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Influence of clay mineralogy in residual shear strength of soil

by
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Abstract

Thirty four samples were prepared with different combination of bentonite, kaolin and standard sand. Those samples were tested in Imperial College type ring shear machine. Residual ϕ had shown good relationship with consistency limit, especially liquid limit and plasticity index. There was distinct upper and lower bound in the relationship curve, for corresponding samples dominated by bentonite and kaolin respectively. Likewise, residual ϕ of the soil samples from landslide area and other 40 samples collected from debris flow area, slope failure area and volcanic eruption area had also shown good consistency with the data for sand-bentonite-kaolin mixture. Particle crushing along the shear zone, confirmed by particle size and mineralogical analysis of post shear samples, caused the increase in shearing zone thickness along with the displacement. Liquid limit of the sample from shearing zone was about 20% higher than the initial sample for the samples from main scarp of landslides and other disaster area. Measured fully softened/critical ϕ did not vary considerably with residual ϕ. However, fully softened shear intercept (c) was almost double of the residual c. Residual c was decreased with the increase in displacement. Residual shear strength from ring shear test and corresponding clay mineralogy was used to make contour of residual ϕ, based on the proportion of smectite and kaolinite, plotted as two sides of a triangle, the third side of which represents the proportion of non clay minerals. There was less than 7 % variation in the estimated and measured residual ϕ for the natural soil samples. Seven different equations for the estimation of residual ϕ with smectite proportion have been proposed, corresponding to 7 zones in the triangular relationship chart.

Keywords : *Residual shear strength, Ring shear test, Clay mineralogy, Consistency limit, Slickensides*

1. Background

Although slope stability analysis is very important for countermeasure planning and evaluation of landslide mitigation works, no single method is globally accepted in practice. Many countries still ignore the need of soil testing for the shear strength of soil during stability analysis of landslides and assume it by empirical formulas. Practitioners in some countries assume mobilized cohesion (c) according to the average depth of sliding surface whereas some others assume zero (for highly plastic clays). Internal friction angle (ϕ) is calculated by back analysis, using those values of c for limiting equilibrium condition. As the difficulty to locate exact sliding surface, ground water table and position of different formation layers in preexisting landslides bring serious errors in stability analysis, which

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itself has to be covered up by the calculated value of ϕ, such empirical assumption for c further enhances the magnitude of the error. Hence, it is necessary to study the appropriate shear strength that is to be assigned for the analysis of preexisting landslides. Frequent modifications in ring shear device to have unlimited shearing by Bishop et al. (1971), Bromhead et al. (1979), Sassa et al. (1989) had enhanced the possibility of more precise shear strength measurements. However, shear strengths are still assigned by empirical formulas without shear testing. Skempton (1985) mentioned that the value of ϕ at shear displacement after fully softened stage, does not vary considerably. Moreover, all professionals now, agree that residual ϕ is dependent on the clay mineralogy. Therefore, it is obvious that the assumption of the magnitude of c according to depth is neither an appropriate practice nor justifiable. Many researchers believe that the shape of failure envelopes for plastic clays is curved. However, no one had demonstrated it by the shear test with very small normal stress, for example less than 50 kPa. In many cases, shear envelopes below that range are simply joined to the origin. As residual ϕ is mainly controlled by the shape of the clay minerals, it is obvious that residual ϕ can be better explained by the mineralogical composition of the soil. Our experience shows that there are many landslides triggered along the mudstone or shale formations. One of the dominant minerals of tertiary mudstone is smectite, which is supposed to have very small residual ϕ due to the shape of clay platelets. Besides, mudstones contain kaolinite too as another dominating mineral. Therefore, the research on the relationship between residual ϕ and the proportion of smectite / kaolinite based clay minerals would be helpful in the understanding of residual ϕ. Twenty four soil samples were collected from mudstone dominated landslide areas in order to work out the relationship of residual ϕ with mineralogical composition as well as other index properties. To have wide range of residual ϕ, soil samples were collected from 3 slope failure areas, 4 debris flow areas and 4 volcanic eruption areas too. Soil testing procedures, test results and their importance in stability analysis of landslides are explained underneath.

2. Review of the past literatures

A number of research papers on residual shear strength have been published since late 1930s. After the reports by Hyorslev (1936, 1939) and Haefeli (1951), many researchers have frequently published their works on residual shear strength of the soil. Skempton (1964) had argued on the term residual shear strength with some experimental data based conceptual relationship model. Borowicka (1965), Chandler (1966, 1969), Kenny (1967) and Lagatta (1970) had added few more steps on the works of Skempton. However, the invention of new ring shear test device by Bishop et al. (1971), generally known as 'Imperial College type ring shear device' brought revolution in the history of ring shear testing and had enhanced the possibility of easy and endless shearing of the soil samples. Chattopadhya (1972), Voight (1973) and Kenny (1976) had separately argued the factors influencing the residual ϕ of the soil. They have compared the residual ϕ with index properties of soil. Lupini et al. (1981) had published the research result based on the drained ring shear tests of natural clays and bentonite-sand mixture, using Imperial College type ring shear device and postulated the relationship between clay fraction and residual ϕ. They discussed on the existence of three types of shearing modes - shearing, transition and turbulent depending on the proportion of clay. Skempton (1985) had compiled most the works on residual shear strength until that time in his reproduced paper of special lecture. One of his main conclusions was that residual ϕ does not decrease considerably after fully softened shearing stage. Mesri (1986) had gone through different approaches to estimate the relationship between residual ϕ and other soil properties. He compiled the works published by other researchers too and postulated the new relationship between residual ϕ and consistency limits as well as clay fraction. Colotta (1989) gave the new relationship terminology-calip, combining consistency limits and clay fraction. He had also published the relationship of residual ϕ with other soil properties. Moore (1991) had conducted the research on pure clay minerals and postulated the impact of pore water chemistry on residual shear strength of the soil. His works were mainly concentrated on pure clay minerals whereas most of the natural soils are mixture of various minerals. Considering the concept of curved failure envelope, Stark (1994) had proposed the relationship curve for liquid limit and residual ϕ for different clay fractions and normal stresses. However, he had not considered the mineralogical aspect, which is one of the most important parameter while estimating the relationship of residual ϕ with other soil properties. Yamasaki (2000) had measured the residual shear strength of pure clay minerals-montmorrilonite, chloride, serrilite and illite and their mixtures. However he did not mention its direct effect on the natural soil samples, nor postulated any mutual relationships.

At present, every one agree on the dependency of residual ϕ with the shape of the clay mineral. However, there are extremely limited researches on the impact of the mixture of minerals with different shaped platelets on residual ϕ. For example, landslides triggered in mudstone and shale are common in many countries. As the mudstone have smectite and kaolinite as dominating clay minerals, their residual soil also have the dominancy of these minerals. Therefore, study on the relationship between residual ϕ and the proportion of smectite / kaolin would be very beneficial in the estimation of the former. This research is mainly focused on the shearing mechanism of the soil and estimation of such relationships in smectite-kaolinite dominated soils. Soil samples were collected from more than 35 sampling areas and 34 samples were prepared in the laboratory with various ratios of bentonite and kaolin by weight with sand. Soil sampling areas, test results and the discussion on the research findings are explained in the following parts of the paper.

3. Soil sampling areas

Soil samples were collected from Niigata, Fukushima and Hokkaido Prefectures of Japan (Fig 1). As the nature of disaster are different, proportion of clay in the soil samples were also different. Sampling landslide areas were Okimi, Yosio, Mukohidehara, Engyoji, Iwagami and Tsuboyama, all of them located in Niigata Prefecture. Soil samples were collected from deeper sliding zones and near the main scarps to observe the variation in shear strength at deep and shallow locations of the same sliding block. Soil samples from the fresh slope failure scarps were collected from 2 slope failure areas : Karamatsu and Habuto, situated in Fukushima Prefecture, in order to observe the characteristics of medium plasticity soil. Four soil samples from fresh volcanic ash deposition

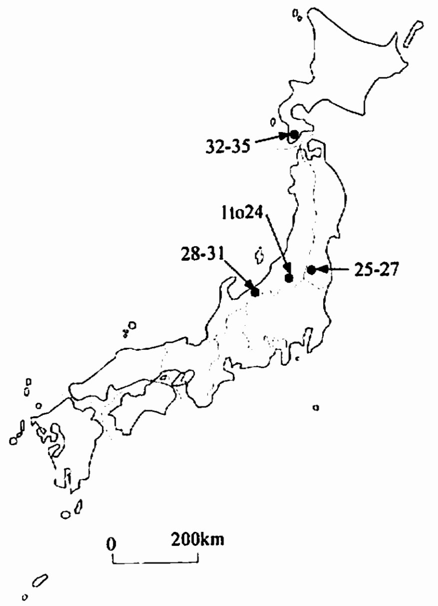

Figure 1 : Location map of the soil sampling points

as well pyroclastic flows in Nishimayama and Kompirayama volcanic eruption areas of Mount Usu, Hokkaido, Japan were also taken in order to observe the characteristics of medium plasticity soil from volcanic area. Four soil samples were collected from the stream banks of Kanayamazawa debris flow area in Niigata prefecture, in order to observe the characteristics of low plasticity soils. The dominating rocks of the disaster areas were also different. All landslide areas had mudstone with small intrusions of sandstones. Slope failure areas had tuff as main rock. Likewise, volcanic areas and debris flow areas were dominated by basaltic lava and andesites respectively. Mineralogical analysis of all soils were done in order to batch the artificial samples, mixing various proportions of dominant minerals.

