. Fig. 7 shows the change in the deviator stress obtained for strains up to 0.05 % for the specimens the same relation as shown in Fig. 6.

Fig. 8 shows the relation between the Secant Young's modulus at 0.001% strain and the mean principal stress, p. The modulus increased to a linear relationship, having a slope of 0.519. Comparing the dependency for general soil(soil materials of no cementation), which is known to be 0.5 (for example, Hardin and Richart, 1963; Hardin and Black. 1969), the compacted material was found to have approximately the same dependency.

In addition, the δ had no influence on the dependency of Secant Young's modulus on p. Tatsuoka(1990) and Kohata(1995) reported similar results for sand. Therefore, the compacted soil also showed a small strain level(elastic deformation), which was not enough to affect the sedimentation angle. The Secant Young's modulus would only be related to the rigidity of a particle.

Fig. 9 shows the relationship between the secant shear resistance angle at peak shear stress (ϕ_{peak}) and δ.

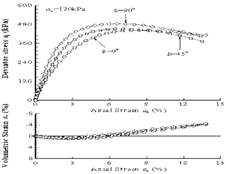

Fig 6. Stress-Strain Relations

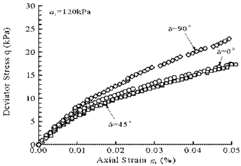

Fig 7. Stress-Strain Relations(ε_a=0.05%)

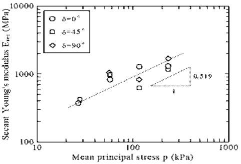

Fig 8. Relationship between Secant Young's modulus and Mean principal stress

The angle φ was calculated by the following equation.

$$\phi = \sin^{-1}\frac{(\sigma_1 - \sigma_3)}{(\sigma_1 + \sigma_3)} \qquad (1)$$

where σ_1 is the major principal stress and σ_3 is the minor principal stress.

As the value of δ increased from 0° to 90° the angle(ϕ_{peak}) also increased. The angle (ϕ_{peak}) of the 10 hours compressed specimens appeared to be approximately 2 degrees lower than that of the 1 hour compressed specimens. The degrees of saturation of the 1 hour and 10 hour compressed specimens were approximately 60% and 65%, respectively. Thus, the decrease in the strength over 10 hours would be related to the increase in saturation of the specimen, which also caused the decrease in suction.

In Fig. 9, the strength data on Ube decomposed granite soils prepared by the air pluviation method has been indicated(Nakata et al. 1998). The angle(ϕ_{peak}) of Ube appeared to be approximately 10 to 15 degrees lower than that of Shimonoseki. On the basis of this comparison, the influences of the sedimentation angle appear to be similar, irrespective of the sample preparation method.

Fig. 10 represents the effect of the sedimentation angle shown by Tatsuoka et al.(1990), who proposed the normalization of the shear resistance angle to that of the shear resistance angle 90° δ, ϕ(δ)/ϕ(δ=90°). The data for Toyoura sand, obtained by Oda et al. (1978) and Tatsuoka et al.(1990), are also shown in this figure. All specimens shown in Fig. 10 appear to have similar tendencies, in that the value of ϕ(δ)/ϕ(δ=90°) increased with increasing δ, although factors relating to the specimens, such as the initial density, the confining stress, the condition of sedimentation and preparation method, were different.

It was thought that the initial fabric arrangement would remain nearly constant in alow confining stress. As the confining stress increased, the particles were rearranged by particle breakage, such that the effect of the sedimentation angle disappeared under high confining stress.

