GEOTECHNICAL FEATURES OF SOILS AND ROCKS RELATED TO LANDSLIDES OF TERTIARY FRACTURE ZONE TYPE IN JAPAN

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SUMMARY

Index properties and shearing strength of soils and mudstone taken from several landslides of Tertiary fractured region were investigated. The results showed that the sliding soils, often referred to as "blue-black clays", have similar properties, and that the sliding diatomaceous earth has properties considerably different from the "blue-black clays". The bed rock of sliding areas, being mudstone in most cases, shows the phenomenon of slaking. The features of slip planes were found in some of the test pits burrowed in sliding areas.

INTRODUCTION

Investigations have been carried out by the authors at several landslide districts in Japan. Typical examples of landslides of Tertiary fractured region are given below (Table 1 and Fig. 1). Two of these landslides *A* and *D* are discussed here in detail, with some reference to the other two, to give a generalized idea on the geotechnical features of soils and rocks related to the landslides of this type.

GENERAL EXPLANATIONS OF DISTRICTS

District A (Fig. 2): The movement was relatively slow and continuous; it was conspicuous in spring and autumn. The movement of sliding was large at the upper portion of the slope along the valley, and it reached over 10 meters a year. Almost no sliding occurs on hill sides. The slide was investigated by boring and test pits. The boreholes B4 and B5 were damaged at the depth of 6 to 7 meters while drilling.

District D (Fig. 3): The movement was relatively rapid and local. Large sliding occurred several times since 1854. The slide was investigated by boring, sounding and test pit.

FEATURES OF SOIL LAYERS

(1) *Test pits* were burrowed at sliding areas for investigating the sequence of soil layers to the depth of a few meters below ground surface and for obtaining undisturbed samples. The locations of pits are shown in Figs. 2 and 3, the observations in the test pit in Figs. 4 and 5, and the field moisture content in Fig. 6.

District A

It is worth noticing that slip planes were observed in test pits P'_1 and P_4 . Two

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Table 1. Localities

District	Name	Geology	Soil
A	Wademura, Otari-mura, Nagano Prefecture.	Pliocene (sandstone and mudstone)	Blue-black clay
B	Kamiyashiro, Kanaya-machi, Shizuoka Pref.	Paleogene (siltstone and mudstone)	Blue-black clay
C	Imajuku, Yui-machi, Shizuoka Pref.	Pliocene (mudstone)	Blue-black clay
D	Kamizakai, Iiyama-shi, Nagano Pref.	Pliocene (diatomaceous sandstone and mudstone)	Diatomaceous earth

Fig. 1. Landslide Districts

Table 2. General Explanations of Districts

District	Area	Mean gradient	Type of movement	History of slide
A	50 ha	About 13°	Slow, continuous; conspicuous in spring and fall	1902, 1959
B	40 ha	About 10°	Slow, continuous	
C	5 ha	About 15°	Continuous, conspicuous in spring and fall	Since 1934
D	20 ha	About 15°	Relatively rapid, with local failures	Since 1854, 1928, 1935, 1960

Fig. 2. Plan of District A

anticlinal slip planes were found in P'_1 , and many planes of different features in P'_4 . All of these planes showed the feature, like slickensides, and the soil were easily teared off at the plane.

District B

Slip planes similar to above were found in a test pit at District B.

District D

Microscopic observation of surface soils at District D proved the existence of diatoms in this area.

(2) *Boring cores*

Fig. 3. Plan of District D

Fig. 4. Soil Profile of Test Pit P₁ District A

District A

Twelve boreholes were drilled to the depth of 25 m (50 m for B₃) below ground surface. It was shown that the subsoil could be divided into four layers as following.

1st layer: 0 to 5 m; soft clay observed in test pits;

2nd layer: 5 to 10 m; clay stiffer than the 1st layer, slide occurring at the depth of 6 to 7 m;

3rd layer: 10 to 20 m; stiff clay and mudstone;

4th layer: below 20 m deep; mudstone with local cracks.

