# Ring Shear Tests on Clays of Fracture Zone Landslides and Clay Mineralogical Aspects

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**Abstract.** Various investigations on tectonically-induced landslides in Shikoku Region of West Japan have been carried out, most of which conclude at tectonic activities through the major tectonic faults and enhanced rock mineral decomposition as being mainly responsible for the landslide occurrence. This paper looks into strength parameters of the landslide clays, as measured in ring shear apparatus, from clay mineralogical point of view. As a result of strength tests and X-ray diffraction analysis, it is found that the drop from peak to residual friction angles for the tested samples reaches as high as 20°, and the residual strength of the landslide clays was found to decrease with higher amount of expansive clay minerals, which was estimated as being relative to chlorite mineral.

**Keywords.** Expansive clay minerals, expansive mineral ratio, fracture zone landslides, ring shear test, X-ray diffraction test

## 13.1 Introduction

Landslides are common natural phenomena that occur mainly under the influence of external inducing factors like earthquake forces and rainfall and internal inducing factors like slope material properties and slope conditions. Although many large-scale landslides do also occur in plains such as near the banks of deep cut rivers, the landslides to be discussed in this paper occur predominantly on mountainous terrains mostly with creeping displacements. Slopes in mountainous terrains often suffer failures in the form of minor collapses like rain-induced surface layer failures to large-scale failures like deep-seated gravitational creeps or massive slope movements. Japan with its more than 70% mountainous terrain has recognized a large number of active and potential landslide sites that threaten a huge loss of life and infrastructure. Shikoku Region in west Japan, for example, has a large number of tectonically induced landslides, often known as fracture zone landslides, as indicated by landslide distribution map in Fig. 13.1 (MLIT 1997). As of March 1997, the number of landslides under the direct jurisdiction of the Ministry of Land, Infrastructure, and Transport (abbreviated hereinafter as MLIT) in Shikoku alone is 670, which is nearly 21% of the number all over the country (ibid.).

The large-scale creeping landslides are often known for no immediate damages, but a rapid progress in their

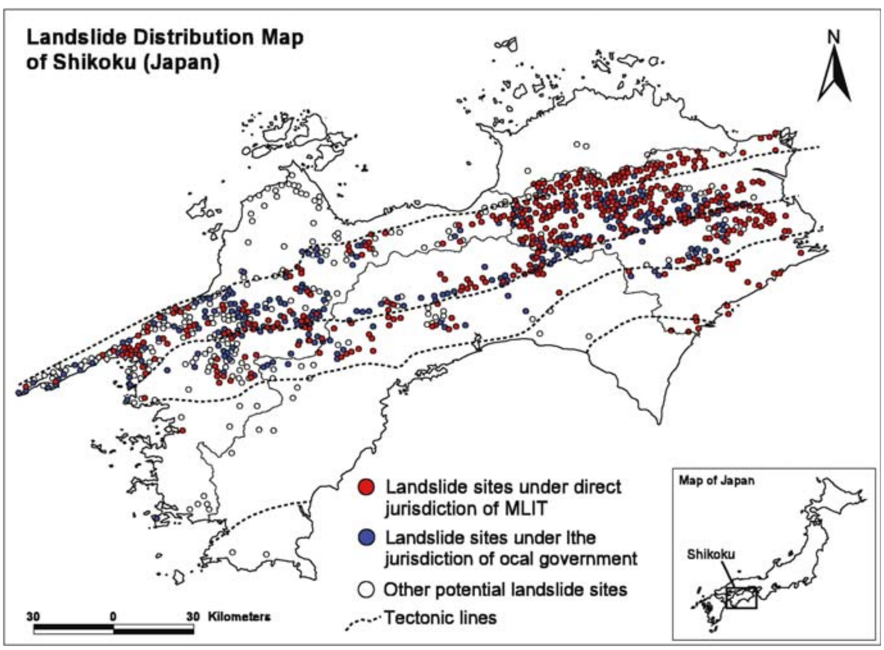



**Fig. 13.1.**
Landslide distribution map of Shikoku (Fracture Zone Landslides, reproduced from MLIT 1997)

displacement may result in disastrous effects such as damming up of rivers, flow of human settlements, destruction of agricultural fields and forests, destruction of highways and roads, damage to dams, tunnels, highways and bridges, and transmission towers, and destruction of nature and environment. To overcome potential landslide threat, various landslide prevention and management projects are in execution throughout the Shikoku Region, some under the supervision of the local governments but most under the direct control of the MLIT. Such project works involve planning and design of proper and appropriate countermeasures, which require greater accuracy in the stability analysis. It is therefore important to have an in-depth understanding of the mechanism involved in creeping displacement of large-scale landslides.

Various field investigations carried out during the execution of landslide prevention and management projects have revealed that the landslides in Shikoku Region consist of clearly identifiable slip layers composed largely of fine clayey soils varying in thickness from a few centimeters to as thick as 20 centimeters (Yagi et al. 1990). As an attempt to elucidate the failure mechanism of slip layer soils and associated landslide displacement behavior, Yagi et al. (1990) and Yatabe et al. (1991a,b) have investigated shear characteristics of several clayey soil samples from the slip layers of landslides in various parts of Shikoku. Based on the laboratory shear tests, they found that most landslides in Shikoku Region are in pre-residual state of shear, which infers that the field angle of internal friction is greater than the residual angle of friction measured in the laboratory. Yagi et al. (1990) also compared the strength parameters of landslide clays in Shikoku with those of marine clays and found that the peak friction angles for the landslide clays were smaller by 3° to 8° than those for the marine clays in Japan. They mention that the fabric of landslide clay is different from that of marine clay and discuss that this difference can be attributed to the process and environment the clay soils are formed in. This paper is an extension of the work the above researchers carried out, and attempts further to look into shear strength characteristics of the clayey soils involved in the slip layers of fracture zone landslides based mainly on the ring shear tests. In addition, emphasis is put on investigating the influence of clay mineralogy on the angles of shearing resistance of the landslide clays, and its role in inducing the fracture zone landslides.

