Researches to Develop the Parallel Boring In-Situ Jack Shear Test "Pabijast"

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平行ボーリング原位置ジャッキセン断試験 "Pabijast" の試作研究

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Résumé

This is the research to develop a new in-situ shear test effective for weathered rock as well as clay. The method is to push the layer between two parallel boreholes dy a plate of the special borehole jack and to shear a pyramid shaped mass.

A trial test of the pabijast was performed at the Kamenose landslide in Osaka Prefecture, Japan.

In the comparisons of the pabijast with 1) the observation of boring cores, 2) the simple shear tests using undisturbed samples taken from about the same place with the Pabijast, 3) a small landslide caused by excavation for sampling and drainage well, the Pabijast presented the values which well corresponds to the geological characteristics including the sliding surface and the ground water surface, and it gave the variation of shear strength similar to the variation of shear strength obtained from the simple shear tests. The values of the papijast (at 1 m, 2 m depth) approximately coincided with the upper yielding values of the simple shear tests which took nearly the same value with the shear strength calculated from the actual landslide.

和　文　要　旨

この研究の目的は粘土にも風化岩にも適用できる新らしい原位置セン断試験機・試験法を開発することであり，その方法は二本の平行なボーリングの間の土層を特殊な孔内ジャッキによって押し切るものである。そしてこの試験法を平行ボーリング原位置ジャッキセン断試験 "Pabijast" と名づけた。

Pabijast 試験機製作後，大阪府亀ノ瀬地すべり地において Pabijast 試験の試用を行った。そしてその試験結果と1)ボーリングコアの観察　2)Pabijast 試験とほぼ同じ場所から採取した 非かく乱サンプルを用いた単純セン断試験　3)サンプリングと集水井のための掘削によって生じた小地すべりを比較検討した結果，Pabijast 試験によって得られた値は地層の差，すべり面，地下水面を明確に反映しており，そのセン断強度分布の形は単純セン断試験によって得られた強度分布と対応しており，かつ同一深度，同一地質での比較が可能であった深度1ｍと2ｍでの

Pabijast 試験の値は，現地で発生した小地すべりから逆算したセン断強度と極めて近い値を与えた単純セン断試験による上降伏値とほぼ一致した。

1. Purpose of this paper

When we examine the stability of natural slope, to measure the in-situ shear strength of layer is desired very much. Hence vane test has been developed for this purpose and used in a clayey ground of Scandinavian countries and England. Vane test was possible to apply for only clay, though recently the interpretation of vane test in sand was proposed by K. Sassa and A. Takei. Anyway it is exactly impossible for a weathered rock. However, weathered rock is a very popular geology in Japan and big landslides are usually composed of weathered rock. Therefore, we started to develop a new in-situ shear test applicable for a weathered rock. This is chiefly the research of hardware of the new in-situ shear test, some researches of software are left to be done.

2. Machine and Method of The Pabijast

The outline of machine and method of the Pabijast are shown in Fig. 1. The method is to shear the layer between two parallel boreholes by a special borehole jack inserted in one borehole. The machine is composed of a borehole jack, two oil jacks, two devices to measure pressure and stroke, and a device to measure the position of borehole center at each depth.

The dimensions of the borehole jack are; the outer diameter is 127mm, the maximum push and pull force is 5000kg, the maximum stroke is 85mm.

The working process of borehole jack is; the two sub-Jacks press the block of jacks on the layer, and the main jack pushes out a plate. After shearing a mass, the sub-jacks and the main jack are drawn into its cover. If something resists return of the plate, the plate is left in the layer.

