Ⅲ-27 A step toward studying creep displacement of landslides as a process influenced by expansive clay minerals

愛媛大学工学部 (正) ○Netra P. Bhandary
愛媛大学工学部 (正) 矢田部 龍一
愛媛大学工学部 (正) 横田 公忠

1 INTRODUCTION

Proper stability analyses of landslides require an in-depth knowledge of the landslide mechanisms, which not only depend on the soil strength parameters but also on various other factors such as mineralogical changes in the slip surface material. One frequently reported fact about creeping landslides especially in Shikoku is that they consist of clayey soils in their slip surfaces that range in thickness from 10 to 20cm. The displacement characteristics of landslides are considered controlled entirely by the strength characteristics of the slip surface material, which at most landslide sites in Shikoku is found composed of weaker clay minerals such as chlorite, smectite, vermiculite, and illite. Various investigations have shown that soils exhibit lower values of strength parameters with increasing percentage of expansive clay minerals like smectite and vermiculite. Therefore, it is important to study the strength properties of soils composed of expansive clay minerals so as to understand the mechanism of landslide activation having creeping displacement.

2 CONCEPT OF LANDSLIDE CREEP

The displacement of a landslide takes place due mostly to rise in groundwater level. A risen groundwater level following a rainfall exerts an increased pore-water pressure resulting in reduced effective stress thereby reducing the shear strength of the slip surface soil.

Figure 1(a) explains the creep failure of a soil by the help of Mohr's stress circle. The authors make a hypothetical consideration that every soil has a zone of creep just below the strength envelope with a lower bound equivalent to 90-95% of the angle of shearing resistance, as shown in the figure, which widens with increase in the effective stress. This implies that the range of creep for deeper landslides is wider than that for shallower ones. In other words, when a landslide has a deeper slip surface, the normal stress will be higher and the range of normal stress required to cause creep failure will be wider, as indicated in the figure. Moreover, the range of creep for weaker soils is wider than that for the stronger ones, as illustrated in Figure 1(b). It is because the range becomes wider with a decreased inclination of the strength envelope. For instance, the decomposition of rock minerals into clay minerals in the slip surface soil results in clockwise rotation of strength envelope with the reduction in angle of shearing resistance for the soil and comes closer to the circle representing stresses in stable condition such that a slight reduction in effective stress owing largely to the rise in pore water pressure results in creep failure as soon as the stress circle shifts toward left and enters the creep zone.

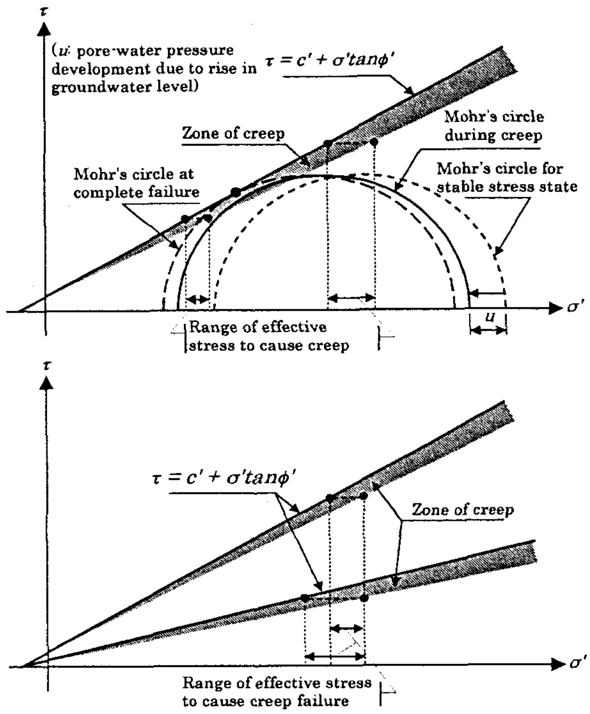

Figure 1: (a) Creep zone for soils and creep failure explained by Mohr's stress circle; (b) Range of creep failure for two different soils

3 TESTS AND RESULTS

The test program basically involved part of the attempts to study strength characteristics of soil heavily composed of montmorillonite. The physical properties of samples (i.e., bentonite and crushed Toyoura sand) used in the experiments are given in Table 1. The tests were conducted in two parts, namely simple ring shear tests on sand-bentonite mixes and consolidation-swell test on bentonite.

