Mechanical Properties of Decomposed Granite Soils and Their Relationships to Degree of Weathering

by

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The main object of this paper is to relate the mechanical behaviour of undisturbed samples of weathered granite soils to the degree of weathering. Four samples of decomposed granite soils, having different degrees of weathering, are treated in this paper.

In the former part, results of the drained triaxial compression tests on a less weathered sample, which is a typical example of decomposed granite being in a transition state between a rock and a soil, are presented and mechanical characteristics are discussed. This sample behaves as a typical elastic and strain-softening plastic material when subjected to the shear stress : The deviator stress decreses by 80% of the peak value to the residual ; as it softens, volume increases by as much as 10%.

In the latter part, effects of the weathering on some mechanical parameters are discussued by evaluating the degree of weathering by the initial void ratio e_0. The results of triaxial tests show that, with higher values of e_0, the effective angle of internal friction ϕ' decreases, shear modulus of elasticity G also decreases and the compressibility expressed by the compression index C_c increases.

Key words : Decomposed granite soils, Drained triaxial compression tests, Effective angle of internal friction, Shear modulus of elasticity, Compression index, Initial void ratio

1. Introduction

It is said that 13% of the country area of Japan is covered or consisted with granite rock mass or decomposed granite soils. Granite has played a very important role as a foundation rock mass or as a construction material. The importance will increase more and more as a number of large scale construction projects are being planned.

On the other hand, it is also the case that we have had many damages resulted from the failures of natural slopes consisted of decomposed granite soils. Decomposed granite soils have certain unknown properties from engineering view point although studies on their physical and mechanical characteristics of them have developed recently.

In the field of soil mechanics, decomposed granite soils are usually treated as one of special soils although the engineering use of them is not rare but very usual. As one of the reasons, we must cite the difficulty of sampling and testing their undisturbed samples. One of the special, if we are allowed to say so, characteristics different from other sedimentary soft rocks or soils may be the susceptibility to the destruction of their original internal structure or fabrics. Such problems as structure or fabrics must be concerned with the weathering.

To investigate the mechanism or the cause of the weathering is of course important, however, for soil engineers who must usually encounter the weathered state resulted from the process of weathering, it is more important and indispensable to investigate the mechanical or physical properties of weathered soils, and also to relate them to the degree of weathering. If we succeed in relating these two, the practical application of the studies on the mechanical characteristics will be possible.

A main object of this Paper is to relate the mechanical behaviour of undisturbed samples of weathered granite soils to the degree of weathering. Four kinds of samples are treated in this Paper; relationships of their mechanical parameters to the degree of the weathering are examined on the basis of the results from the drained triaxial tests performed on them.

Another object is to show the mechanical behaviour of a less weathered sample , being in a transition state between a rock and a soil. It appears to be a soft rock rather than a soil but can be easily pulverized as is the case with common decomposed garanite soils.

2. Samples

Samples of which mechanical behaviour will be discussed in this Paper were from three sites within the Chugoku district in Japan. In the following a sample name will be denoted by the site at which the sample was taken out.

Table 1: Physical properties of samples

Sample Name	TOTTORI	KAGAWA-B	KAGAWA-C	OKAYAMA
Specific Gravity G_s	2.65	2.64	2.68	2.67
Ignition Loss (%)	7.35	-	-	1.87
Initial Void Ratio e_0	0.751	0.515	0.387	0.19

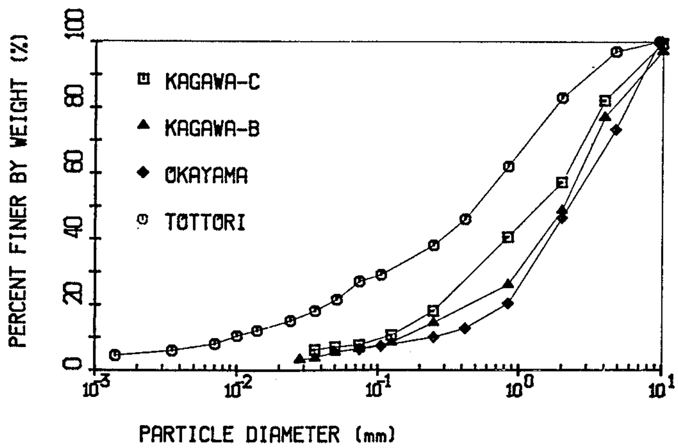


Fig.1: Grain size distribution curves.