## 13.2 Features of Fracture Zone Landslides

### 13.2.1 Geological Background

Some of the major tectonic lines in Japan, often measuring hundreds of kilometers, pass through Shikoku Island. For example, Japan's longest tectonic fault system called *chuokōzōsen* (also known as median tectonic line and abbreviated as MTL) separates two major geological formations in the northern half of Shikoku Island. Likewise, two equally important tectonic lines known as Mikabu Tectonic Line and Butsuzo Tectonic Line (note: other minor tectonic lines have not been mentioned due to their insignificant role in causing landslides) pass across the island, and together with the MTL they divide Shikoku Region into four major strips of geological formations, namely Izumi Group, Sambagawa belt, Chichibu belt, and Shimanto belt including a few more minor geological formations (Fig. 13.2; GSJ 1992). The belts recognized as being particularly prone to landslides are Izumi Group,

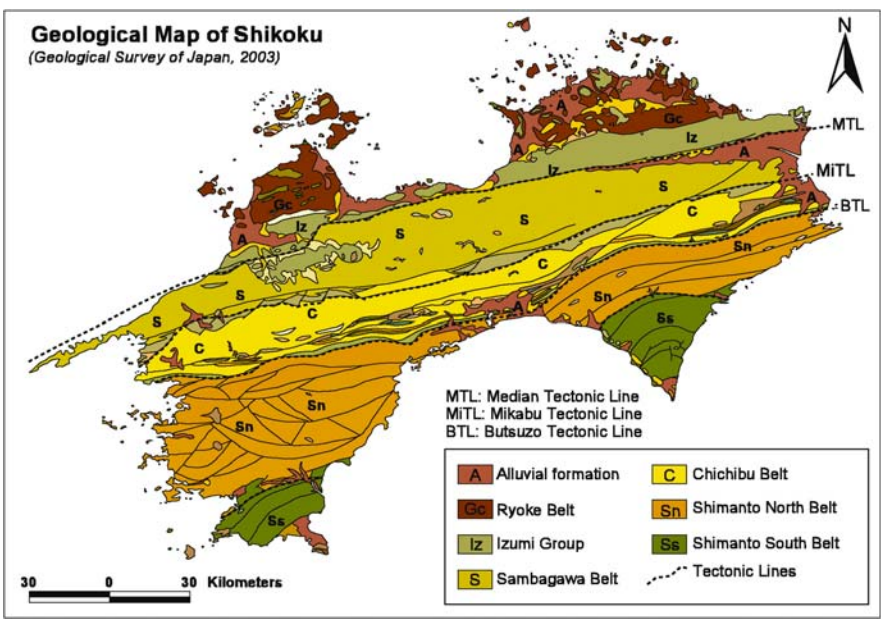



**Fig. 13.2.** Geological map of Shikoku showing major tectonic lines (GSJ 2003)

Sambagawa belt, Mikabu belt (a narrow strip along the Mikabu Tectonic Line separating Sambagawa belt and Chichibu belt), and Chichibu belt, which can be verified by superimposing Fig. 13.1 over Fig. 13.2.

The Izumi group is a sedimentary deposit composed primarily of sandstone with the intrusion of thin layers of shale, whereas the Sambagawa belt is a deposit of metamorphic rocks consisting mainly of green schist and black schist. Similarly, the narrow strip of Mikabu belt consists of greenstone (often known as Mikabu greenstone) as a metamorphic deposit. The rock type in the Chichibu belt is sedimentary composed of green schist, mudstone, conglomerate, etc.

### 13.2.2 Landslide Activation

The investigation and study on landslides in Japan began only half a century ago when the landslide prevention law was enacted in 1958 (JSSMFE 1985). While many landslides have remained active since long, a considerable number of relict landslides or landslide remnants in Shikoku were reactivated during the construction of express highways along the median tectonic line (MTL). Such reactivation is attributed mainly to huge slope cuts and tunneling. As also indicated in Fig. 13.1, the distribution of landslides in Shikoku can be found notably concentrated over two areas between the median and Mikabu tectonic lines.

In addition to other various factors, the internal factors causing landslides in Shikoku are flow behavior of groundwater, fluctuations in its level, and changes in soil strength properties. As reported by Scheidegger (1970) and supported by subsequent researchers such as Slivosky (1977), Carraro et al. (1979), and Varnes et al. (1989) the tectonic activity and earthquakes might also be related to the activation of landslides in the form of deep-seated gravitational creep. Particularly in Shikoku, the role of tectonic activities in inducing the landslides has been significant.

One highly favorable condition for land sliding in Shikoku is considered to be fractured state of the bedrocks, especially near the tectonic lines (MLIT 1997). It is believed that the fragile state of the bedrocks accelerates the rock mineral weathering by facilitating water ingress into the rock mass. Such weathering or decomposition of the bedrock minerals on a certain plane results in formation of clayey soils composed of weaker minerals such as chlorite, smectite, and vermiculite. Smectite and vermiculite minerals possess significantly small angles of shearing resistance at the residual state, which is because they absorb excess water and swell significantly upon wetting. So, a slight decrease in effective stress due especially to rise in groundwater level and simultaneous increase in driving shear stresses result in sliding of the slope material through the clay layer composed of weak minerals.