4. Mineralogical composition of soil samples

Mineralogical composition of the soil samples were measured by x-ray diffraction method. Soil samples extracted from the shearing zones after ring shear test were powdered and set in to the x-ray diffraction device. Approximate proportions of minerals were measured from the ratio of major peaks. Minerals having proportions of less than 0.5 % were neglected. Clay portions were separated by settlement and centrifuging in order to confirm the type of clay minerals in the sampled soil (Fig 2). For that, the extracted clay portions were diffracted thrice: at naturally dried stage, heated at high temperature and after the absorption of ethylin glycol. Due to the possible error in setting the actual proportion of representative sample in to the test cartridge, at least five samples were diffracted in x-ray diffraction device and average compositions of those samples were considered as the mineralogical proportion. Approximate proportion of dominant minerals in all soil samples is shown in Fig 3. According to the test result, almost all of the samples from landslide areas were dominated with qurtz as the main mineral and smectite (montmorillonite) and kaolinite as main clay minerals. Besides, there were considerable amounts of feldspar too. Samples from Karamatsu were dominated with halloysite, kaolinite, feldspar and quartz but no smectites. As most of the samples had smectite and kaolinite as main clay minerals and quartz as the main dominating mineral, soil samples were prepared in the laboratory with 39 different mixtures of Toyoura sand, commercial

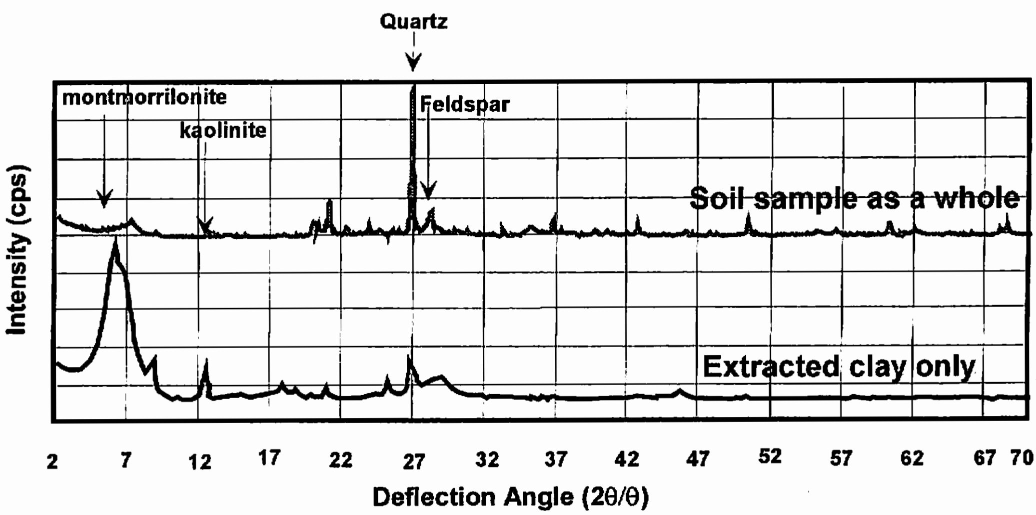

Figure 2 : Typical x-ray diffraction result (Sample 9)

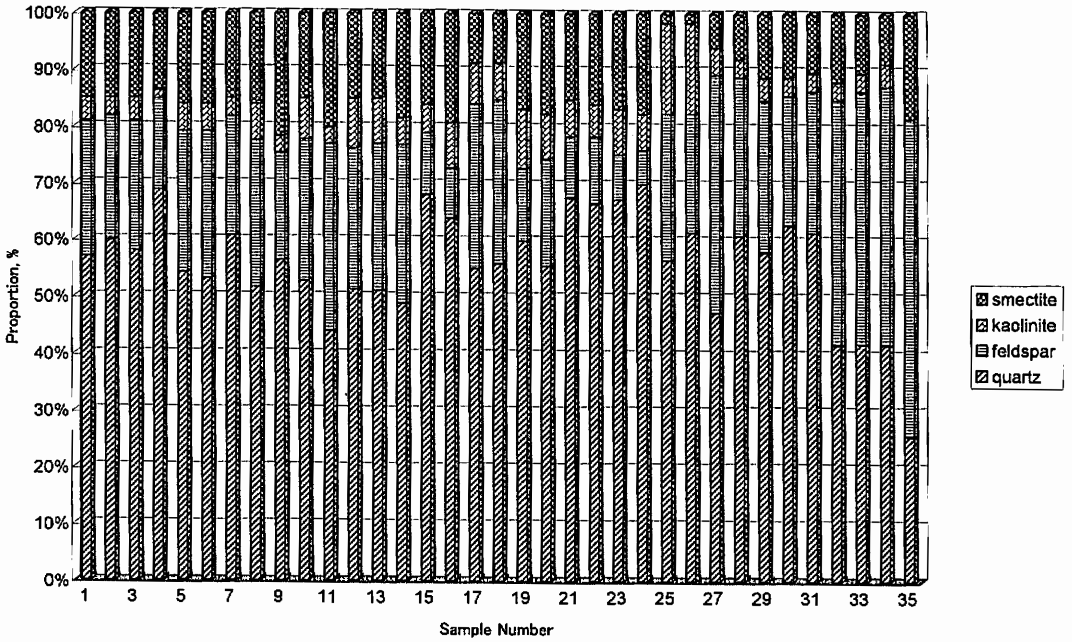

Figure 3 : Mineralogical composition of the tested soil samples

bentonite and commercial kaolin. Commercial bentonite had 80 % smectite, 10 % crystaballite and 10 % quartz whereas commercial kaolin had 80 % kaolin, 5 % halloysite, 5 % feldspar and 10 % quartz. Toyoura sand is the standard sand having 100 % quartz. As there were wide ranged proportions of minerals in the mixture, they were expected to yield wide range of residual ϕ. The relationship curve would be helpful in the estimation of the residual ϕ for various combinations of quartz, smectite and kaolinite, especially in the smectite based parent rocks such as mudstone and shale. Then, all 74 samples were tested in ring shear device for residual shear strength measurement. The results of ring shear and other index tests are explained in the following chapters.

5. Results of the soil tests

The main objectives of the study are to understand the shear zone formation mechanism and comparison of residual ϕ with other soil properties. Therefore, particle size analysis was done for all disaster area soil samples before and after ring shear test. All the soil tests were done according to the Soil Testing Manual, published by Japan Geotechnical Society. Soil samples passing through 2 ㎜ sieve were carefully batched for the ring shear test. The Imperial College type ring shear device (Photo 1), used for the study has outer and inner diameter of 20 and 13 ㎝ respectively. The thickness of soil sample is 4.5 ㎝. The outer rings and water jacket of the apparatus is transparent so that the soil sample can be observed from outside during shearing process. Drainage is performed by the porous stones at the upper and lower platens. The radial fins, fixed to upper and lower platens provides the grip between the platens and soil samples.

In each batch, 1224 gm dry weight of the soil samples were taken to make the same dry unit weight of 1.5 t/㎥. The soil samples were loaded into the ring shear machine and

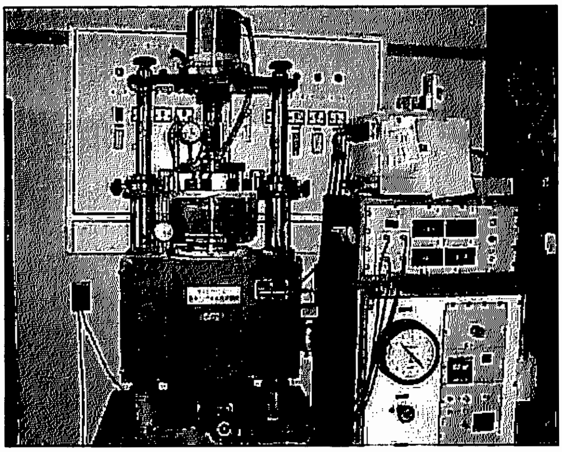

Photo 1 : Modified Imperial College type ring shear device used for the research

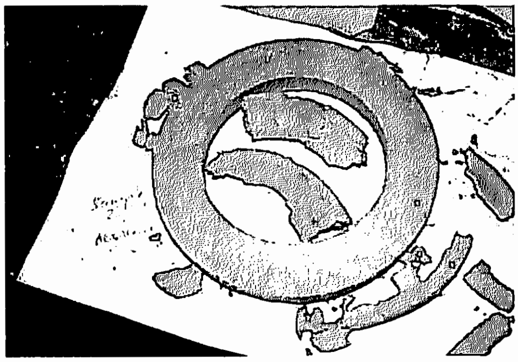

Photo 2 : Annular post shear specimen with slickensides

compacted uniformly in to 5 layers. Those samples were supplied with distilled water from the water jacket and the upper platen. Supplied water penetrated through the upper and lower porous stones. The sample was kept observing water for more than 72 hours until the full saturation was confirmed through the eye observation as well as the water level in the water jacket. The sample was then subjected to the effective normal stress of 250 kPa and consolidated for more than 60 hours until the time-settlement curve shows optimum settlement stage. Then the sample was sheared at normal consolidation with the velocity of 0.016 mm/sec. Stress-displacement-settlement data were automatically recorded through out the shearing process. All samples were sheared at least for 2000 mm displacement at the initial normal stress as this stage is important to form the slickensides. Then the normal loads were gradually reduced to 6kPa and residual shear strengths corresponding to at least 10 different normal stresses between 250 to 6 kPa were measured by reducing load multi-stage ring shear test. The annular samples (Photo 2) were then taken out carefully at the end of the test to measure the particle size distribution, consistency limit and mineralogical composition in order to compare them with the pre-shearing results.

