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RING SHEAR TEST ON EXPANSIVE CLAY IN RELATION TO ITS ROLE IN CAUSING CREEP ACTIVATION OF LANDSLIDES

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1 INTRODUCTION

Shikoku has a fragile geology mainly because the tectonic faults such as median tectonic line, Mikabu tectonic line, and Butsuzo tectonic line pass across the island. Investigations have shown that the rock masses near the faults are heavily fractured. This has led most rock masses to undergo in-depth chemical weathering, which as a major preparing factor for the slope failures has given rise to landslide activation under considerably slow rate of displacement. The Ministry of Land, Infrastructure, and Transport (1997) reports that the number of designated landside-threatened areas in Shikoku under the direct jurisdiction of the ministry is 670, which is nearly 21% of the number in whole country. Moreover, the total area of these landslides measures about 310km^2, which is nearly 30% of the total landslide area in the country (total landslide area in Japan is about 1115km^2). Nearly all these landslides in Shikoku have occurred within a strip of width 20 to 50km from median tectonic line to Butsuzo tectonic line. More specifically, most landslides have occurred on Sambagawa and Mikabu belts, where dominant distribution of metamorphic rocks composed primarily of greenstone and green schist can be found.

One frequently reported and widely accepted fact about large-scale creeping landslides in Shikoku is that they slip through clayey soil layers of thickness from 10 to 20cm (Yagi et al., 1990; Yatabe et al., 1991a, 1991b). The displacement behavior of these landslides is considered controlled largely by the strength characteristics of the slip layer material, which at most landslide sites in Shikoku is found composed of weaker clay minerals such as expansive chlorite, smectite, vermiculite, and illite (Yokota et al., 1995; Yagi et al., 1999). It is therefore important to investigate the strength properties of soils composed of expansive clay minerals so as to understand the mechanism of landslide activation in Shikoku, which mostly have creeping displacements. Improper interpretation of landslide mechanism may result in unsafe stability analysis, and hence in the risk of unexpected failure in terms of accelerated displacement. In addition to change in underground water conditions, mineralogical changes in the slip layer material greatly influence its strength parameters causing increase or decrease of the displacement rate.

This study aims at briefly explaining the displacement mechanism of landslides on the fault zones of Shikoku and then at carrying out a preliminary study on the strength properties of smectite-rich clay soil with the help of simple ring shear apparatus. Although it may not be easy to fully understand the strength behavior of landslide clays composed of expansive clay minerals, it is expected that certain lab investigations of the strength behavior of bentonite-like clays will reveal the relative strength properties of expansive clays.

2 LANDSLIDE MECHANISM IN SHIKOKU

According to Terzaghi, (1952) the landslide-causing factors can be broadly put into three groups: preparatory, activating, and sustaining. The preparatory factors prepare a soil slope for failure, the activating factors induce the failure, and the sustaining factors keep the failed slope in motion either continuously or intermittently, often resulting in creeping displacements. Rock mineral weathering and formation of weaker clay minerals, for example, are preparatory factor for landslides. The activating factors, on the other hand, include external forces like earthquake, rainfall, human activities like cutting and filling works, other construction works, etc. Similarly, the sustaining factors include changes like rise and fall of groundwater level.

In general, the creep phase of the landslides is considered to be the preliminary stage of a progressive failure. It may mean that the creep movement of natural slopes is preparatory stage of large-scale landslides, which gives enough signals before turning into a catastrophic landslide disaster. It is also considered, however, that the creep phase of the landslides is reactivation of relict landslides that took place in the long past due mainly to earthquake forces or tectonic activities through the faults.

リングせん断試験による膨張性粘土の強度特性と地すべりのクリープ移動挙動への影響に関する研究
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One strong point to justify this point in case of the landslides in Shikoku might be formation of the clay layers as a result of rock mineral weathering through the existing plane of failure. The authors consider that the decomposition of rock minerals due to chemical action of underground water takes place through a previously developed plane of failure. After a certain degree of decomposition, the shear strength of the slip layer material reduces considerably causing slope instability but not necessarily resulting in complete failure. The stress conditions in the slip layer material are such that it undergoes creep failure, in which the stresses usually do not change but shear deformations do occur under constant high confining stresses. It is therefore worth viewing the mechanism of the landslides in Shikoku in terms of preparatory and activation stages.