### 13.2.3 Topographical Features

Topographical features of creeping landslides, in general, consist of mild and gentle natural slopes, especially slopes that probably collapsed in the past and have attained a relatively gentle angle of inclination, i.e., in a range between 15° and 30°. The mountains of Shikoku compared to those in other regions are steep with slope inclinations ranging from 20° to 40°. It is also evidenced by a topographical feature that despite a narrow width of the island, some of the tallest mountains in west Japan lie in this region. Most mountain ranges form an alignment parallel with the fault lines extending in east-west direction, but minor cross alignments can be seen all throughout the fault lines due especially to cuts produced by streams and rivers. It is therefore obvious that the angles of slope for the landslide sites in Shikoku are high. Figure 13.3 (Ishii 1994) shows a comparative chart indicating that the landslide sites in Shikoku (represented by fracture zone landslides) have steeper slopes than those in Tertiary system deposits, which are dominant in the northern Japan and consist mainly of weathering-prone soft rocks such as Neogene sedimentary rocks, Palaeogene sedimentary rocks, and Tertiary volcanic rocks (MLIT 1997).

The average inclination of slope for the fracture zone landslides, as seen in Fig. 13.3, is much higher than that for the tertiary zone landslides, which more or less means that the size of a fracture zone landslide must be smaller than that of a tertiary zone landslide. This is evidenced by Fig. 13.4 (Ishii 1994), which compares the sizes of the fracture zone landslides and the tertiary zone landslides. This figure shows that most of the landslides in the fracture zone (represented by Sambagawa, Mikabu, and Chichibu in the figure) are smaller than those in the tertiary zone although the size ratio of the landslides in both the zones varies from $L = 0.5W$ to $L = 3.0W$, where $L$ represents the slope length and $W$ represents the width. The

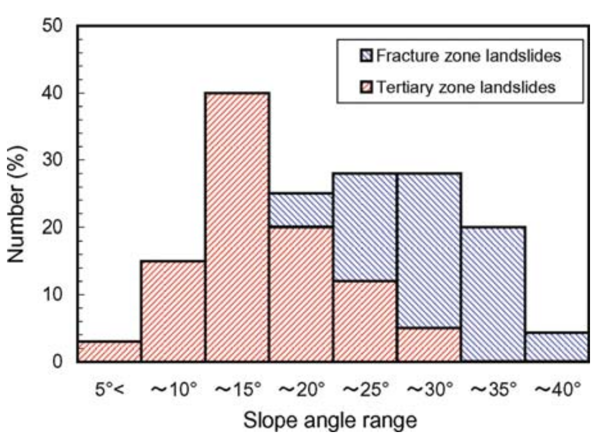



**Fig. 13.3.** Range of landslide slope angles for fracture zone landslides and tertiary landslides (from Ishii 1994)

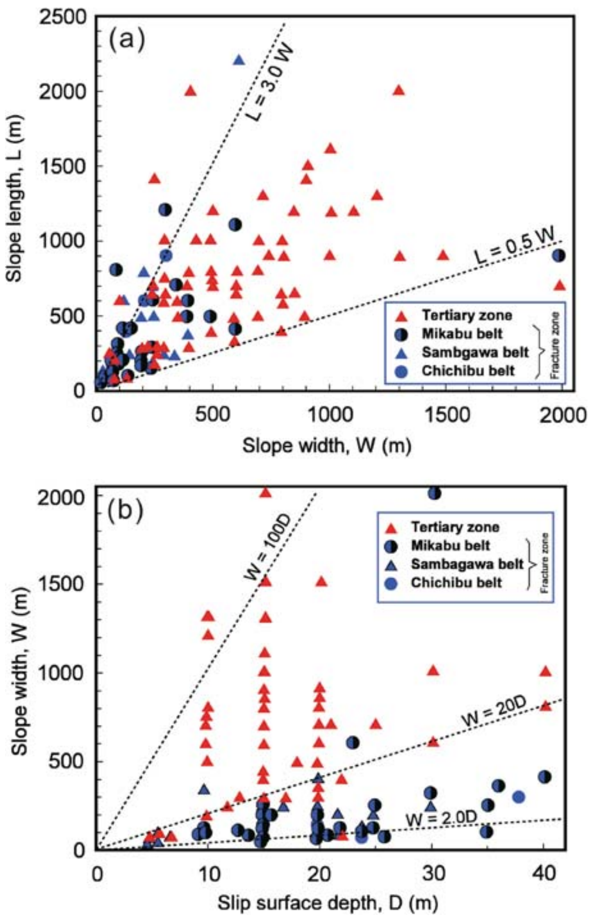



**Fig. 13.4.** Topographical features (slope width vs. slope length and slip surface depth vs. slope width) of fracture zone landslides (from Ishii 1994)

figure also shows that the average depth of sliding mass for the same width in the fracture zone is much higher than that in the tertiary zone. It is seen that the ratio of width to depth for most fracture zone landslides varies from $W = 2.0D$ to $20.0D$ ($D$ represents the depth), whereas that for most tertiary zone landslides varies from $W = 20.0D$ to $W = 100.0D$. These width-depth relations make it clear that the landslides in the fracture zone have comparatively deep slip surfaces. In addition to large surface areas, which may range from thousands of square meters to a few square kilometers, landslide depths (or the depth of sliding mass) at the deepest point have been reported to be as high as 100 meters in the Shikoku Region (e.g., Kage landslide in Kochi Prefecture (Yube 2001)).

One another notable feature of the landslides in Shikoku is rate of sliding, which is regarded relatively faster than that for tertiary landslides (MLIT 1997). Ishii (1994) mentions that the rate of displacement of creeping landslides in the fracture zone is often found to go as high as several tens of centimeters a month, especially during rainy periods. With the lowering of groundwater level, however, the monthly rate of displacement comes down to only a few centimeters. Such slow displacements of the landslides take place under a condition when shear stress through the slip surface (i.e., slip layer) is slightly less than the shear strength of the slip layer soil, which causes a condition of slope instability with a factor of safety slightly greater than the unity. The failure of slip surface soil in such a condition is supposed to be due particularly to creeping behavior, in which a soil sample subjected to stresses equivalent to 90–95% of the shear strength in a longer time span deforms without resulting in complete failure.