The shape of the stress-displacement curve (Fig 4) was similar to the curve proposed by Skempton (1985). Once the shearing displacement crossed the fully softened / critical stage, the decrease in shear torque was very small. Minimum shear torque was observed at the displacement of 300 to 500 mm. Although it showed constant torque for about an hour, further shearing had slightly increased the shear torque. After small displacement, shear torque decreased again to the minimum value, which was smaller than the first minimum. The shear torque was constant for more than 2 hours in this stage. When the shearing was continued beyond this point, the shear torque again slightly increased and decreased gradually to the third minimum value, smaller than the second one. This shear torque, which has been assumed as residual shear strength, remained constant for more than 4 hours. As a result, clear shear zones of 1 to 2 mm thickness with a number of slickensides were formed (Photo 3). Pattern of stress-displacement curve and residual shear strength envelope have been presented in Fig 4 and Fign 5-7 respectively. In order to complete the concept of Skempton on the difference between the softening and residual ϕ, shear strengths at intermediate minimum points were also measured. Although Skempton (1985), with very limited experimental data, mentioned that value of softening and residual ϕ does not vary with more than 10 %, he did not

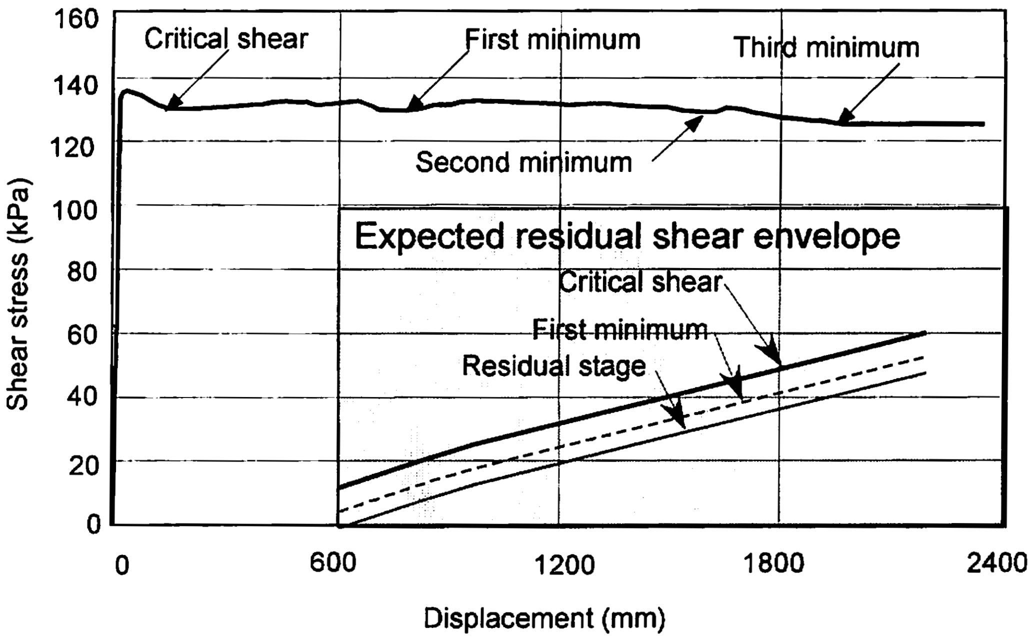

Figure 4 : Typical stress-displacement curve and expected shear envelope

mention about the value of c at those stages. Present research result shows that the value of softening and residual ϕ does not vary by more than 10 %, as proposed by Skempton (Fig 8). However, the value of c was gradually decreased with the increase in displacement (Fig 9). As the shape of the shear envelopes were curve for lower stress ranges (although the natures of the curves were different for each sample), magnitude of c was measured with linear regression of whole data whereas value of c was estimated with the linear regression of the data for less than 50 kPa normal stress.

Except in the case of the samples from the sliding zones of landslide areas, the soil samples at the shear zone after ring shear test (Photo 4) had shown substantial increase in fine particles compared to the initial propor-tions (Fig 10). Interestingly, the proportion of fines at the deeper sliding zones of landslide areas and shearing zone soil samples of the corresponding soil near main scarps after ring shear test were similar (Fig 10). The propor-tions of fines outside the shear zone were similar to the initial proportions. It clearly shows that the major

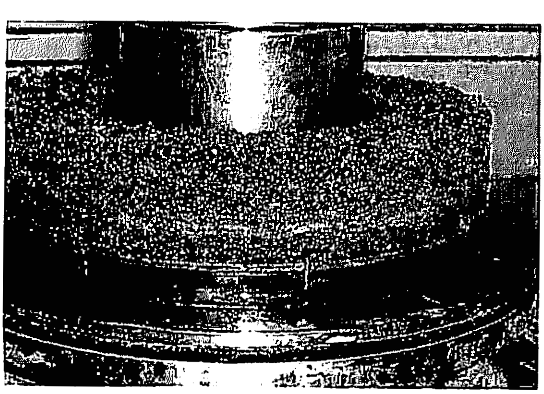

Photo 3 : Accumulation of fines along shear zone which yielded lower c

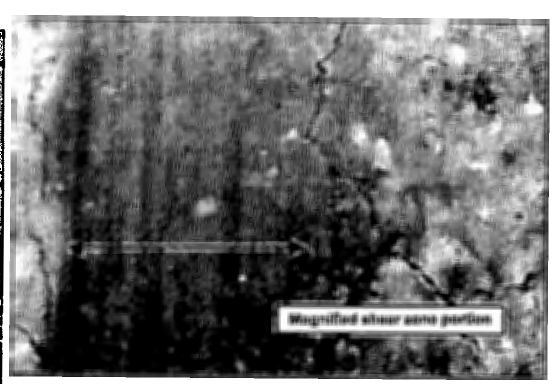

Photo 4 : Enlarged shear zone, showing the shear band, taken by 75M microscopic camera