Average values of some measures which can be effective in estimating the degree of weathering of the sample are presented in Table 1. The grain size distribution curves are shown in Fig.1. As can be seen from the table and the figure the degree of weathering is different between the samples: the Tottori sample is an extremely weathered one; and the Kagawa - B and -C are well weathered but less than the Okayama. Especially, the Okayama sample has a very low value of only 0.19 of the initial void ratio; it appears to be a soft rock rather than a soil, but can be easily disturbed, as is the case with commonly encountered decomposed granite soils.

For the detail of the method for preparing undisturbed specimens of the Tottori and Kagawa samples, a previous paper[1] should be referred to. The method applied to the Okayama sample will be described briefly in the following.

The Okayama sample was taken from a slope at a borrow-pit in Wake-Cho, Okayama prefecture. Naturally made cubical blocks of about 30x30x30 (cm) were brought to the laboratory; the blocks were frozen in a electric freezer. They were further frozen by the liquified nitrogen just before the coring. The frozen blocks were bored by the use of a boring machine with a special core bit. The core bit used for this sample has the same shape as that of the core bit originated in Ehime University[2]. Cylindrical specimens of 5 cm in diameter and 10 cm in height were made and served for triaxial tests.

3. Mechanical Behaviour of a Less Weathered Granite Soil with High Density

3.1 Testing Procedure

Mechanical behaviour of only the Okayama sample will be presented here because those of other samples were already discussed elsewhere[1],[3].

Isotropic cconsolidation drained triaxial compression tests (CID) were performed. Specimens were left in the triaxial cell for more than 12 hours to be melted before the consolidation. The isotropic consolidation pressure were varied ranging from 40 to 300 kPa. After the consolidation the strain-controlled triaxial compression was performed; the axial deformation rate was 1.5×10^{-3} cm/min.

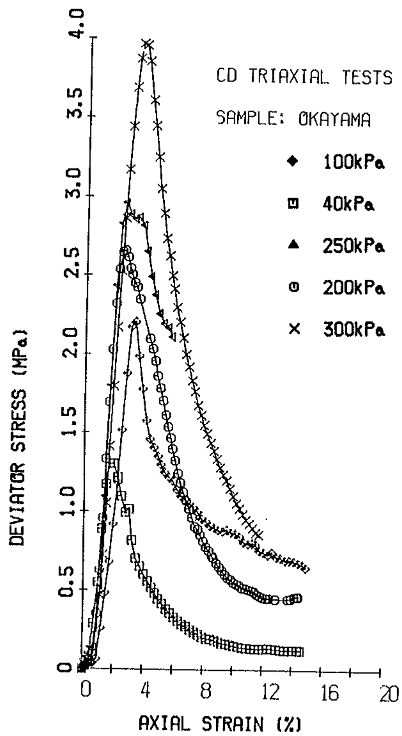
3.2 Stress Strain Behaviour

Fig.2 represents the relationships between the deviator stress and the axial strain. As can be seen in the figure, for any confining stress the deviator stress, increases to a peak value with a high linearity and drastically decreases after the peak. The brittleness index proposed by Bishop[4] that expresses the degree of the softening is 90-80% for this sample.

In Fig.3 the relationships between the volumetric strain and the axial strain are shown. For any confining stress the volume of a specimen decreases in the beginning of the deformation and subsequently increses due to the dilatancy as the deformation develops. The change in the volumetric strain attenuates when the deformaetion develops to the residual state. It should be noted here that the degree of the dilation is larger for the less confining stress.

Fig.4 represents the variation of the void ratio e during the shearing deformation with the effective mean normal stress p. The void ratio e does not change in the beginning of the deformation and there is found strong linearity in the relationship. In the stage where the linearity is reserved the dilatancy does not yet occur; just before p reaches the peak value, e begins to increase due to the positive dilatancy; and the positive dilatant behaviour becomes more extensive after the softenig occurs.

The slope of the linear portin observed in the beginning part where no dilatancy yet occurs corresponds to the compression index C_c; the avarage value of 6.05x10-3 was obtained for this sample.