## 13.3 Experimental Program

### 13.3.1 Material

The test samples were obtained from boring cores of over 20 landslide sites in Shikoku, most of which were from Sambagawa and Mikabu belts (Fig. 13.2) including a few samples from Chichibu belt. In addition to determination of physical properties of all collected samples, the laboratory investigation consisted of (*1*) shear strength measurement in ring shear apparatus and triaxial compression cell, and (*2*) identification of constituent minerals in the collected samples and quantitative evaluation of the constituent clay minerals with the help of X-ray diffraction analysis.

Figures 13.5 and 13.6 summarize a part of the physical properties of the tested clay samples (finer than 425 μm) in terms of grain size distribution, liquid limit, and plastic limit values. In Fig. 13.5, an overall picture of grain size distribution (in three gradations of finer than 2 μm, 2 to 20 μm, and 20 to 425 μm) of all collected samples has been plotted against the plasticity index values. As seen in the figure, the clay fraction (i.e., the percentage by weight of the soil particles finer than 2 μm) varies from a

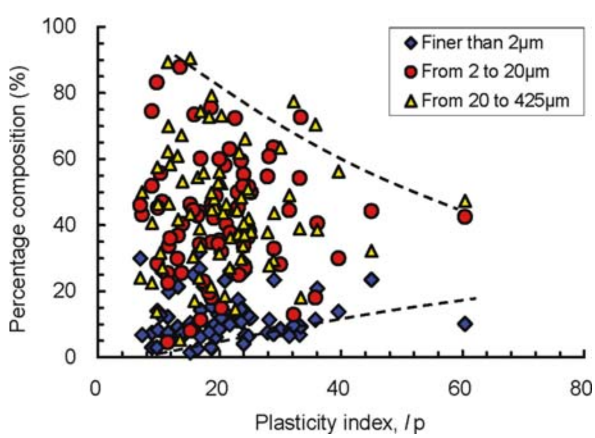



**Fig. 13.5.** Grain size distribution (finer than 425 μm) for tested landslide soil samples plotted against the plasticity index

few percent to 30 percent, and the amount of coarse particles (i.e., 20 to 425 µm) varies from a few to as high as 80 percent. Although in general the plasticity index value for a soil increases with higher percentage of clay fraction, this figure indicates that rather than due to increase in the percentage of clay fraction, the plasticity index seems to increase with the decrease in the percentage of coarse particles.

Figure 13.6, on the other hand, indicates the range for liquid limit and plastic limit for the collected samples. It is apparent that the plastic limit values remain within a range between 10% to 40%, while the liquid limit values

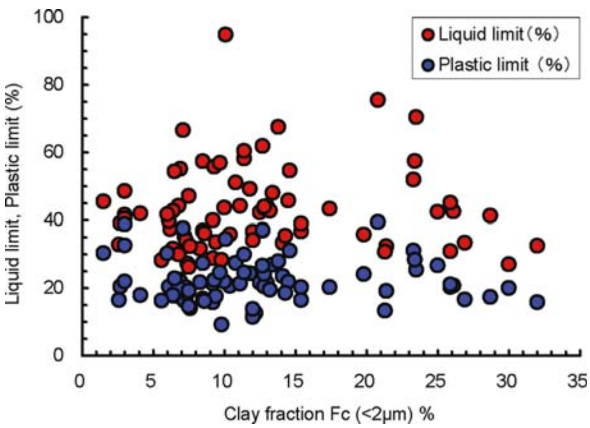



**Fig. 13.6.** Liquid limit and plastic limit values for the tested landslide soil samples plotted against clay fraction (<2 µm)

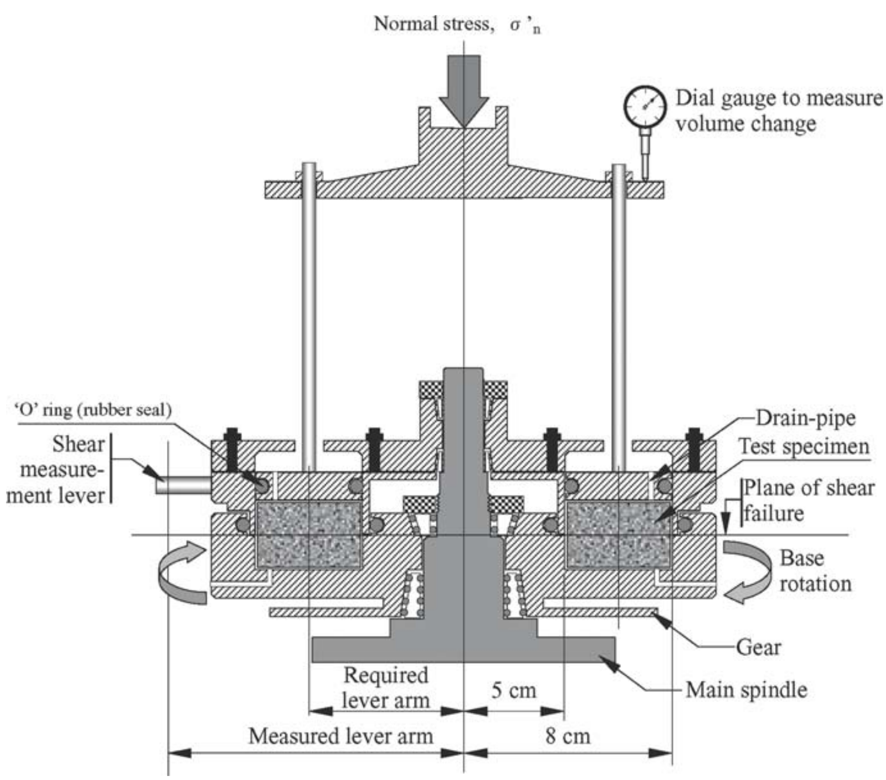
vary from 25% to as high as 95%. Although there are slight indications that the liquid limit values increase with the increase in clay fraction value up to 15%, the scattered data indicate that the influence of clay fraction is less significant than other factors such as clay mineralogy.