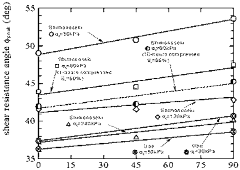

Fig 9. Relationship between shear resistance angles at peak shear and δ

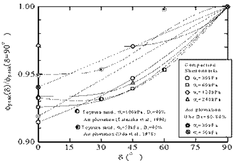

Fig 10. Relationship between ϕ (δ)/ ϕ(δ=90°) and δ

3.3 Discussion on initial fabric of compacted material

Lambe(1958) started a way of thinking about the structure of compacted soil, and although many researchers have tried to examine the fabric(structure) of compacted soil (Seed et al 1960) no consensus has been completely established. Attempts to observe the fabric arrangement of the specimens used in this paper using microscopy were made, but the fabric anisotropy could not be confirmed. Generally, the fabric anisotropy of saturated sand due to air-pluviation sedimentation has been known to develop in the direction of sedimentation. Onitsuka and Hayashi(1979) pointed out that the strength anisotropy of compacted sandy soils was caused by the direction of fabric arrangement, as the direction of the fabric arrangement was willing to orientate perpendicularly to the compaction direction.

4. Conclusions

A series of unsaturated-drained triaxial compression tests were performed on compacted materials. The tests were planned to find the effect of, not only the degree of anisotropy for sedimentation angle of compacted material, but also the influence of confining stress, degree of saturation and specimen preparation method on the anisotropic properties. The following conclusions can be drawn from this study:

(1) The compression strains of specimens were strongly influenced by the sedimentation angle.

(2) The Secant Young's modulus showed the same behaviour for all specimens, even with different δ.

(3) The effects of the sedimentation angle on the triaxial compression strength and deformation were clear of a low confining stress.

(4) The compacted specimen can be considered to have anisotropic mechanical properties, the same as the initial fabric anisotropy of sand.

References

1. Hardin, B.O. and Richart, F. E.Jr. 1963. Elastic wave velocities in granular soils, Journal of SMF Div., ASCE, Vol. 89, No. SM1: 3365.
2. Hardin, B.O. and Black, W. L. 1969. Vibration modulus of normally consolidated clay, Journal of the SMF Div., Proc. ASCE, Vol. 95, No. SM6: 15311537.
3. Kohata, 1995. Inherent and induced anisotropy of sedimentary softrock, Proc. Of 10ARC: 3336.
4. Livneh, M., Komornik, A. 1967. Anisotropic strength of compacted clay, Proc., 3rd ASIAN Reg. Conf. On SMFE, Vol.1: 298304.
5. Lambe, T. W. 1958. The structure of compacted clay, Proc., ASCE., Vol 84, SM2: 16551165535.
6. Nakata, Y., Hyodo, M. and Murata, H 1998. Single particle crushing and mechanical behaviour of decomposed granite soil, Proceedings of the international symposium on problematic soils., IS-TOHOKU'98. SENDAI, JAPA:497483.
7. Oda, M. 1981. Anisotropicstrength of cohesionless sands, Jour of Geotechnical Eng. Div., Proc. Of ASCE, Vol.107, No. GT9: 12191231.
8. Oda, M., Koishikawa, I., and Higuchi, T. 1978. Experimental study of anisotropic shear strength of sand by plane strain test, Soils and Foundation, Vol. 18, No.1: 2538.
9. Onitsuka, K., Hayashi, S., 1979. Studies on compression and strength Anisotropy of compacted soils, in Japanese. JSCE. Vol.19, No.3, Sept: 113123.
10. Seed, H.B., Mitchell, J.K. and Chan, C.K. 1960. The strength of compacted cohesive soils, ASCE., Research Conf. on the Shear Strength of Cohesive Soil, Boulder, Colorado: 169273.
11. Shibuya, S., Mitachi, T., Fukuda, F. and Degoshi, T. 1995. Strain rate effects on shear modulus and damping of normally consolidated clay, Geotechnical Testing Journal, Vol. 18, No. 3: 365375.
12. Tatsuoka, F., Nakamura, S., Huang, C. and Tani, K. 1990. Strength snisotropy and shear band direction, Soils and Foundation, Vol. 30, No.1: 3554.
13. Tatsuoka, F., 1994. Stiffness of hard soils and soft rocks in engineering applications, Pre-failure deformation of geomaterials, Vol. 2: 9471063.
14. Yong, R.N., Warkentin, B.P. 1966. Introduction to Soils Behavior: 106107.