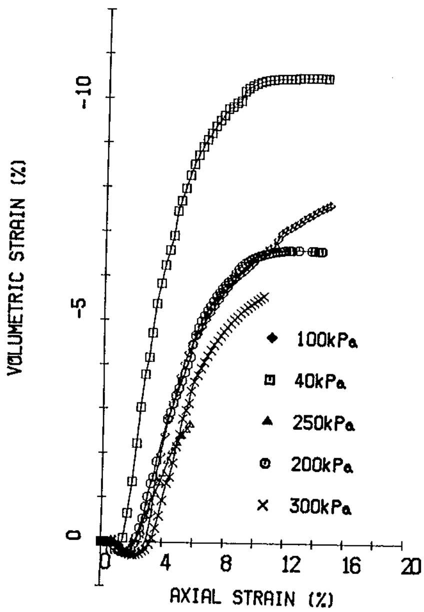


Fig.2: Deviator Stress vs. axial strain (OKAYAMA sample)

Fig.3: Volumetric strain vs. axial strain (OKAYAMA sample)

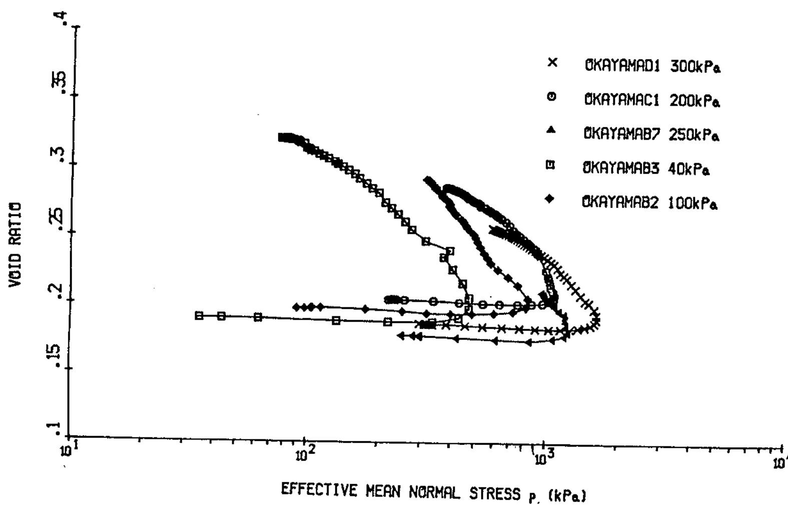


Fig.4: Variation of void ratio with log of effective mean normal stress

3.3 Strength Behaviour

In Fig.5 peak and residual values of the deviator stress are plotted versus confining stress, respectively. In this figure the results of the tests in which the lateral stress was not held constant are also included because the effective stress parameters at failure will not be affected by stress paths. There is scatter for the resudual values, since all the specimens did not reach the complete residual state as is shown in Fig.2.

Two straight lines drawn in the figure, one for the peak and one the residual state, were obtained by the use of the least square method. According to the relationships expressed by these two lines, strength parameters in terms of effective stresses can be obtained. The values of the parameters, ϕ' and c', are given in the figure. c' and ϕ' decreases by about 150 kPa and 23°, respectively, from the peak to the residual state. Thus the decrease in values of strength parameters is extraordinary.

We find that the value of ϕ' at the residual state is nearly the same as the peak value of ϕ' of the Tottori sample (see Fig.8); this implies that the decrease in strength due to the weathering is well comparable to its decrease caused by the shear deformation. The weathering is a long term effect, whereas shear deformation in the laboratory is a short term effect. The strength decrease due to the weathering will be discussed later.

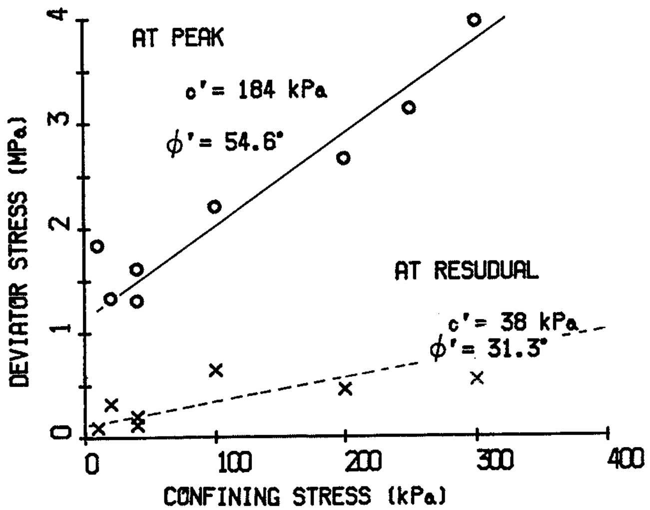


Fig.5: Effective stress states at the peak and residual

3.4 Elastic properties

By defining the elastic state as the state under which the stress-strain relationship is linear and dilatancy does not occur, two elasticity parameters can be determined, the shear modulus G and the bulk modulus K. Values of G were determined as the slope of the initial linear portion of the stress-strain relationships shown in Fig.2 ; those of K were determined by the following expression:

$$K=\ln 10 . |(1+e)/C_c| . p \qquad (1)$$

where C_c is the slope of the linear portion of an e-log p relation shown in Fig.4.