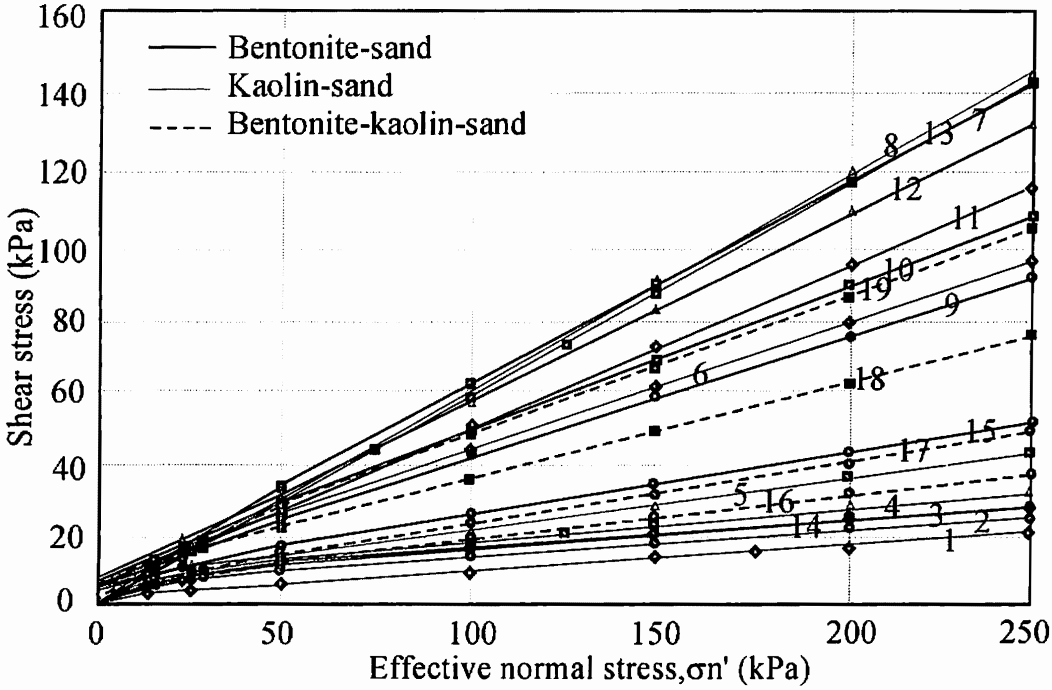

Figure 5 : Residual shear envelope of bentonite-kaolin-sand mixture

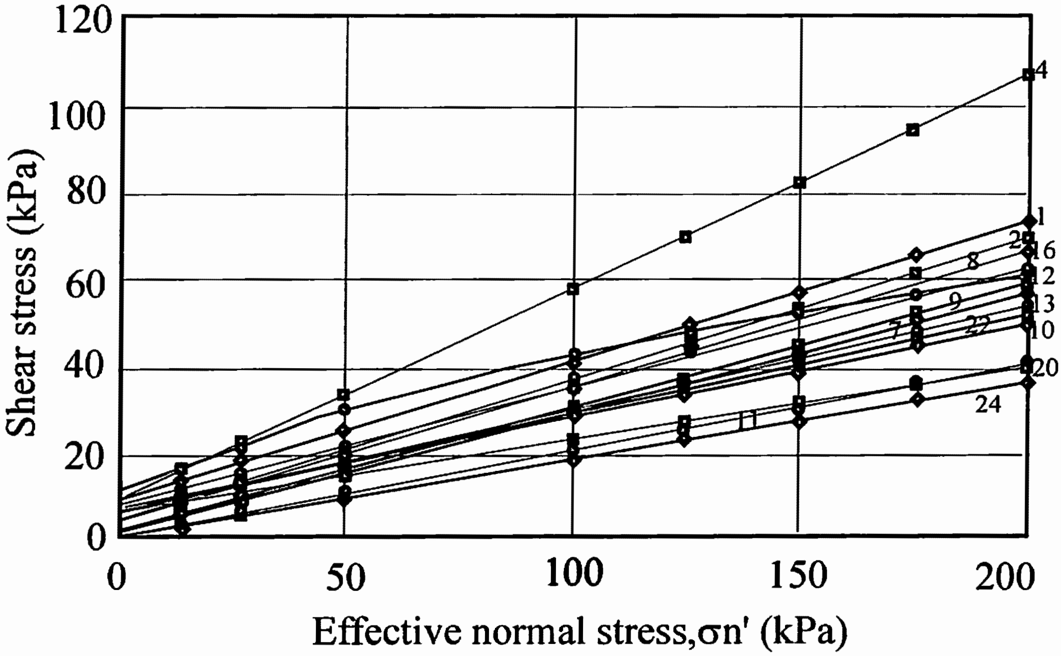

Figure 6 : Residual shear envelope of the soil samples from landslide area

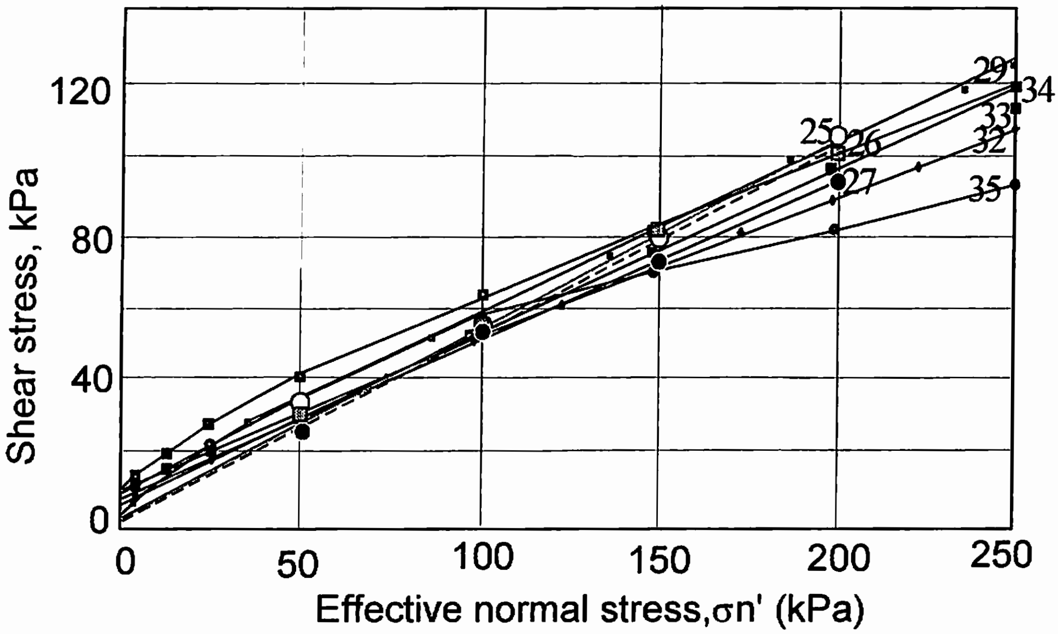

Figure 7 : Residual shear envelope of the soil samples from slope failure, debris flow and volcanic area

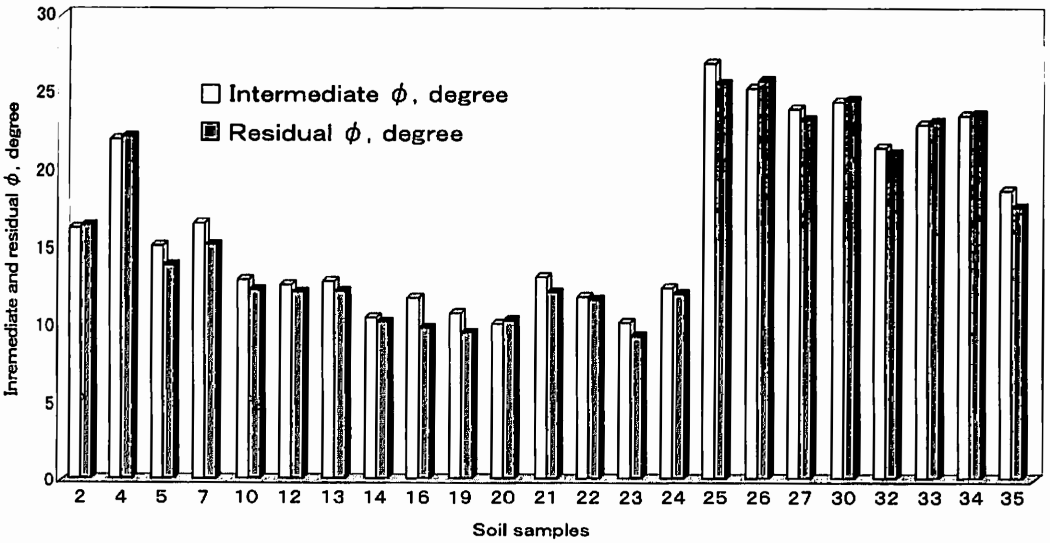

Figure 8 : Intermediate and residual friction angle for natural soil samples

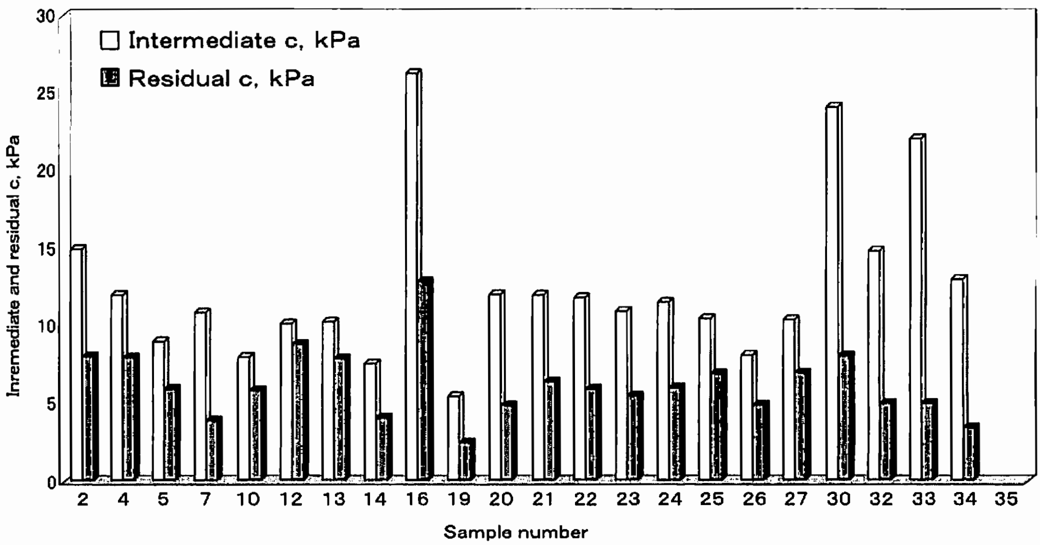

Figure 9 : Intermediate and residual shear intercepts of natural sol samples

portions of the fine materials at the shear zone were yielded due to crushing of the soil particles and detachment of the cemented clay from the surface of the coarser particles due to continuous shearing. As the soil from the sliding zone of landslide area had already been sheared, there were not further detachments or crushings.

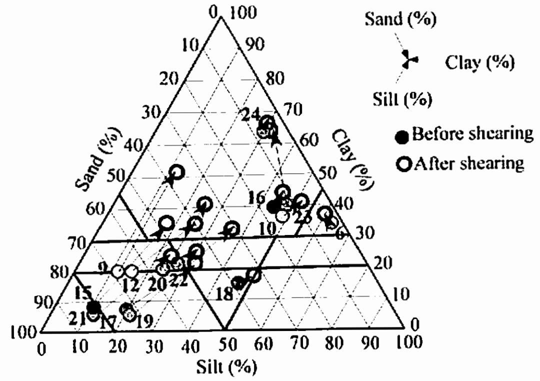

Figure 10 : Textural classification of few soil samples before and after ring shear test

Again, except in the samples from the sliding zones of landslides, the samples from the shearing zone after ring shear test had shown 15 to 20 % increase in liquid limit. However, plastic limit was more or less constant. This is simply due to the increase in surface area of the soil particles in the shearing zone as a result of the increase in proportion of fines. In contrary, there was not so big difference in mineralogical composition of the samples before and after the ring shear test. This might be due to the powdering of the samples before loading into the x-ray diffraction device. As the equilibrium water content was maintained during residual shearing stage, water content of the soil after ring shear test was slightly higher than plastic limit in all the samples. The mechanism of clay size-rich shear zones during ring shear test and their role in residual shear strength is briefed in the discussion chapter.