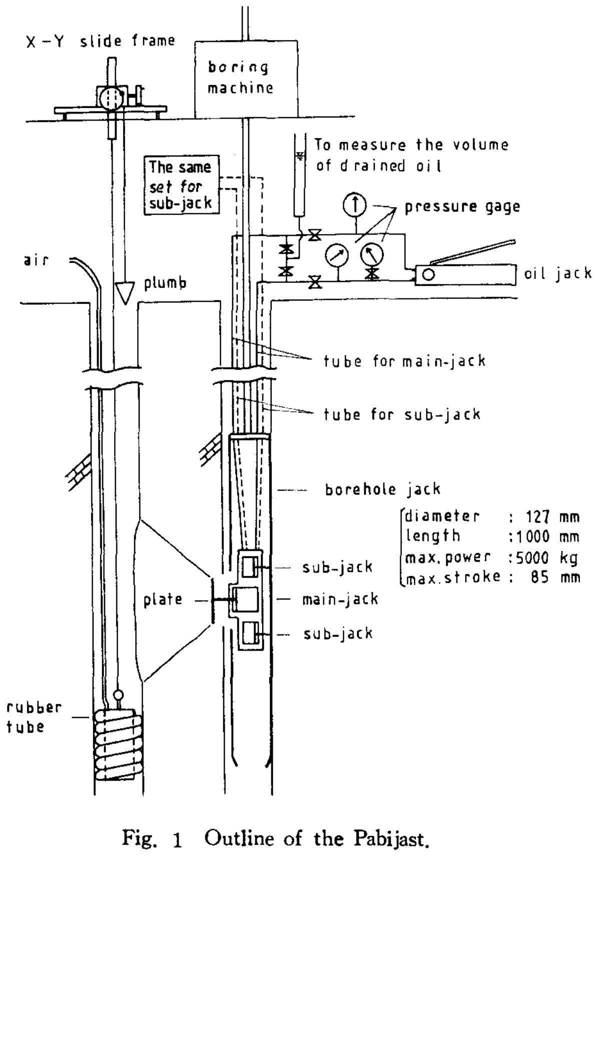


Fig. 1 Outline of the Pabijast.

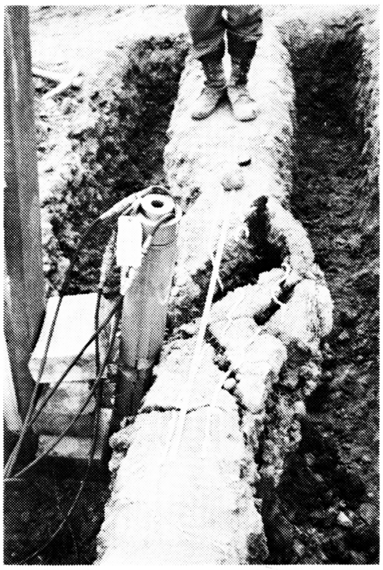
Pressure is measured by two oil gages (one is for small pressure, one is for high pressure). Stroke of the plate is measured by the quantity of oil drained by the movement of piston in borehole jack. The borehole center at each depth is measured by fixing a cylinder with air tube at the borehole center of each depth and deciding the point just vertically above the center by X-Y sliding frame and plumb on the ground surface. The size and ability of pabijast machine can be changed according to the purpose. We will mention the second and the third machine completed already for reference. The second machine is designed to get big strok , the borehole jack is composed of one main jack, two sub-jacks and four sub-sub-jacks. The outer diameter is 112 mm, the max. power is 4 ton, the max. stroke is 108 mm. The third one is the compact type of Pabijast. The outer diameter is 60 mm, the max. power is 2.5 ton, the max. stroke is 38 mm.

3. Preparation of the Pabijast in the Kamenose landslide

We had a trial test of pabijast in the Kamenose landslide where the most expensive works in Japan has been done to stabilize it. We selected a point where the slip plane is not so deep in the Kamenose landslide for the test.

In advance of main test, we performed preparatory tests in tight tuff layer excavated near the point of Pabijast, to decide the appropriate distance between two boreholes, and also the appropriate size of push plate, and to examine the vertical and horizontal shape of sheared mass. Six shallow holes (1 m depth) were bored, their distances between each other are 15-30 cm, Two parallel ditchs were dug also to presume the vertical shape of sheared mass.

Ph. 1 Horizontal shape of sheared mass in the Kamenose landslide (weathered rock)



Ph. 2 Correspondence to vertical shape of sheard mass in the Kamenose landslide

Photo. 1 shows the horizontal shape of sheared mass, the shear planes run from the edges of push plate to the edge of borehole. Ph.2 shows what corresponds to the vertical shape of sheared mass. This shape is a trapezium, their shear planes spread from the edges of plate.

According to these preparatory tests, we observed that the nearer distance between two holes can give the better shear. Then we decided the distance between two boreholes 25cm in consideration of boring technique.