2.1 Preparatory stage

The rock mineral weathering even at greater depths in the slopes of Shikoku Mountains is enhanced by the presence of fractures in the rock masses. This is because the underground water in the fracture space reacts with the rock minerals under extremely favorable geo-chemical environment to form weaker minerals along the fracture planes. This process takes place repeatedly for a long period and gives rise to the formation of a great amount of clay minerals inside the rock mass resulting in the development of clay layers that act as the slip layers for the landslides.

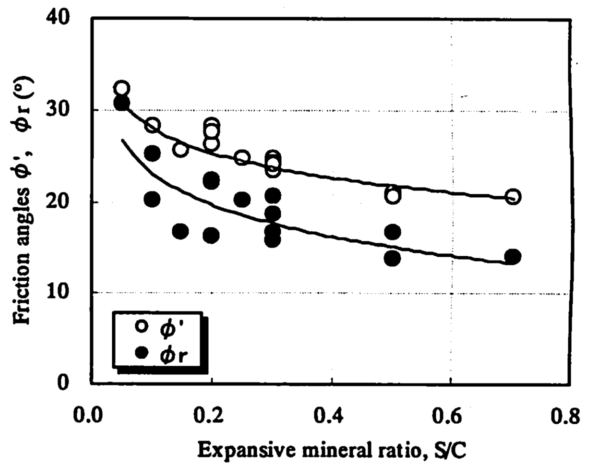

Figure 1: Drop in strength of landslide soil with increasing amount of expansive clay minerals. (After Yatabe et al., 2000)

Previous studies involving role of expansive clay minerals in activating landslides have shown that even a small amount of expansive clay minerals in soil greatly affects its strength behavior. Yatabe et al. (2000) have shown that the increase in relative amount of expansive clay mineral in landslide clays results in decrease of angle of shearing resistance, as illustrated in Figure 1. The expansive mineral ratio (S/C) in the figure refers to the relative amount of expansive clay minerals in the tested samples, as estimated from x-ray diffraction patterns (details in Yatabe et al., 2000). These experimental data make it clear that the expansive clay minerals play a significant role in causing the landslides in Shikoku, but it needs further investigation to elaborate the activation and displacement mechanisms of the landslides having slower rates of displacement.

2.2 Activation stage

The activation of the landslides in Shikoku may have two possible causes. First, it might have been caused by the seismic forces in the past, and the present state might be reactivation of the relict landslides under the influence of rock mineral decomposition through the plane of previous failure. Such a reactivation may also take place due to human intervention such as slope cutting, tunneling, etc. Second, the activation might have been caused by the clay layers formed as a result of rock mineral decomposition along the fracture planes, joint planes, and planes of mineral deposition. Whichever is the cause of activation, however, the prime factor must have been an earthquake or a heavy rainfall. Earthquakes may not be frequent enough to cause continuous movement of landslides, while the rainfalls not only activate the landslides but also help them sustain the motion.

The rainfall-induced failure through the slip layer soil of a landslide takes place due to rise in groundwater level. The influence of development of excess pore water pressure during shear might also be taken into account but the shear rate is often so slow that the generated excess pore water pressure dissipates with the shearing. Moreover, a shear after previously occurred large displacements may result in no change of volume, which means the chances of development of excess pore-water pressure are very less. A risen groundwater level following a rainfall, on the other hand, exerts an increased pore-water pressure resulting in reduced effective stress thereby reducing the shear strength of the slip layer soil.

Figure 2 explains the creep failure of a soil with the help of Mohr's stress circle. The authors make a hypothetical consideration that every soil has a zone of creep strength with a lower level equivalent to 90-95% of the angle of shearing resistance, as shown in the figure. The zone widens with the increase in the effective stress, which means the range of creep for deeper landslides is wider than that for the shallower ones. In other words, when a landslide has a deeper slip layer, the normal stress will be higher and the range of effective 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 3. It is because the range becomes wider with a decreased inclination of the strength envelope. The decomposition of rock minerals in the slip layer soil results in clockwise rotation of strength envelope with the reduction in angle of shearing resistance for the soil. This process brings the strength envelop closer to the circle representing the stresses in stable conditions such that a slight reduction in effective stress owing particularly to the rise in the pore water pressure results in creep failure as soon as the stress circle shifts toward left and enters the creep zone, as shown in Figure 2.