Another aspect of the research was to study the effect of the smectite and kaolinite proportions in the residual ϕ. The ring shear test results of 34 different batches made by the mixture of different proportions of bentonite, kaolin and sand had given very important hint on the role of individuals minerals in residual ϕ. The shear strength pattern of bentonite-sand mixture, bentonite-kaolin mixture, kaolin-sand mixture and bentonite-kaolin-sand mixture were different. About 50 % bentonite was sufficient to yield minimum residual ϕ in the mixture with sand. Whereas, 30 % bentonite was sufficient to yield the minimum residual ϕ when mixed with kaolin. The combination of kaolin-bentonite proportion and proportion of bentonite combinely controlled the residual ϕ The proportion smectite and kaolinite based on the proportion of bentonite and kaolin had given very good relationship curves. The variation of residual ϕ was negligible below 10 % and over 50 % bentonite proportion. The transition phase had exponential curves, equations of which have been shown in Fig. 11. As the combination of clay minerals changed the characteristics of the soil in this research, estimation through single mineral sometimes may lead considerable error. The most important aspect is to find the domination range of each individual minerals as well as their mixtures with other minerals. Different equations have been proposed for different combinations, as mentioned below.

$$\phi = 49.39 \times e^{-0.066 \times (\% of\ smectite)} \quad \text{for smectite = 8 to 40\%, kaolinite=0\%} \quad (1)$$

$$\phi = 18.76 \times e^{-0.0510 \times (\% of\ smectite)} \quad \text{for smectite < 24\%, kaolinite+smectite<40\%} \quad (2)$$

$$\phi = 44.7 \times e^{-0.103 \times (\% of\ smectite)} \quad \text{for smectite = 8 to 24\%, kaolinite+smectite=28 to 40\%} \quad (3)$$

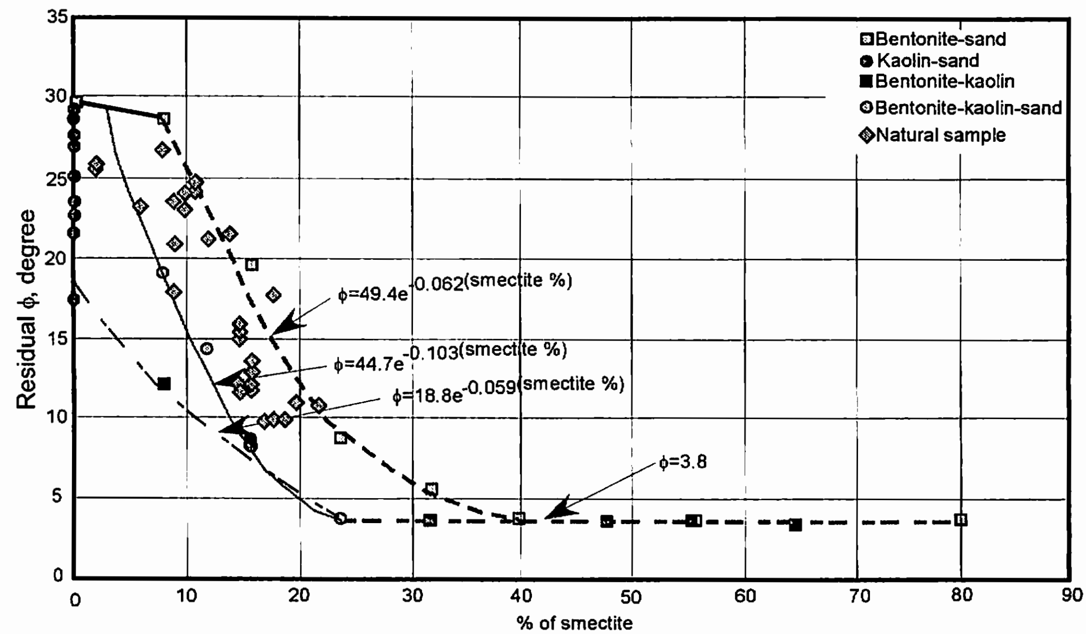

Figure 11 : Relationship between smectite proportion and residual ϕ for mixture

The test results of the mixtures and relationship of index properties with residual ϕ could provide triangular relationship curve for residual ϕ, three sides of the triangle representing the proportion of smectite, kaolinite and other minerals (mainly quartz) respectively (Fig 12). 3D analysis software (ArcGis 8. 1) was used to make the iso-residual ϕ lines based on the test results of the mixtures. X and Y coordinates were plotted with the coordinate relationship of smectite and kaolinite as shown in equation 4 and 5. Five different relationship regions could be observed (Fig 12b). Proportions of smectite, kaolinite and other non-clay minerals of the natural soil samples were plotted in the relationship triangle and residual ϕ were estimated. Estimated residual ϕ had less than 7 % difference with the tested value except