In Fig.6 G is plotted versus the confining stress; and in Fig.7 K is plotted versus the mean effective normal stress p. It can be seen from these figures that G increases linearly with the confining stress and K also increases linearly with the effective mean normal stress.

3.5 Summary of the stress-strain behaviour

The sample Okayama, which has not heavily been weathered, behaves as a typical elastic and strain-softening plastic material when subjected to the shear stress; deviator stress decreases by 80% from its peak to the residual. As it softens, it dilates and the volume increases by as much as 10%.

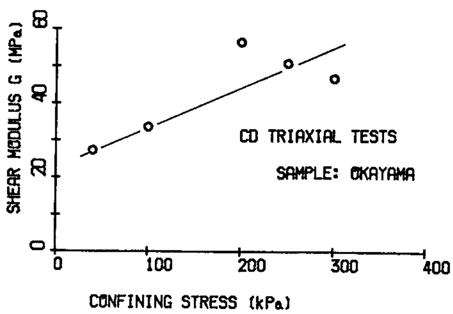


Fig.6:Shear modulus of elasticity and the confinig stress.

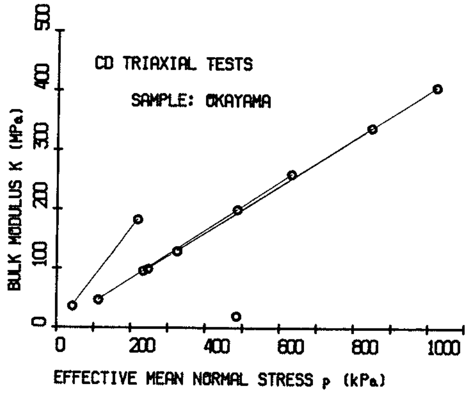


Fig.7: Bulk mudulus of elasticity and effective mean normal stress

4. Degree of weathering and the mechanical characteristics

4.1 Measure of the degree of weathering

Effects of the degree of weathering on the mechanical characteristics will be examined on four samples shown in Table 1. We have some measures that can express the degree of the weathering of granite: for example, the Ignition Loss, X-ray diffraction intensity, specific gravity of feldspar, apparent specific gravity, in-situ porosity and so on. In this article , the initial void ratio ,i.e., the void ratio that the sample possessed in the field, will be adopted as a measure ; for the practical use this void ratio may be replaced by the value of the void ratio when the sample is taken to the laboratory without disturbing.

4.2 Loss of strength due to the weathering

In Fig.8 the peak value of the effective angle of internal friction ϕ' is plotted against the initial void ratio. ϕ' was determined for the range of the confining stress less than 1 MPa ; with a different range of the confining stress the value of ϕ' would be different somewhat from that shown in the figure. It can be seen in the figure that ϕ' tends to decrease with the increase in the initial void ratio.

This tendency is similar to the case of sands[5] as to sands the relationship of ϕ' to the initial void ratio depends on the grain size and the shape of particles[6]. In the case of decomposed granite soils, the ϕ'-e_0 relation , like that shown in Fig.8, might depend on the type of their original granite rocks.

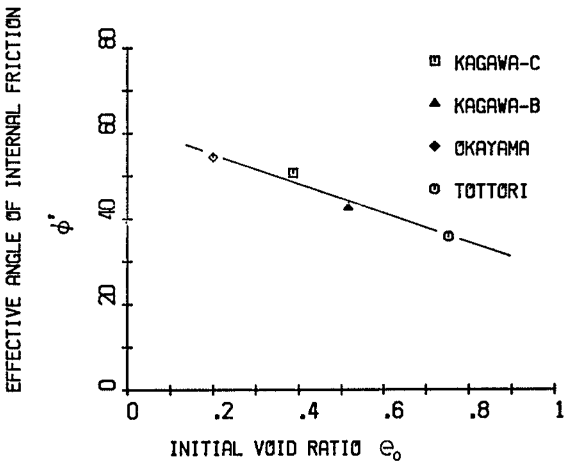


Fig.8: Relationship between effective angle of friction and void ratio

4.3 Effect of the weathering on the shear modulus of elasticity

Shear modulus of elasticity G is plotted agaist the initial void ratio in Fig.9. In the figure the modulus is normalized by the confining stress; in general the relationship between G and the confining stress is not a straight line passing the origin, for example, as already observed in Fig.6; therefore, even for a certain value of the initial void ratio the normalized shear modulus is not constant but varies within some range. This range is limitted for a sample. It is seen from Fig.9 that G decreases exponentially with the initial void ratio e_0 for a given confining stress.