District D

Three boreholes were drilled to the depth of 35 m (50 m for B₂). Their cores were mostly of diatomaceous mudstone containing sand and gravel layers. The results of standard penetration test are shown in Fig. 7.

INDEX PROPERTIES OF SOILS

(1) Materials and method

Index properties had been performed according to JIS standard. Unit weight and solid and air contents were measured by the samples prepared for shearing tests. Samples were obtained from undisturbed soil blocks in test pits.

Fig. 5. Soil Profile of Test Pit P₁ District D

Fig. 6. Field Moisture Content

(2) Test Results

The detailed results are shown in Figs 8–11 and Table 3. As would easily be observed, the diatomaceous earth from district D had physical properties considerably different from the “blue-black clays” from the other districts.

Fig. 7. Number of Blows of Standard Penetration Test and Unconfined Compression Strength, District D

Fig. 8. Solid and Air Contents of Undisturbed Samples

Fig. 9. Unit Weight and Dry Unit Weight

PERMEABILITY TEST

(1) Materials and method

Cylindrical samples were used in permeability test, 5 cm in diameter and 5 cm in height. They were prepared from undisturbed soil blocks obtained from test pits. Falling head permeability test was performed on these samples.

Fig. 10. Plasticity Chart

Fig. 11. Soil Classification by Triangular Coordinates

Table 3. Properties of Soils

District	Pit	Depth of sampling (m)	P.L. %	L.L. %	Colloidal activity	Specific gravity of soil
A	P_1	2.7—3.0	17	39	2.6—3.4	2.72 ± 0.01
		0.8—1.0	18	32	1.8—1.9	2.71 ± 0.05
B	P_5	1.6—1.8	16	45	1.3	2.70 ± 0.06
		3.0—3.2	18	45	1.3—1.5	2.73 ± 0.07
C	12	surface	18	37	2.4—2.9	2.74 ± 0.01
	75	surface	17	38	2.1—2.6	2.74 ± 0.01
D	P_1	0.7—1.0	42	79	—	2.72 ± 0.00
		2.6—2.9	58	115	—	2.60 ± 0.04
		4.7—5.0	52	105	—	2.61 ± 0.04

(2) Results

Though the values of the coefficient of permeability considerably scattered, it showed the tendency that permeability decreases by remolding. The range of scattering is shown in Fig. 12. Considering the effect of cracks in soils, larger scale tests in situ would be desirable. So that in situ determination of permeability would be preferable when the effect of cracks concerned.

Fig. 12. Coefficient of Permeability

SHEARING TEST

(1) Materials and method

Undrained quick triaxial compression test and unconfined compression test were performed on cylindrical soil samples, 5 cm in diameter and 12 cm in height. The strain was controlled at about 0.5% per minute. A creep test was conducted with a miniature vane tester on soil samples from district A. All materials were obtained from undisturbed samples taken out from test pits.

(2) Results

Strength is expressed by $(\sigma_1 - \sigma_3)/2$ for triaxial test and by $q_u/2$ for unconfined test (Fig. 13). Test results were considerably scattered, and no practical difference was revealed between the results of these two tests.

Many of the samples from District D were too soft for trimming, so that the tests were performed on stiff ones. Therefore, the shear strength must actually ranges from 0 to τ_{max} .

The "blue-black clays" have conspicuous elasto-plastic viscous characters of deformation, as shown by vane test results (Fig. 14). They seem to obey the rheological equation established by Vialov¹⁾, though with some deviation due to the heterogeneity of soils.

PROPERTIES OF BED ROCK

(1) Mechanical properties

Undisturbed rock samples obtained from the beds exposed at the bottom of gullies at district C and D were tested. The samples were trimmed into square prisms (district C) or cylinders (district D). Compression test was conducted on some of these samples. The others were air-dried and unconfined compression test was made on the samples when no visible cracks observed after drying. The results are shown in Figs. 15 and 16.