## 13.3.2 Method

### 13.3.2.1 Strength Tests

As stated above, ring shear tests and triaxial compression tests were planned for the strength measurement of the collected landslide clay samples. The ring shear tests were based on Bishop's ring shear apparatus (Bishop et al. 1971), and were performed on remolded reconsolidated specimens under drained conditions. The annular specimen in the ring shear apparatus measured 160 mm in outer and 100 mm in inner diameter with a thickness of 10 mm. Figure 13.7 shows a not-to-scale diagram of the ring shear apparatus used in the study, which was produced for the purpose of trial experimental studies. Similar to most other commonly employed ring shear apparatuses, it consisted of lower and upper parts. The shearing assembly was such that the lower part of the apparatus could be rotated with the help of speed regulated electric motor, while the upper part including the loading platen could only move but not rotate. The torque due to the rotation of the lower part could be transferred to the

**Fig. 13.7.** Ring shear apparatus employed in strength tests



upper part through the specimen in the sample box and measured at the shear measurement lever in contact with an automatic load measuring device. Being a machine produced for the trial purpose, the expectations of mechanical friction, such as due to bearing contact, O-ring rubber seal contact, and contact between the loading platen rods and movable upper part of the apparatus, were high. However, it was confirmed that the friction remained below 30 N even during the application of a vertical pressure of 300 kPa. Moreover, sufficient drainage during shear (due to sample contraction) was ascertained by providing four drain holes through the loading platen as well as the lower base and using high permeability filter papers on top and bottom of the specimen.

The test specimens were prepared in the ring shear apparatus itself by one-dimensionally consolidating remolded pre-saturated samples passing through 425 μm sieve. Each specimen was sheared under three sets of effective pressure (i.e., vertical pressure, $\sigma_v$) of 98.1 kPa, 196.2 kPa, and 294.3 kPa. The rates of displacement for pre-peak and post-peak shears were set to be different because a faster shearing rate could produce excess pore-water pressure and a slower shearing rate could result in many days to achieve the displacement required for measuring the actual residual strength of the sample. Based on the method described by Garga (1971), the rate of shear until the peak strength value had been exhibited was set to be 0.044 deg min$^{-1}$. (i.e., 0.05 mm min$^{-1}$), which was then increased 10 times (i.e., 0.44 deg min$^{-1}$ or 0.5 mm min$^{-1}$) for the post-peak shear until steady states of shear strength and volume change were confirmed. Depending on the state of consolidation and sample type, the displacement required to achieve the peak strength value ranged from 2 to 7 mm. Likewise, the angular rate of shear of 0.44 deg min$^{-1}$ in the ring shear apparatus employed could produce a total displacement of 72 cm in 24 hours through the plane of shear, but for most samples the residual state of shear was achieved before a shear displacement of 50 cm.

The triaxial tests, on the other hand, were performed in isotropically consolidated-undrained conditions with pore-water pressure measurement (i.e., CU-bar tests) at controlled rate of strain. The test specimens were prepared by one-dimensionally consolidating remolded pre-saturated samples passing through 425 μm sieve. Each prepared cylindrical specimen measured 35 mm in diameter and 80 mm in height. Each sample was tested under three sets of total confining pressure ($\sigma_3$) of 98.1 kPa, 196.2 kPa, and 294.3 kPa. The rate of compression was set at 0.044 mm min$^{-1}$, at which the development of pore water pressure throughout the specimen would be uniform. To ascertain highest possible degree of saturation, a back pressure of 196.2 kPa was applied to each specimen, which raised the B-value (confining pressure-dependent coefficient of pore water pressure) up to 0.95 against a rise in confining pressure of 98.1 kPa.

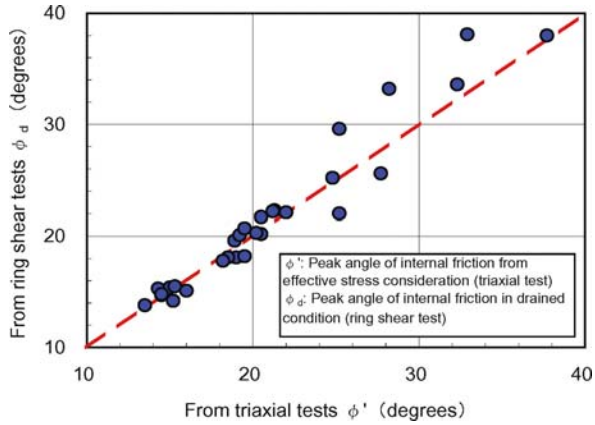



**Fig. 13.8.** Comparison of angle of effective internal friction (triaxial compression test) and angle of drained internal friction (ring shear test)

The main purpose of conducting triaxial compression tests was to ensure accuracy and applicability of the soil strength measured in the ring shear apparatus. Moreover, as the peak strength (drained strength) measured in ring shear apparatus could be erouneous due to unknown friction as well as unconfirmed excess pore water pressure development in the plane of failure during pre-peak shear, the results of triaxial compressions tests were employed to evaluate the peak shear resistance of the tested samples. Figure 13.8 shows a comparison of the angles of drained shear resistance for a few samples as measured in ring shear apparatus and the angle of effective shearing resistance as obtained from triaxial compression tests. The figure is evident that the drained shear strength (ring shear apparatus) and effective shear strength (triaxial compression test) are almost equal. Theoretically, the peak friction angle measured in ring shear apparatus at drained conditions must be close (if not equal) to the effective angle of friction measured in triaxial tests. It is because the total stress during fully drained conditions is equivalent to the effective stress. So, the choice was made of effective friction angle to represent the actual value of peak angle of friction for the tested samples.