4.4 Effects of the weathering on the compressibility

On the basis of the results from a triaxial compression test during which the confining stress is held constant, the compression index C_c, which is defined as the compressibility under the all round stress state, can be determined in the way described below.

So long as the shear deformation is small, the dilatancy does not occur and the e-log p relation is to be linear; the slope of the line can be considered as the compression index C_c, because the change in e does not include the change due to the deviatoric component of stress.

C_c determined by such a way is plotted against the initial void ratio e_0 in Fig.10. This figure shows, although some scatter is found, that, as e_0 increses, C_c increases and tends to reach a limitted value of about 0.2.

Furthermore, if the linear portion of the e-log p relation is also elastic, as was the case with the sample Okayama, the bulk mudulus of elasticity K can be determined from C_c using eq.(1); therefore, the relation as shown in Fig.10 will be usefull for estimating K.

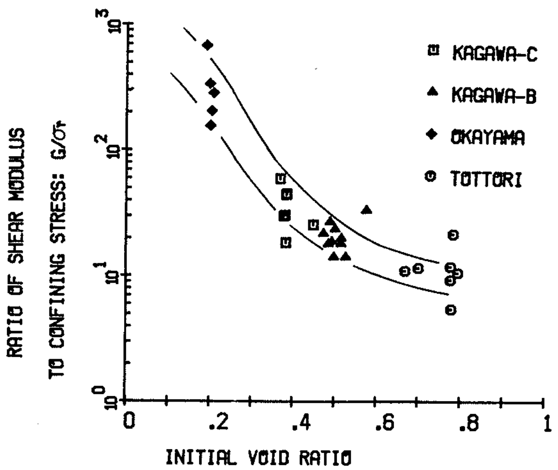


Fig.9: Normalized shear modulus and initial void ratio

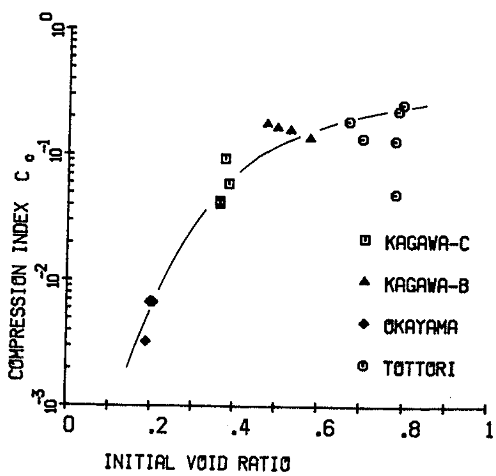


Fig.10: Relationship between compression index and initial void ratio

5. Conclusions

Four samples of decomposed granite soils, having different degrees of weathering, from three sites within the Chugoku district in Japan are treated in this paper.

In the former part of the Paper, results of the drained triaxial compression tests on less weathered sample was presented and mechanical behaviour was discussed. This sample behaves as a typical elastic and strain-softening plastic material when subjected to the shear stress; the deviator stress decreases by 80% from the peak to the residual. As it softens, it dilates and the volume increases by as much as 10%.

In the latter part of this paper, effects of the weathering on the mechanical behaviour was examined on the basis of the results from drained triaxial compression tests performed on the four samples. The degree of weathering was evaluated by the initial void ratio e_0.

The effective angle of internal friction ϕ' decreases with the increase in e_0. The obtained relationship between ϕ' and e_0 is similar one as that obtained for sands. The residual value of ϕ' of the sample having the lowest value of e_0, 0.19 was comparable to the peak value of ϕ' for the sample having the highest value of e_0,0.75.

The shear modulus of elasticity G decreases exponentially with the initial void ratio for a given confining stress.

The compressibility, evaluated in terms of the compression index C_c, increses withe the increase of the initial void ratio. In certain cases, C_c can be related to the bulk modulus of elasticity K by an expression (1), therefore the C_c -e_0 relation obtained here will be useful for estimating K.

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