Fig. 13. Shearing Strength of Undisturbed Soil Samples

Fig. 14. Creep Curves for Undisturbed Soil Samples (2 m deep, Pit P₄, District A)

Fig. 15. Stress-Strain Curves of Mudstone, District C

Fig. 16. Stress-Strain Curves of Diatomaceous Mudstone, District D

(2) Index properties

Some examples of the results of physical tests are already shown in Figs. 8 and 9.

(3) Slaking test

In analyzing the long-term stability of slopes, the destructive effect of water on mudstone must be considered. For this purpose, slaking tests were carried out on mudstone.

Mudstone samples taken from the bottom of gullies at districts C and D and those from the boring cores at district A were tested. Samples air-dried sufficiently were immersed in water and, after the completion of disintegration by slaking, the produced small particles or flakes were sieved in water to investigate the grain size distribution. The particles or flakes were again air-dried and the same procedures were repeated for the samples from district C and D.

As shown in Figs 17 and 18, all the samples were easily disintegrated when immersed in water after being air-dried. The same effect was observed even in the cores taken from the depth of 25 to 50 meters at district A.

Fig. 17. Distintegration of Dried Mudstone after Immersion in Water

Fig. 18. Results of Slaking Test (Borehole B₃, District A)

In order to find the nature of disintegration due to slaking, tests were carried out with several liquids other than water. Table 4 shows the destructive effect of liquids and the values of dielectric constant as has already been pointed out by Denisov and Reltov²⁾. The results show that the destructive effect of liquids is not governed only by dielectric constant but some other factors though it will be undoubtedly the dominant factor.

Table 4. Disintegrating Effect of Liquids on Air-dried Mudstone

Liquid	Surface tension when in contact with air ³⁾ dyne per sq cm (20°C)	Dielectric constant ³⁾ c. g. s. e. s. u. unit	Disintegrating effect*
Water	72.8	81.0	⊙
Ethylene glycol	47.7	41.2	○
Nitrobenzene	43.9	35.7	—
Ethyl alcohol	22.3	25.7	○
Acetone	23.7	21.0	⊙
o-Toluidine	40.0	6.4	—
Xylene	m. 28.9	m. 2.4	—
	p. 28.4	p. 2.3	
	o. 30.1	o. 2.6	
Carbon tetrachloride	26.8	2.2	—
Benzene	28.9	2.3	—

* Intensity of distintegrating effect: ⊙ > ⊙ > ○ > —.

ANALYSIS OF STABILITY

(1) Calculation of safety factor

As previously mentioned, the observation in the test pits at district A proved the existence of several slip planes not being a single one. Their shape was rather irregular and complex. For the slides of the other districts, the number and shape of slip planes could not be observed. Therefore, no appropriate methods are available, at the present stage of investigations, for calculating the factor of safety of slopes. As a rough approximation, however, the factor was calculated by the following formula, assuming the slip surface to lie parallel to the ground surface.

$$F_s = \frac{2}{w_o \sin 2\theta} \cdot \frac{C}{h}$$

where w_o = unit weight of soil; $C = q_u/2$ or $(\sigma_1 - \sigma_3)/2$;
 h = depth of slip surface; θ = average gradient of slope.

(2) Results and consideration

An example of the relationships between F_s , h and C is shown in Fig. 19. The shaded area shows the unstable range where $F_s \leq 1$. A similar consideration was made for district D. The safety factor was calculated from the data shown in Fig. 13. and the soils listed in Table 5 fall in the unstable range.

Fig. 19. Relationship between F_s and h

Table 5. Soils in the Range of Danger

District	Test pit	Depth of sampling
A	P_4	1.5—1.8 m
		1.9—2.2
		2.3—2.6
D	P_1	4.7—5.0

The slope for the test pit P' at district A is considered to be stable, this site is in critical equilibrium. However, if the strength of the soil 10 meters deep is the same as that at the depth of 4 meters, the safety factor would fall in the unstable range. As for district D, considerations must be made on the fact that the shearing strength ranges from 0 to $\tau_{\max}$ as mentioned before. Thus, the actual slope should be unstable by the nature of this shearing strength.

In addition, due to the locally steeper and irregular slopes, the actual safety factor of slope is to be taken smaller than the calculated one.

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