### 13.3.3 X-ray Diffraction Analysis

Two basic methods employed to identify the constituent minerals of the collected soil samples were powder method and oriented aggregate method. The powder method of X-ray diffraction analysis, in which all size particles are crushed into fine powder, is meant for identifying all constituent minerals of the sample, while the oriented aggregate method, in which only finer than 2 μm particles are sampled from the soil-water suspension, helps to identify constituent minerals of clayey particles. Other relevant information in relation with the X-ray diffraction analysis method will be discussed in the following section.

## 13.4 Results and Discussion

### 13.4.1 Shear Characteristics

All strength test results have been expressed and presented in terms of angle of internal friction. In Fig. 13.9, the angles of internal friction as obtained from the triaxial compression tests (i.e. angle of effective internal friction, $\phi'$) and ring shear tests (i.e. residual angle of internal friction, $\phi_r$) have been plotted against the clay fraction $F_c$. Various curves of $F_c$-$\phi_r$ relationship such as by Skempton (1964), Borowicka (1965), Binnie et al. (1967), and Blondeau and Josseaume (1976) have also been included for reference and comparison. No clear drop in internal friction angles with the increase in clay fraction value can be confirmed from this figure. This infers that unlike common natural clays, the landslide clays of fracture zone exhibit scattered correlations between strength and physical parameters.

In next attempt to see the influence of physical properties on strength behavior, the internal friction angles were plotted against the plasticity index ($I_p$), as shown in Fig. 13.10. For reference and comparison, various other data ranges (Vaughan et al. 1978; Bucher 1975; Kanji 1974; Seyček 1978; Fleischer 1972; Voight 1973) for $I_p$-$\phi_r$ relationship have also been included in this figure. The angles of residual internal friction for most tested samples lie within the range mentioned by Bucher (1975), but it is hard to draw a clear relationship between the plasticity index and angle of residual internal friction for the fracture zone landslide soils. The range of variation is very significant in compared with the results obtained for other natural clays such as by Vaughan et al. (1978), Kanji (1974), Seyček (1978) Fleischer (1972), and Voight (1973). As seen in the figure, the effective friction angles range from 20° to 40°, and the residual friction angles range from 10° to 25°. It is also worth noting that the drop from peak to residual values of friction angle is comparatively high. Figure 13.11 shows that such drops for the landslide soils in Shikoku range from a few degrees to as high as 20°. In general, the drop from peak to residual strength is considered to be higher for more plastic soils because higher drop means higher clay content and higher clay content means higher plasticity. However, it is interesting to note that the drops for the tested landslide soil samples do not definitely follow this pattern. Except for some, most samples exhibit scattered pattern.

In Fig. 13.12, the residual friction angles ($\phi_r$) of only selected samples (the samples that were tested for constituent minerals, which are also plotted in Fig. 13.9) have been plotted against clay fraction, $F_c$. As also discussed above, the $F_c$-$\phi_r$ relation for these soil samples cannot be interpreted as what is generally regarded (such as Lupini et al. 1981; Skempton 1985). Rather than having a fine reduction in residual friction angle with the increase in