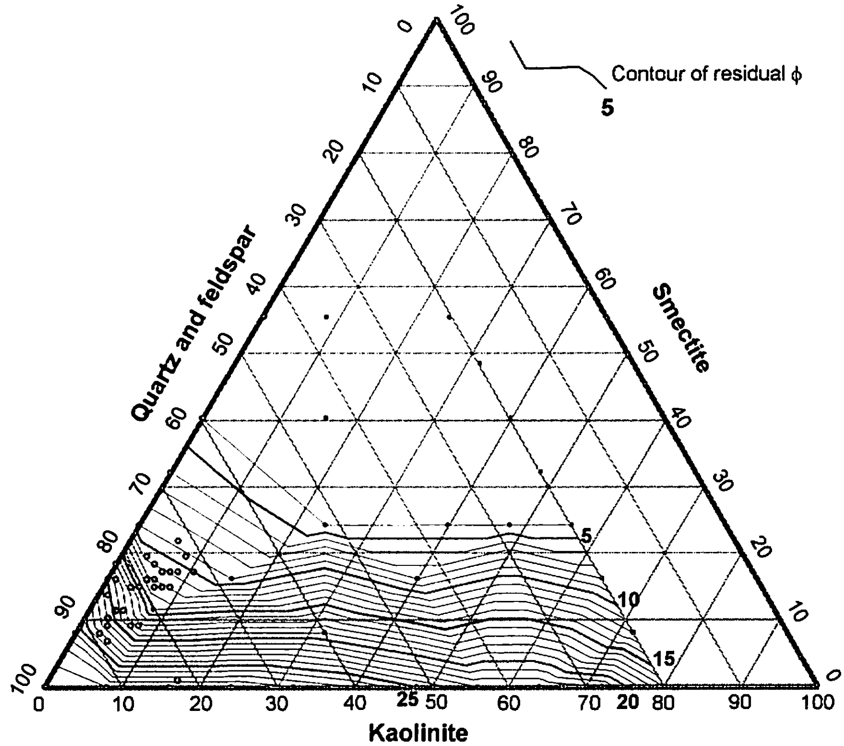

Figure 12a : Proposed contour of residual ϕ based on mineralogy

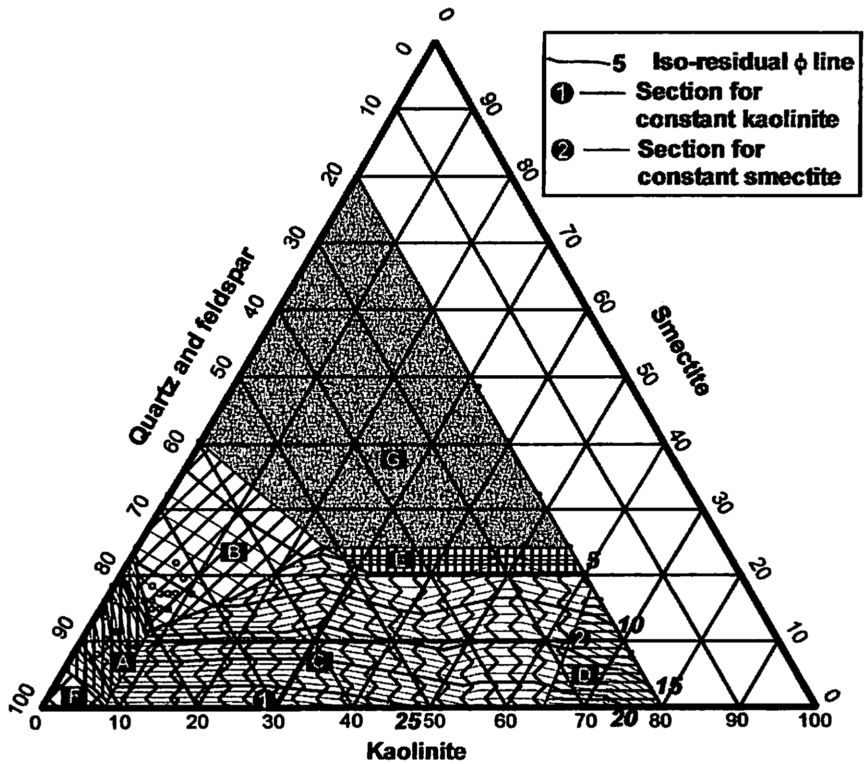

Figure 12b : Proposed zones with different nature of ϕ

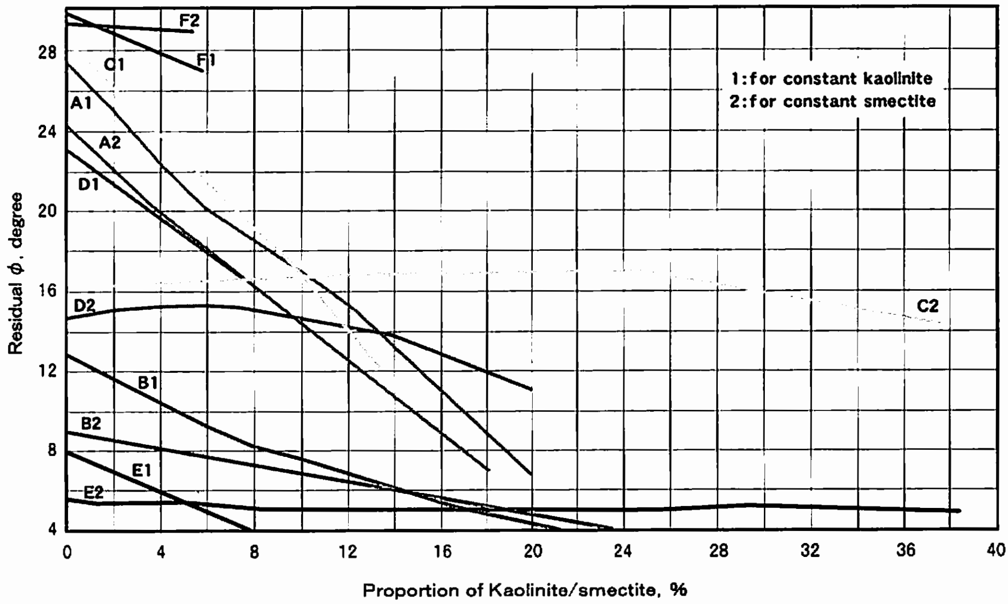

Figure 12c : Cross section of residual ϕ for constant smectite or kaolinite

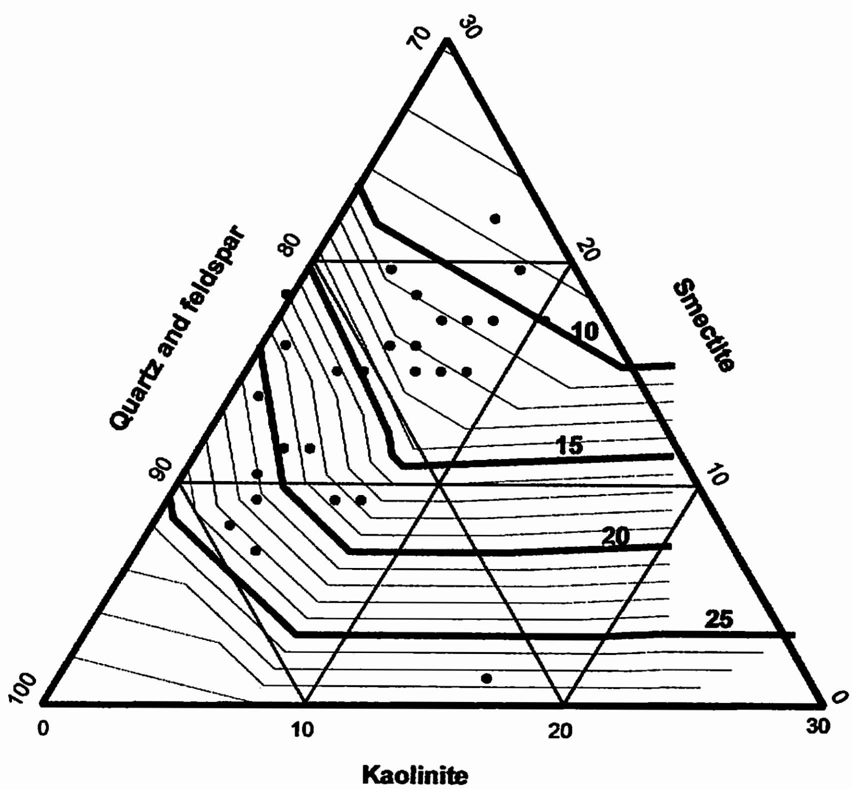

Figure 12d : Magnified portion of zone A and B, which covers majority of the tested natural soils

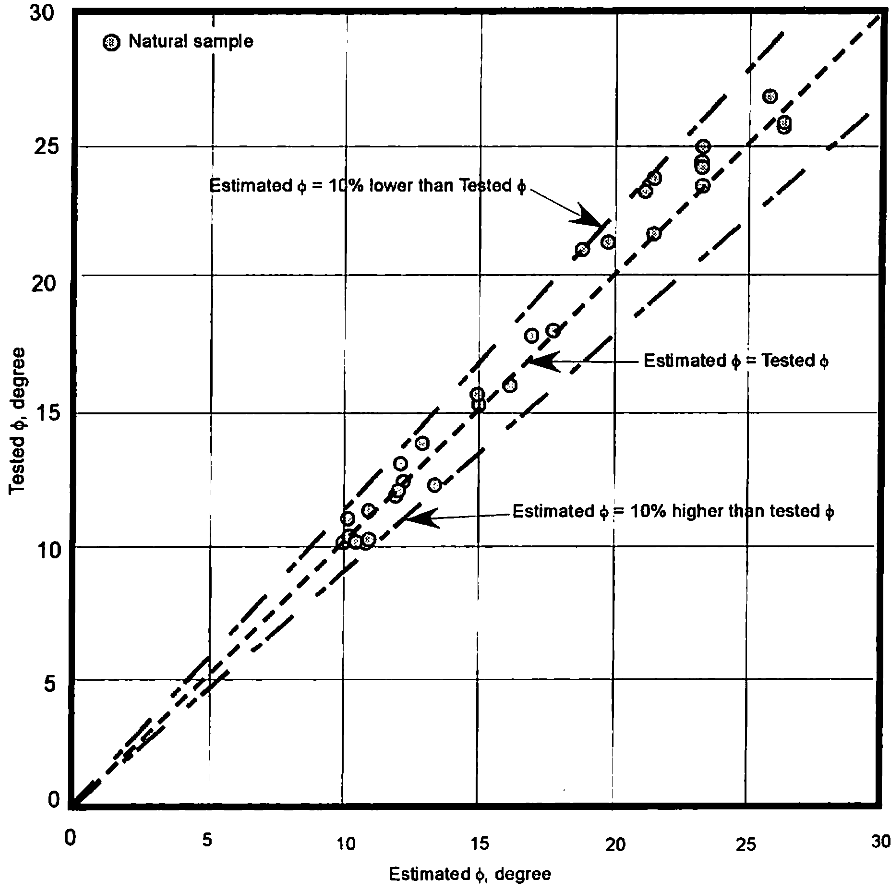

Figure 13 : Variation between tested and estimated residual friction angle

in 3 samples (Fig 13). This will help the researchers to verify their tested residual ϕ with mineralogy or estimate it based on the mineralogical composition.

$$X = \%\ of\ smectite \times Cos60^{\circ} \qquad (4)$$

$$Y = \%\ of\ kaolinite + \%\ of\ smectite \times Sin60^{\circ} \qquad (5)$$

One interesting result of the research is that mineralogy of soil near main scarp and deeper sliding zone of same landslide block in all landslide areas was similar. And the residual ϕ of those locations were also close to each other. This is notable outcome of this research as it shows that residual shear strength of the soil near main scarp can be considered for stability analysis in the absence of deep shearing zone soil, provided the mineralogical composition of the soil samples is similar. Quantity of drilling core revealed sample is enough for the x-ray diffraction of the soil samples although it is not sufficient for ring shear test.

6. Discussion on the shearing mechanism of soil

As natural soil is the composite mixture of various soil particles and many fine particles might be cemented strongly to the surface of coarse particles, sieve and hydrometer analysis only may not give the exact proportion of clay size fractions. This, however, is not the problem in the laboratory batched soil samples to have mixture of various pure materials. During ring shear test, there is continuous shearing among the wet soil particles. This may cause the detachment of finer particles attached at the surface of coarser particles. Besides, the corners of the coarser particles gradually break down to form relatively more equilibrium shape. The crushed soil particles, which are finer than the original particles accumulate along the shear surface and make the shearing zone (Photo 4). All the clay platelets of the platy minerals like smectite align parallel to the shearing surface to make slickensides. Those platelets piles up one over the other and skid over each other during shearing. Rotund particles, however, rotates against each other or over the aligned platelets along the slickensides.

The aforesaid shearing mechanism was expected for the mixtures. In agreement with the concept, crushing of the sand particles along the shearing zone was evidenced at the shear zone. Post-shear sample had higher proportion of finer sands than the initial proportion. This yielded residual shear strength of about 29^0 in the sample having less than 10 % clay fraction. The increase in proportion of fines had increased the liquid limit of the shearing zone soil and thus decreased the residual ϕ.