The sand-bentonite samples were prepared at 90/10, 80/20, and 70/30 mix ratios by simply dry-mixing them. The mixed dry samples were then placed in the ring shear apparatus such that the samples would attain a void ratio of 2 at a specimen thickness of 15mm. The samples were then wetted by passing water through the sample bottom. When traces of water permeated through the samples were seen on the top, the water was also passed from the top. Before wetting, each sample was consolidated under a pressure of 9.81kPa. After 48 hours of continuous wetting, each sample was consolidated under a desired normal stress of the test conditions. One mix sample was tested under the normal stress of 98.1, 196.2, and 294.3kPa, and each sample was sheared for 15 hours by which the shear displacement would be equivalent to one rotation of the ring shear apparatus.

In the second part, attempts were made to see how bentonite sample actually behaves during consolidation and swelling. As seen in Table 1, the liquid limit for pure bentonite is in a range of nearly 7-10 times that of ordinary clay soils and it has a free swell value above 1200% at 430% water content, in ordinary conditions it was difficult to carry out consolidation test on a fully saturated sample of bentonite because of the restriction of the depth of oedometer ring. To overcome this problem and to have a relative idea of consolidation behavior, dry sample was first put under a pressure of 9.81kPa and then allowed to swell until fully swollen volume was attained by passing water first from the bottom and then from top and bottom after traces of water permeating upward were seen. Due to depth restriction of the consolidation ring, the initial thickness of dry specimen was kept 10mm with a void ratio of 2, which was ascertained to be a state looser than that obtained by applying a pressure of 9.81kPa. The second purpose of this test was to determine the swelling pressure of bentonite,

Table 1: Properties of soil samples tested

Sample	ρ_s (g/cm^3)	LL (%)	PL (%)	I_p	Grain size distribution (%) <5μm	5~75μm	>75μm	Free swell value (%)
Na-bentonite	2.67	460	28	432	77	23	0	1215 at w=430%
Crushed sand	2.66	-	-	-	25	75	0	-

膨張性粘土鉱物の影響を受ける地すべりのクリプ現象に関する研究の一歩

which was based on consolidation-swell test (Nelson & Miller, 1992). The average length of time required for one stage of consolidation was 3 days, and that required during single stage swelling was one week.

The strength parameters obtained from the ring shear tests are summarized in Figure 2. It clearly shows that the strength of the mix decreases with the increased amount of bentonite. If the obtained results bear high accuracy, it can be inferred that the strength of a soil in residual state drops by nearly 50% even at a montmorillonite content of as low as 15% (i.e. 30% bentonite). The peak strength is also seen to have reduced by nearly 40% at this state of mix ratio. Therefore, it can be said that the creep range for a soil increases with the increase in expansive clay mineral content since the strength envelope rotates clockwise with a decreased angle of friction.