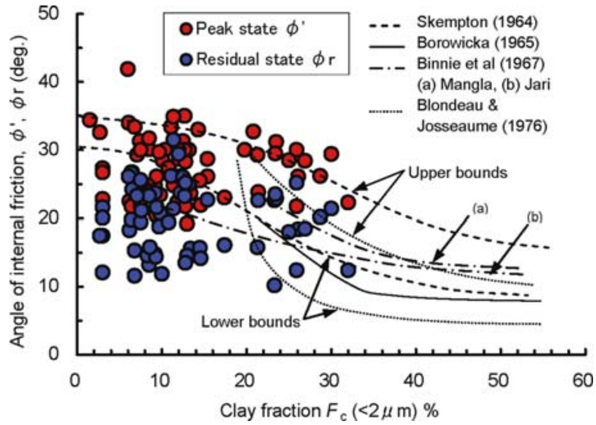



**Fig. 13.9.** Angles of internal friction for the landslide soil samples plotted against the clay fraction

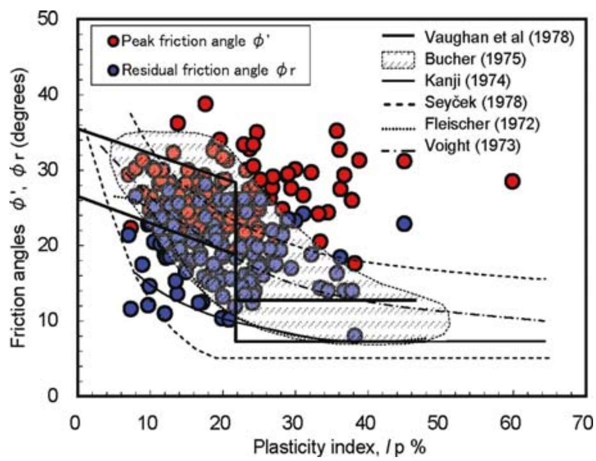



**Fig. 13.10.** Angles of internal friction for the landslide soil samples plotted against the plasticity index

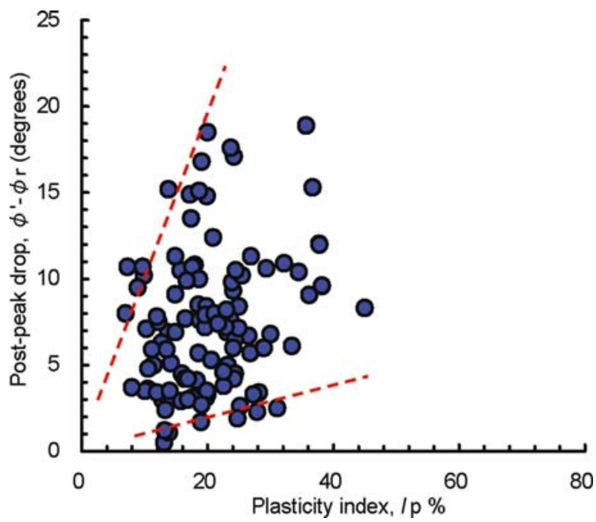



**Fig. 13.11.** Amount of drop from angle of peak internal friction to angle of residual internal friction

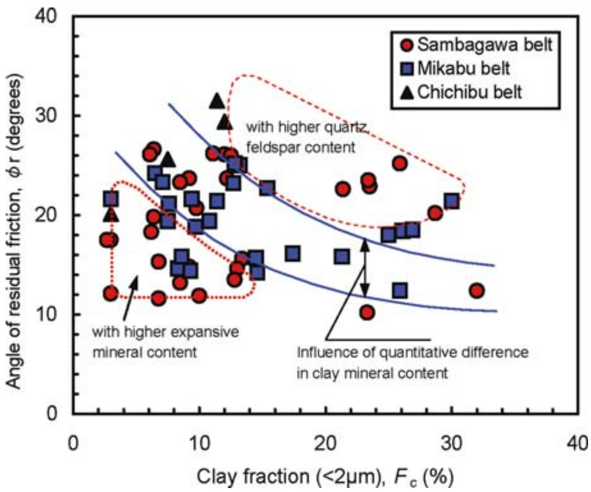



**Fig. 13.12.** Angles of internal friction for selected landslide soil samples plotted against the clay fraction also showing the influence of constituent minerals

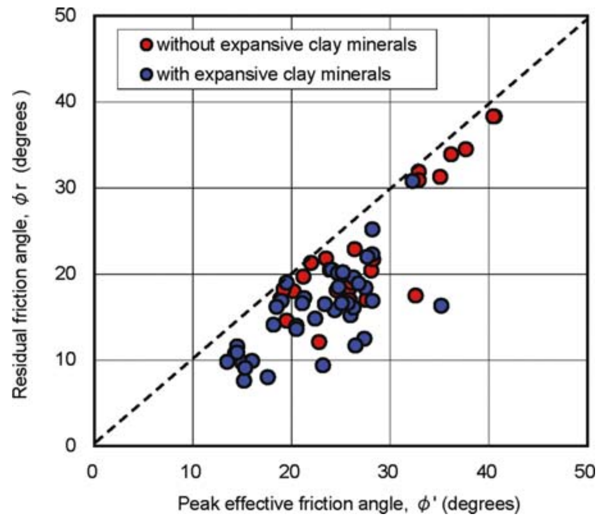



**Fig. 13.13.** Comparison of angles of internal friction for the samples with and without expansive clay minerals

clay fraction, most data are scattered, which can be attributed to influence of proportional composition of different clay minerals, as also indicated in the figure (based on X-ray diffraction analysis data). It can be interpreted from the results of X-ray diffraction analysis (to be discussed in the following section) that different amounts of constituent minerals might have given different shearing properties to the samples tested. Moreover, even at a clay fraction of as low as 10–30%, the average value of residual angle of friction is seen to be close to 15°, which is generally considered a low value. To investigate the reason for lower values of residual friction angle it was therefore essential to analyze the results of strength tests and X-ray diffraction analysis together, which is discussed in the following section.

### 13.4.2 Clay Mineralogy

Figure 13.13 compares the angles of shearing resistance for some of the tested samples possessing expansive and non-expansive minerals (expansive minerals here refer to smectite and like clay minerals, especially in interstratified state of expansive and non-expansive clay minerals such as smectite and chlorite). Although it is difficult to draw a clear relationship between expansive clay mineral content and shear strength properties, interpretations from this figure can be made that the angles of internal friction for the soils with expansive clay minerals are smaller than those for the soils with no such clay minerals, especially in residual state.

To investigate the influence of expansive clay mineral content on the strength, attempts were made to estimate the relative amount of the expansive clay minerals present. A method was adopted for the quantitative estimation of expansive minerals such as smectite and expansive chlorite. The term expansive chlorite here is used to represent a phase of chlorite mineral, in which certain portion of chlorite behaves like smectite in presence of water (i.e., interstratified chlorite-smectite mineral). Chlorites are formed generally by the alteration of smectite in presence of sufficient Magnesium ($Mg^{2+}$), which causes a crystalline brucite layer (octahedral structure with Mg) to replace the interlayer water (Mitchell 1976). It is however considered that the phase change also takes place from chlorite to smectite especially during mineral decomposition resulting in partial alteration of the mineral layers in chlorite to produce expansive chlorite, part of which behaves like smectite.