As mentioned by Lupini (1981) and Skempton (1985), sand dominates the residual shear strength in bentonite-sand mixture upto 10 % bentonite i. e. turbulent shear mode. Probable shearing of the polished rotund particles over each other might be the main reason for it. In the other hand, when the proportion of bentonite exceeds 50 % i. e. shearing mode, total volume of the bentonite and water becomes considerably high (as liquid limit of bentonite is about 500 %) to embed the minute sand particle inside the smectite platelets along the slickenside. As one swelled smectite platelet slid over the other, they yielded the residual ϕ corresponding to smectite. Mixed effect was observed when the proportion of bentonite was between 10 to 50 %. Very high water absorption capability of bentonite is the reason for exponential decrease in residual ϕ with the increase in bentonite during transition mode. Both rotational and sliding mechanism takes place in this phase. Similar concept works in bentonite-kaolin mixture. Due to the different shape of kaolinite platelets than smectite platelets, the liquid limit of kaolin is considerably low compared to bentonite. This phenomenon was responsible for low water retention in case of kaolin. This caused kaolin to have considerably high residual ϕ than bentonite. The mixture of kaolin-bentonite-sand therefore had mixed effect. Sand particles could be imbedded when proportion of both kaolin and bentonite was more than 60 % with proportion of bentonite more than 30 %. Besides, if the proportion of sand particles is high, it obviously influence the residual ϕ. This makes easy to come up with the general relationship for different combination zones of residual ϕ-index property curve (Figs 14, 15). The estimation of residual ϕ was, therefore, possible with the proportion of smectite. The cross check at both liquid limit-residual ϕ (Fig 14) curve and smectite-residual ϕ curve (Fig 11) would be the best approach to estimate the residual ϕ of the soil. Due to the equilibrium water content at residual stage, water content after ring shear test was close to plastic limit (Fig 16).

Most of the researchers are bias to start the shear envelope from the origin, irrespective of shearing history. In case of highly plastic soil, all the clay platelets skid over each other to yield zero residual c. However, if the proportion of big sized particles, which have attached clay particles at the surface is considerably high, alignment of clay platelets is interrupted by the sand particle, which yields measurable amount of c. However, if the