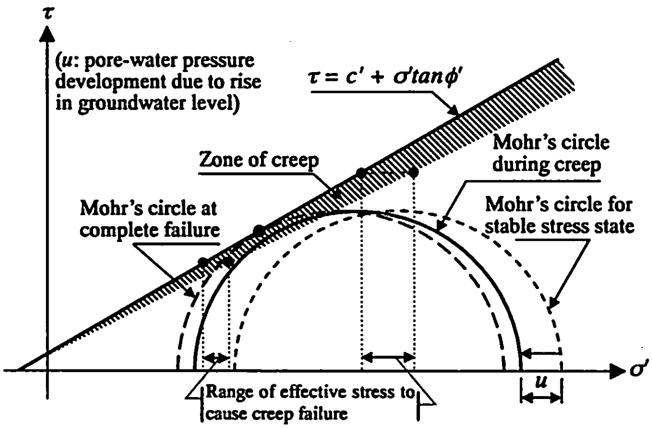

Figure 2: Creep zone for soils and creep failure explained by Mohr's stress circle.

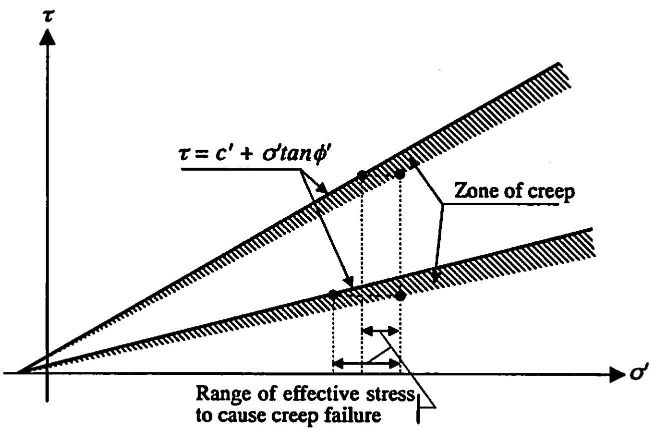

Figure 3: Range of creep failure for two soils with different strength parameters.

3 EXPERIMENTS

As a preliminary study on the strength properties of soils composed of smectite in an attempt to understand the mechanism of landslides with creeping displacements, Na-bentonite, which contains nearly 50% montmorillonite, was used in the tests. The tests were carried out on a ring shear apparatus based on simple shear principles. The merits of using a simple ring shear apparatus over a direct type are that the wall frictions are reduced and the shear plane during post-peak shear develops through the weakest plane across the depth of the specimen. It infers that the strength of a sample is overestimated in the direct ring shear apparatus, whereas it nears the true value if measured in simple shear type apparatus.

3.1 Method

The test program basically involved part of the attempts to study strength characteristics of bentonite as a highly expansive clay. The physical properties of bentonite and crushed Toyoura sand sample, which were employed in the experiments, are shown in Table 1. The test program consisted of three parts, namely ring shear tests on sand-bentonite mix samples, consolidation-swell test on bentonite, and ring shear tests on bentonite alone.

The test samples of sand-bentonite mix were prepared in 90/10, 80/20, and 70/30 proportion 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,

Table 1: Physical properties of soil samples tested

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

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 (i.e., 31.5cm of linear displacement).

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 consolidation 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 of the consolidation ring 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 the one obtained by applying a pressure of 9.81kPa. The second purpose of this test was to determine the swelling pressure of bentonite, which was based on consolidation-swell test as described by Nelson & Miller (1992). The consolidation was carried out as per the conventional method but the time required to achieve the end of primary consolidation for each stage of consolidation was 3 days in average, and that required during single stage swelling was one week.

In the third part, ring shear tests were carried out on post-swollen bentonite sample based on the consolidation behavior. One problem observed while passing water through the bottom of the specimen was that the thicker specimens took days to swell to full volume, while thinner specimens took comparatively short time even under the same pressure. It was found that the bottom part of the specimen started swelling as soon as the water entered the specimen, which resulted in extremely less permeability and blocked the water passage upward. So, the initial thickness of the dry sample in the ring was set at 10mm with a void ratio of 2. The dry sample was then consolidated under a pressure of 9.81kPa, after which it was allowed to swell by passing water exactly in the same manner as in consolidation-swell test. Three sets of tests were planned such that the swollen sample after consolidating could be sheared under three different over consolidation ratios (OCRs), i.e., OCR=1, 2, and 3 so as to see if the state of over-consolidation makes any sense in case of purely expansive clays. To reduce the influence of initial parameters, the conditions set were full swelling, full consolidation, full stress release, and constant time of shearing, i.e., equal shear displacement. The shear displacement applied was similar to one during the tests on mix samples.