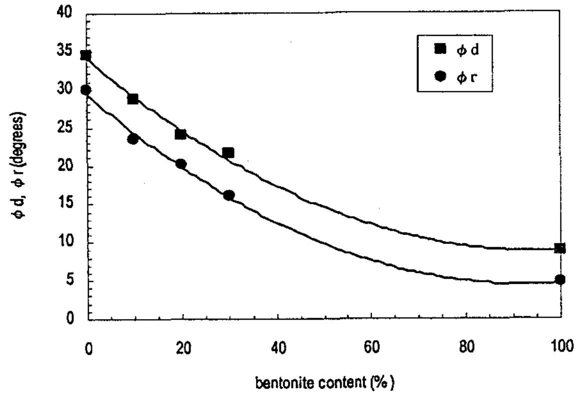

Figure 2: Summarized results of ring shear tests on sand-bentonite mix samples

The results obtained from consolidation test on unsaturated bentonite sample are plotted in Figure 3. It is seen that with an initial void ratio of 2 and thickness of 10mm, the bentonite could be wetted until the water content reached as high as 188%, at which the degree of saturation was calculated to be 80%. The consolidation of the swollen sample showed that the stress required to compress it back to the initial volume is 380kPa, which according to Nelson & Miller (1992) is the swelling pressure for the tested sample. The consolidation at 1255kPa resulted in a drop of water content up to a level as low as 15% and the void ratio dropped to nearly 0.5, which is a significant drop compared with that for ordinary clay soils. Here, it is worth mentioning the state of water in expansive soils. An unsaturated soil consists of three forms of water, which are free water (or gravity water), meniscus water, and structural water. Most unsaturated soils contain first two types of water but most expansive soils consist additionally of structural water. Structural water in soils is generally considered to leave the soil upon heating, but consolidation test on bentonite shows that it can largely be removed from montmorillonite minerals by applying pressure. This is what is supposed to make expansive soils behave differently during shear. The rebound curve in the figure shows that the tested bentonite sample could achieve a swollen state nearly the same as the fully swollen sate. It is considered that if the water could be passed uniformly through the specimen, the volume after the release of stress would have equaled the initially swollen volume.

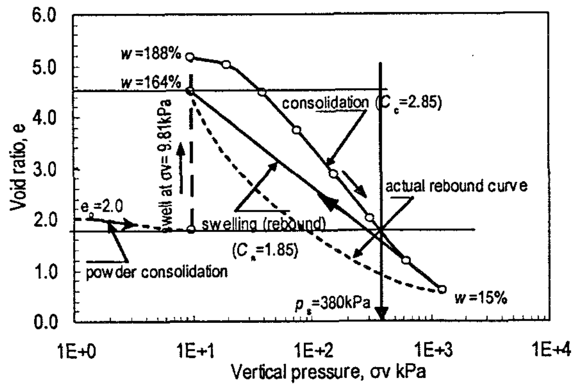

Figure 3: Results of consolidation-swell test on bentonite using an oedometer

The notably small angles of shearing resistance for bentonite are considered to be entirely due to presence of montmorillonite, which is considered to lose the state of solidness upon wetting beyond a certain limit. The Soil Mechanics assumes all the soil particles to be solid and incompressible, and the strength theories are all based on this assumption. Due to high amount of water in montmorillonite particles, however, are considered to exhibit compressible behavior, which may greatly influence the shear characteristics.

4 CONCLUSIONS

Following conclusions are drawn from the results obtained from the tests conducted in this study.

- ✧ Even a 15% expansive clay mineral content in crushed Toyoura sand sample resulted in nearly 50% drop in residual and 40% drop in peak strength values.
- ✧ Increased expansive clay mineral content is considered to cause wider range for creep failure by reducing the inclination of strength envelope.
- ✧ Despite having a free swell value above 1200%, the bentonite used in the in the tests could swell only 615% with 188% of water content when wetted under a pressure of as low as 9.81kPa.
- ✧ As per the consolidation-swell test, the bentonite used in the tests was found to have a swelling pressure of 380kPa, which means it has a swelling potential of 380kPa when confined by an external pressure above 380kPa.
- ✧ The application of 1255kPa of pressure on swollen bentonite could reduce the water content to as low as 15% and the void ratio as low as 0.5. It implies that the structural water in montmorillonite can also be squeezed out.
- ✧ The water content after full rebound reached as close as the value at initial swelling. It is considered that if the water could be passed into the specimen uniformly, the water content could reach the initial value.

Finally, as it was just a step toward studying strength behavior of expansive clay soils during creep failure, the paper lacks enough data to support its hypothetical points. It is expected that the results from creep tests planned in future would clarify the creep mechanism of large-scale landslides.

REFERENCES

Ministry of Land, Infrastructure and Transport, 1997. *Landslides in Japan*, A booklet published by Sabo Publicity Center, Slope Conservation Division.

Nelson, J.D. & Miller, D.J., 1992. *Expansive Soils: Problems and Practice in Foundation and Pavement Engineering*, John Wiley and Sons, Inc., pp.81-87.

Terzaghi, K., 1952. *Mechanisms of Landslides*. Geol. Soc. Am., Berkey Volume, pp.83-123.

Yagi, N., Enoki, M., and Yatabe, R., 1990. Consideration on mechanical characteristics of landslide clay. *Landslides* (Ed. C. Bonnard), *Proc. the Fifth Int. Symp. On Landslides*, 10-15 Jul. 1988, Vol. 1, pp.361-364.