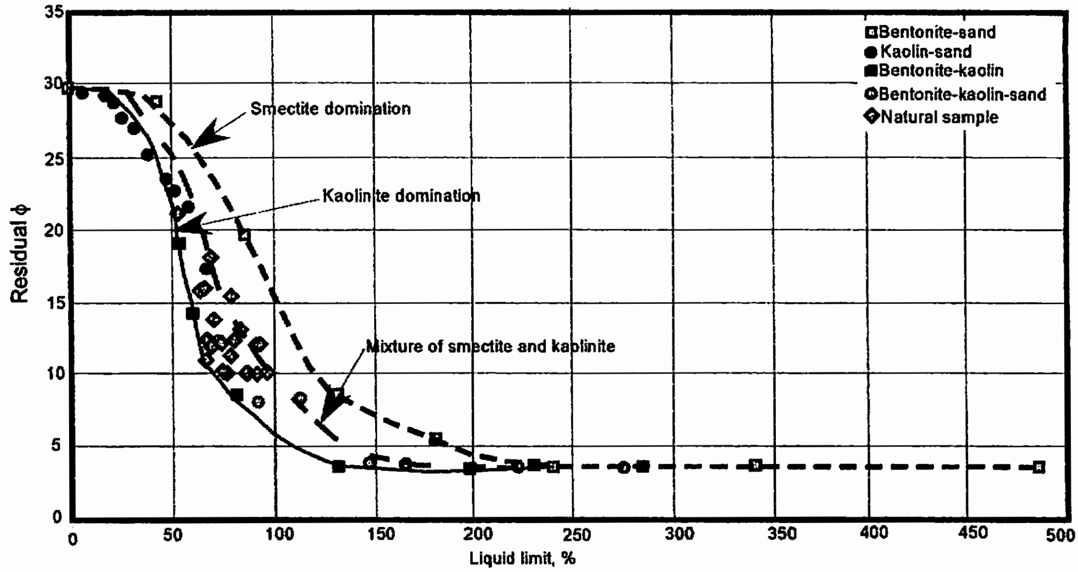

Figure 14 : Relationship between liquid limit and residual ϕ

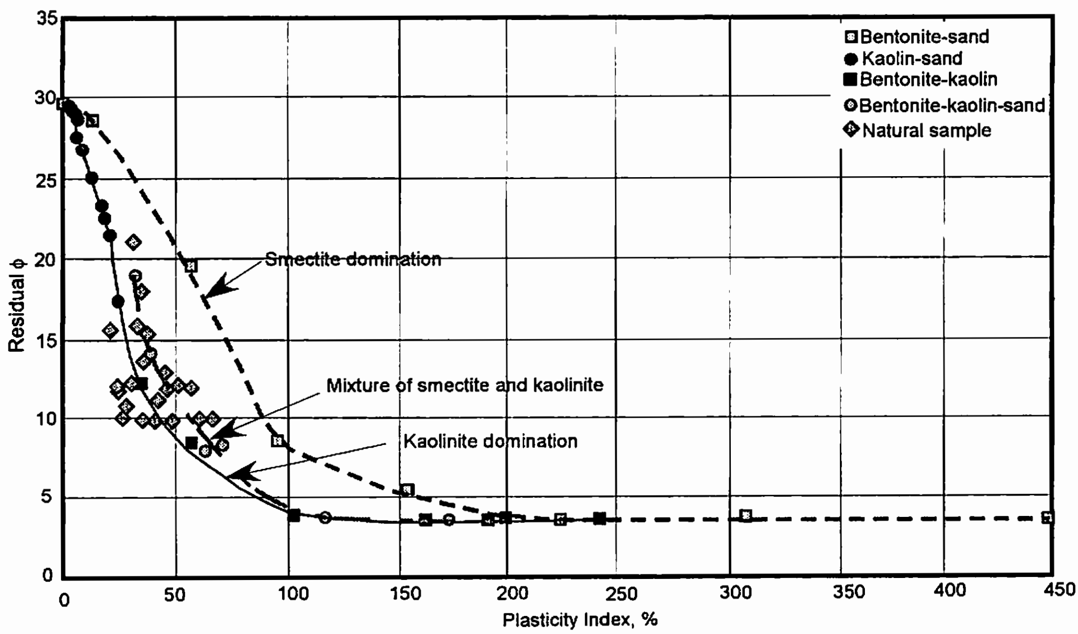

Figure 15 : Relationship between plasticity index and residual ϕ

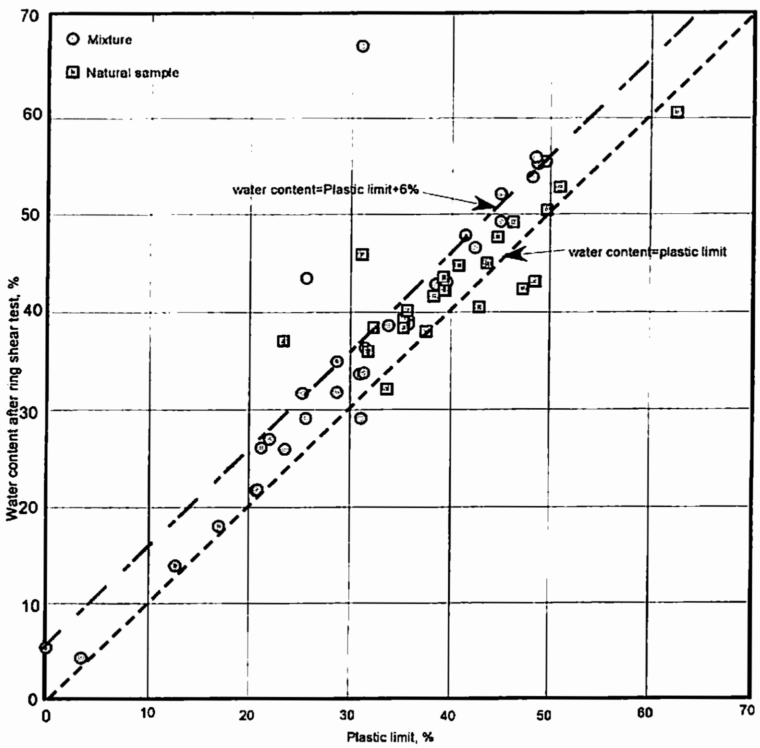

Figure 16 : Relationship between plastic limit and water content after the test

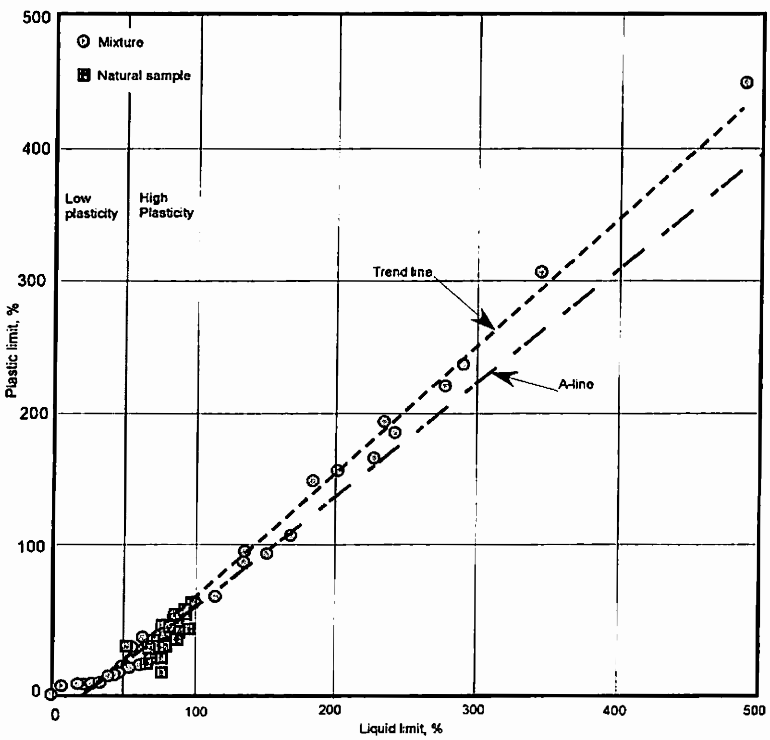

Figure 17 : Post test plasticity chart of the mixture and natural soil samples

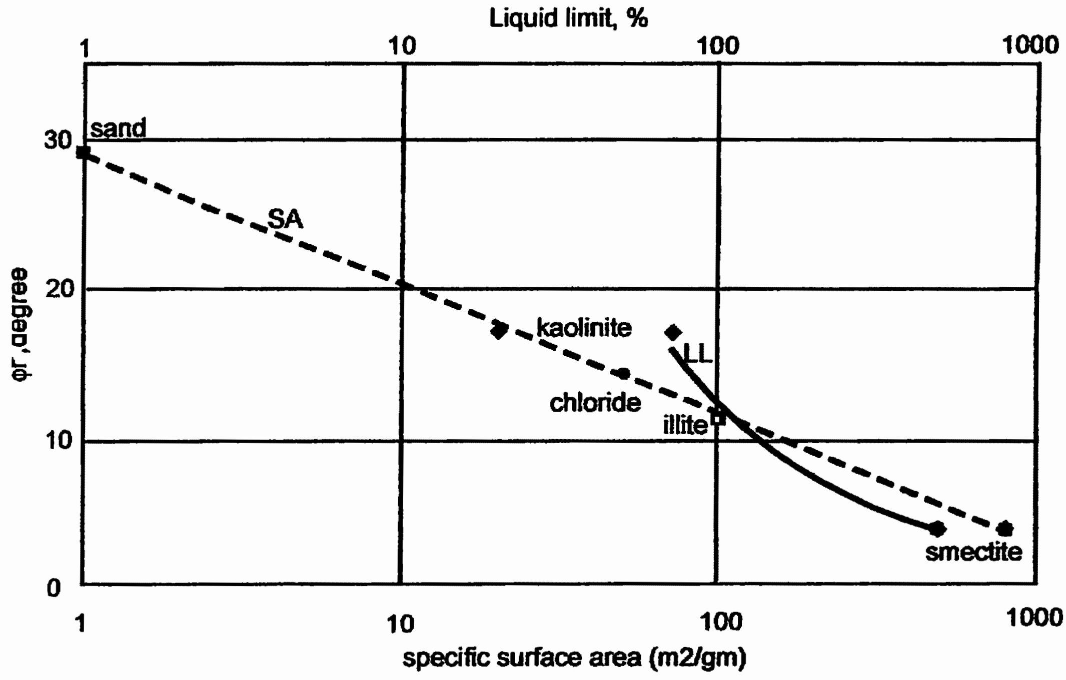

Figure 18a : The specific surface area and liquid limit of pure minerals has exponential relationship

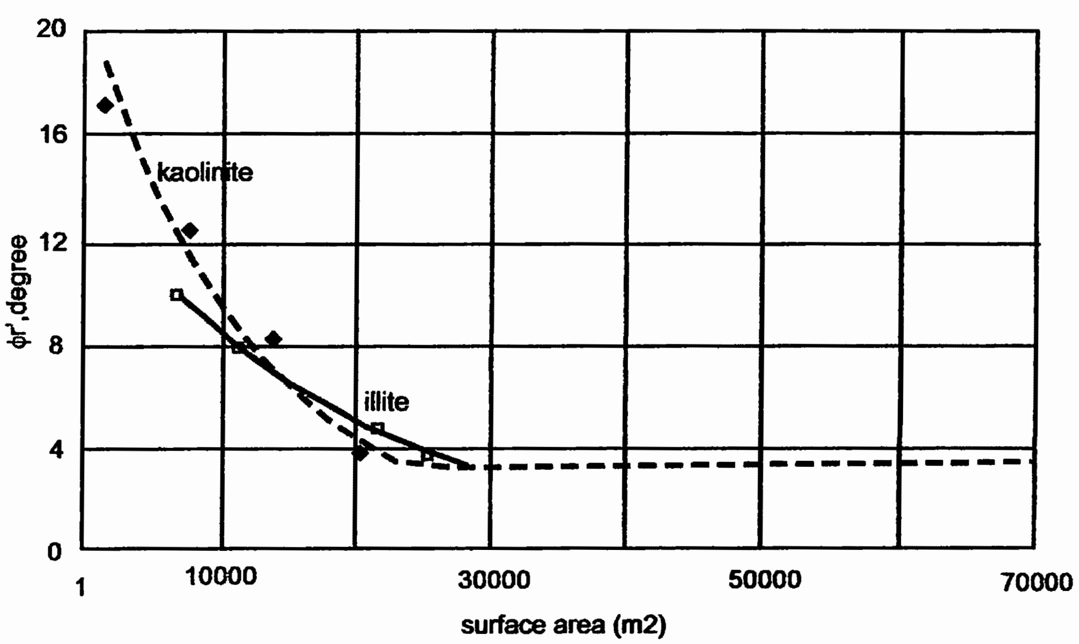

Figure 18b : The mixture of kaolinite and illite with smectite also has exponential relationship with ϕr

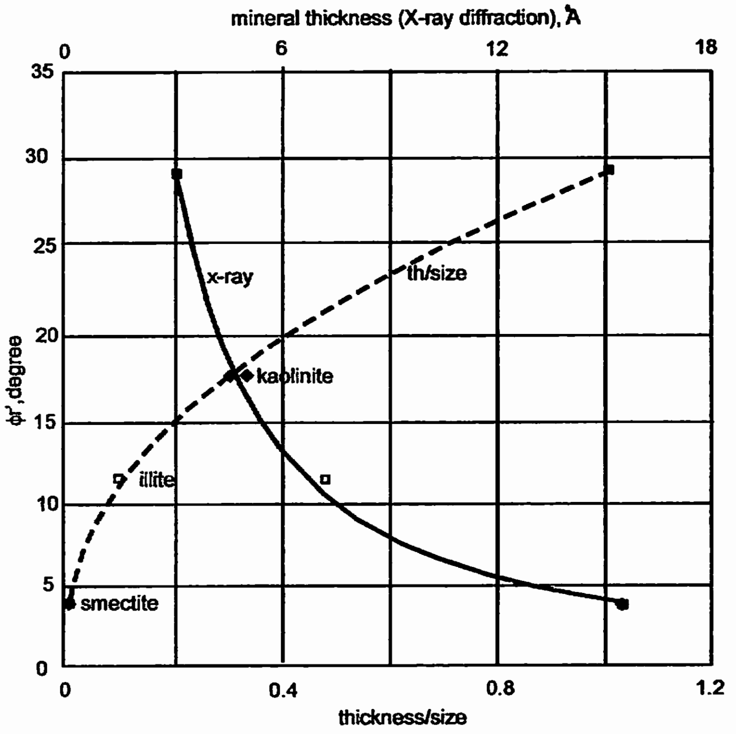

Figure 18c : There is good relationship between thickness of individual mineral and stable mixture with residu

displacement amount is sufficiently high so that almost all clay particles uniformly align to each other completely along the slickensides, residual c can ultimately be zero. As the value of residual ϕ is mainly controlled by the proportion and type of clay minerals, the shape of shear envelope shifts down in parallel fashion along with the displacement amount (Fig 4). Besids, the natural soil samples were located slightly below the A line (Fig 17) in plasticity chart. Therefore, it is not always necessary to have highly plastic samples only. The sliding zone soil samples of the landslide area had also shown the turbulent shearing stage, according to the test result.

7. Conclusion

From the extensive research works and the research output, new arguments on the mechanism of residual shear strength can be proposed, which are as follows.

- Owing to the formation of shear zone, shear intercept of soil samples except the highly plastic clays decreases with the increase in displacement. Therefore, for the highly plastic clays and soil with medium to high clay fraction, residual c can be zero after considerable displacement.
- For the value of actual displacement, value of c can be estimated from the stress-displacement curve and can be utilized in stability analysis.
- Internal friction angle (ϕ) does not decrease considerably after the fully softened / critical stage of shearing. Therefore, in the case of measurement of ϕ only, the test can be stopped after 100−150 ㎜ displacements beyond fully softened shearing stage.

- Residual shear strength of smectite and kaolinite dominated soils can be estimated by the proposed triangular chart as well as the proposed relationship curves, based on the proportions of smectite and kaolinite. Those data should be verified with the consistency limit relationship curves. In case of the mixture of other minerals like illite, chloride, serrilite and so on with smectitte, separate research should be conducted and the relationship curves should be developed.

8. Spectial acknowledgement

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