3.2 Interpretation of the test results

Figure 4 shows the results of ring shear tests on sand-bentonite mix samples in terms of their strength envelops. It is clearly seen in this figure that the strength of the mix decreases with the increased amount of bentonite. If the obtained results bear enough accuracy, it can be inferred that the strength of a soil in the residual state drops by nearly 50% even at a montmorillonite content as low as 15% (i.e. 30% of bentonite). The peak strength is also seen to have decreased by nearly 40% at the same 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.

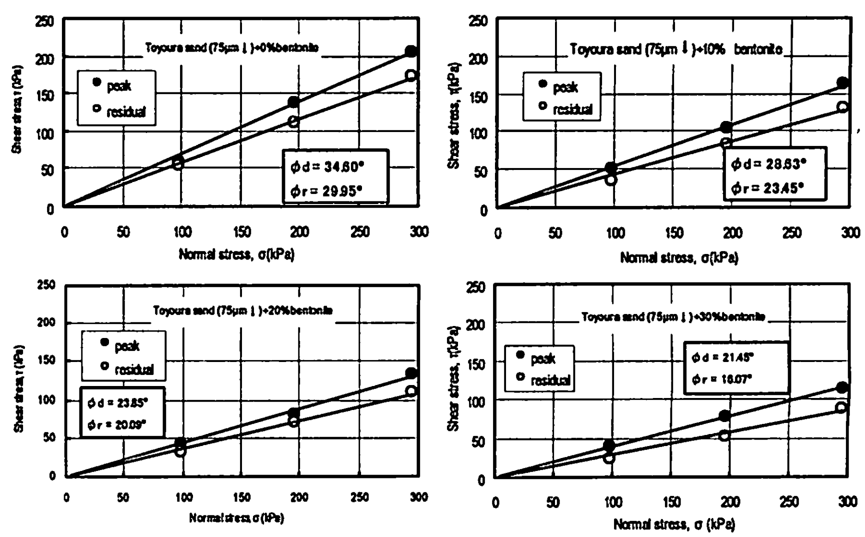

Figure 4: Strength envelopes for sand-bentonite mix samples as obtained from the simple ring shear tests.

The data obtained

from consolidation test on the unsaturated bentonite sample are plotted in Figure 5. The figure shows that the bentonite with an initial voids ratio of 2 and thickness of 10mm could be wetted until the water content reached 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 its initial volume is 380kPa, which according to Nelson & Miller (1992) is the swelling pressure for the tested sample. The consolidation under a pressure of 1255kPa resulted in a drop of water content down to a level as low as 15% and the voids ratio dropped close to 0.5, which is a significant drop compared to that for ordinary clay soils. From this, it is understood that the structural water in the bentonite can be largely squeezed out from the montmorillonite minerals by the application of 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 achieved a swollen state quite close to the state of full swelling. It is therefore considered that if the water could be passed uniformly throughout the specimen, the volume after the release of stress would have equaled the initially swollen volume.

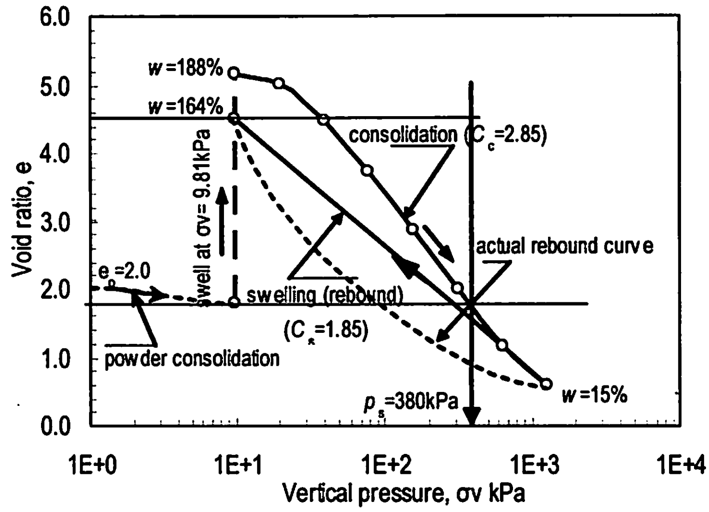

Figure 5: Results of consolidation-swell test on bentonite using oedometer.