To estimate the relative amount of smectite in expansive chlorite, the use is made of X-ray diffraction patterns in powder method and glycol-treated oriented aggregate method. When an untreated sample containing expansive chlorite is placed on X-ray diffractometer, the peak for chlorite mineral, which has a basal spacing of 14 Å, is exhibited at a diffraction angle between 6.1°–6.4° and the peak for the expansive fraction of the mineral is exhibited near a diffraction angle of 6°. When treated with ethylene glycol, the position of the peak for the chlorite portion remains the same, whereas the position of the peak for the expansive fraction shifts to 5.2°–5.5°, which is a range of diffraction angle for glycolated montmorillonite (smectite). When a ratio of peak intensity for montmorillonite to that of chlorite near the diffraction angle of 6° is calculated, as illustrated in Fig. 13.14, it should represent the relative amount of smectite mixed with chlorite in a test sample (i.e., A/B in the figure). The data obtained from such calculations are plotted against the angle of shear-

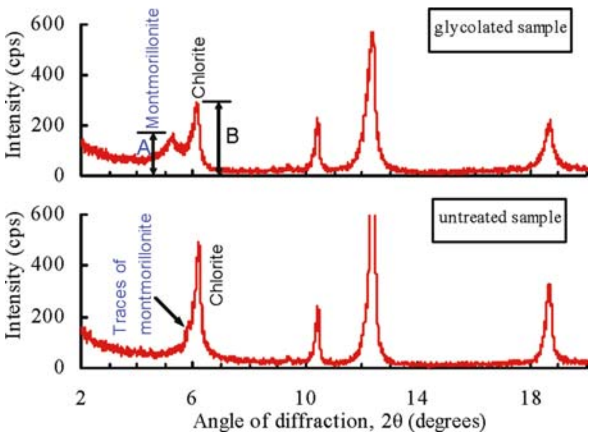



**Fig. 13.14.** Typical X-ray diffraction patterns for untreated sample and glycolated sample in oriented aggregate method showing the presence of smectite (montmorillonite) and the parameters (A, B) used in estimating relative amount of expansive clay mineral

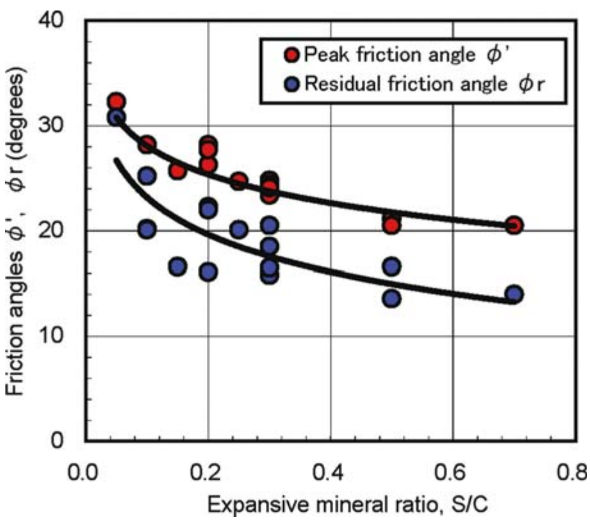



**Fig. 13.15.** Influence of expansive mineral ratio (S/C) on the angles of internal friction for selected landslide soil samples with smectite content

ing resistance for the tested samples in Fig. 13.15. The relative amount of expansive clay mineral is termed as expansive mineral ratio and is represented by a ratio S/C, in which S stands for smectite and C stands for chlorite.

The figure shows that the angles of shearing resistance for the tested samples decrease with the increasing values of the expansive mineral ratio. The data are slightly scattered, which can be attributed to experimental errors, especially with the ring shear tests. Moreover, the influence of some of the common constituent minerals of the tested samples such as mica, quartz, tremolite, and other non-clay minerals is not considered. So, if this effect is supposed to be minimal and the effect of expansive clay minerals is exaggerated, a drop of 10° in the angle of friction can be clearly seen against an increase of expansive mineral ratio by 0.5. Such a variation of angle of shearing resistance with the amount of smectite and similar minerals reveals that the degree of instability for a creeping landslide increases with increasing chances of other minerals turning into smectites. On the other hand, if the geochemical environment is favorable for smectite minerals to turn into other minerals like chlorite, the stability will increase. Therefore, it might be possible to stabilize creeping landslides to some extent chemically by introducing some substance in the slip surface soil to control the formation of weak clay minerals like smectites.

## 13.5 Concluding Remarks

This paper introduced the features of fracture zone landslides in Japan, where a large number of active and potential landslide sites have been identified and are under preventive efforts. The tectonic activities have led to formation of fracture zone underneath the Shikoku Region of Japan, which has given rise to abundant space for underground water that causes decomposition of the rock minerals leading to formation of weaker clay minerals in a layer of soil potentially acting as slip surface for the landslide. Smectites, chlorite, vermiculite, and illite are found as common clay minerals in the landslide clays in Shikoku. In this paper, certain attempts were made to analyze the strength properties of landslide clays based on the influence of expansive clay minerals such as smectite and expansive chlorite.

The peak effective angle of shearing resistance for the lab samples of landslide soils were measured to be 20° to 40°, and a significant drop of as high as 20° at the residual state was seen. Moreover, the drops from peak to residual values were not seen to have followed the increasing pattern with the increase in plasticity index. Mineralogy of the tested samples, on the other hand, was found to have significant influence in the residual strength of the tested samples. Semctite was detected in most samples but with great variation in the proportion. Samples composed of smectite were found to have lower angles of friction than those composed of non-expansive minerals. An analysis of angles of internal friction and relative amount of smectite in the tested samples showed a clear drop in the former with the increase in the latter. This implies that the angle of residual friction decreases with the increase in the amount of smectite minerals, and even an insignificant variation in the inclusion of expansive clay minerals significantly changes the strength properties. Finally, most fracture zone landslides, which have occurred in metamorphic deposits composed of black and green schist and sedimentary deposits composed of sandstone and shales, are probably active in creeping phase due to the influence of smectite like clay minerals.

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