Finally, the data obtained from ring shear tests on the pure bentonite sample are presented in Figure 6. A variation of 10% in the values of angles of shearing resistance can be seen depending on the over-consolidation ratio. However, if this variation is attributed to experimental errors, the strength values for the bentonite samples tested in three states of over-consolidation ratio are similar. What can be inferred from this is that the highly expansive clays lose the state of over-consolidation up on wetting. It must be noted, however, that the water content in the over-consolidated specimens is higher, which under similar initial conditions, means higher volume. So, the volume of over-consolidated specimens during the shear must have been greater than that of normally consolidated specimen. It might have been one reason for the insignificant change in strength parameters for an over-consolidated specimen. The reason for the higher water content may be comparatively flattened ring specimen, which could absorb more water than a thicker specimen could. In all cases, the peak angle of shearing resistance for bentonite was measured to be around 9º, whereas the residual angle was measured to be around 4º. Similarly, the cohesion intercepts in all cases were measured to be around 10kPa and 5kPa at peak and residual states, respectively.

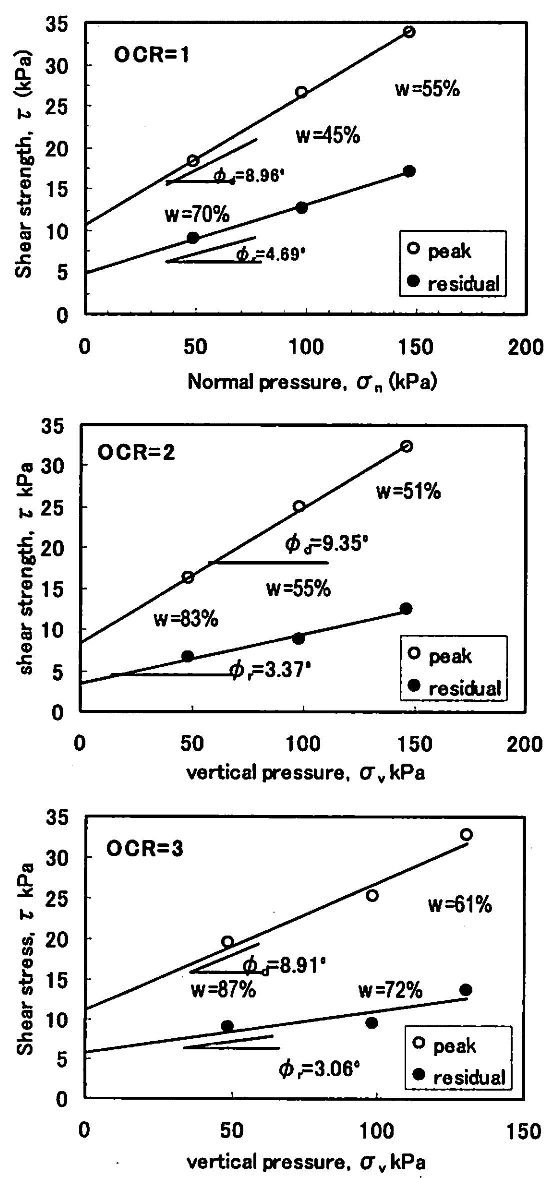

Figure 6: Results of simple ring shear tests on post-swollen bentonite samples.

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. However, due to high amount of water montmorillonite particles are considered to exhibit highly deformative behavior, which may greatly influence the inter-particle friction.

4 CONCLUSIONS

Expansive clay minerals composed in the slip layer clays of the landslides in Shikoku play an important role in activating the landslides. Particularly, the presence of expansive clay minerals in the slip layer soils reduces the shear strength, and as their amount increases, the strength parameters decrease. Moreover, it is considered that the strength behavior of expansive clays is responsible for causing creep displacement of most landslides in Shikoku. Therefore, attempts were made in this paper to study the strength behavior of expansive clays so as to understand more the mechanism of creep displacement of landslides in Shikoku. The following conclusions can be drawn from the results obtained from the laboratory tests.

a) 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.
b) Increased expansive clay mineral content is considered to cause wider range for creep failure by reducing the inclination of strength envelope.
c) Despite a free swell value above 1200%, the bentonite employed 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.
d) 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.
e) 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 greatly squeezed out.
f) 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 and the swollen volume could reach the initial values.
g) State of over-consolidation was not found to have significant effect on strength behavior of bentonite. However, there might have occurred some experimental errors that resulted in similar strength parameters regardless of the state of over-consolidation.

Finally, as it was just a step toward investigating 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 further would clarify the creep mechanism of large-scale landslides.

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