We observed also that, if the size of push plate is not enough in comparison with the distance between two holes, the plate can not make shear, but penetrate itself in the layer. When the distance is 30cm, the plates with the width more than 40cm could make shear, but the plate with the width of 25mm penetrated itself. However, the same plate with the width of 25mm could make shear, when the distance between two holes is 15cm. (Here, the shape of push plate is a rectangle which its lenght is twice the width, which was decided to shear a bigger mass and to prevent the plate from deformation.)

Then we decided the size of plate 70mm in width, 140mm in lenght, which is the maximum size possible to use and enough big for shearing the borehole wall with 25cm thickness.

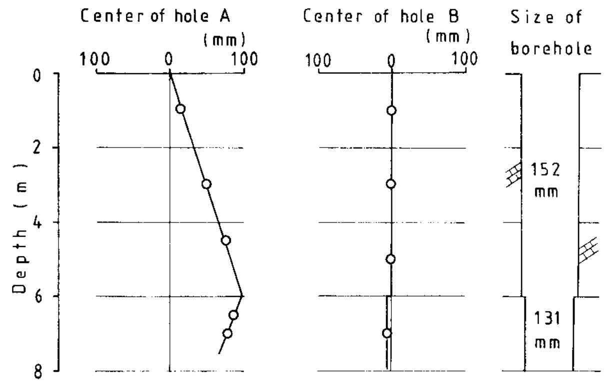


Fig. 2. Position of borehole centre

Table 1.
Thickness of the borehole wall at testing depth

depth (m)	thickness (mm)
0	260
0.8	246
1.0	242
2.0	230
3.0	216
4.0	197
5.0	180
5.7	170
6.3	185
7.0	202

Fig.2 shows the position of borehole center measured by the device. One borehole is vertical, but another one is inclined. Table 1 shows the distance at each depth calculated from Fig. 2.

4. Calculation of Shear Strength

To calculate shear strength from the maximum force of jack measured by oil pressure gage, the area of shear plane, namely the size of sheared mass is necessary. The horizontal section of sheared mass is fixed by both edges of plate and borehole like Ph. 1. But the vertical section of sheared mass is not limited, then measurement of it is necessary.

Now we are designing the device to measure the vectical length of sheared mass, however, we couldn't measure it in this trial test. As a substitute of it, we measured the vertical length and the angle between shear plane and horizontal plane in the test of 0.8m by spreading one borehole by hand so as to observe the deformation of borehole wall directly without any disturbance of shear. Fig. 3 shows the shape of sheared mass at the test of 0.8m.

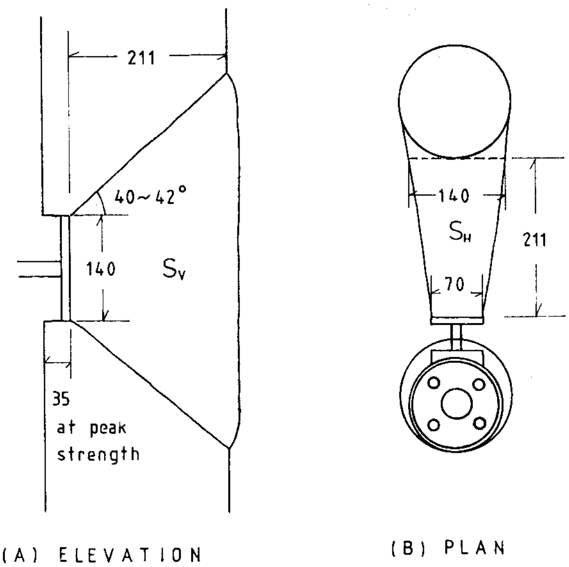


Fig. 3. Shape of the sheared mass at the test of 0.8m

The opening angle of failure plane was 40-42 degrees as Fig.3. We have supposed this angle is 41 degree constant, and decided the vertical section at each depth using this angle and the distance from the plate at the peak value to the opposite borehole. (The plate compacts the borehole wall and penetrates itself to a certain extent before shearing the mass. Then it would be better to take the distance from the plate at the peak value to the opposite hole as the horizontal length of sheared mass.) This assumption to regard the opening angle of failure plane constant may not always be right.
However, it is not unreasonable in this case because the angle of internal friction which chiefly dominates the opening angle of failure dose not change in this ground composed of brown weathered granite debris, blue disturbed granite and the sliding zone between them, which is demonstrated by the results of simple shear tests, Fig. 7 and Fig. 8 though only tuff breccia is left unknown.

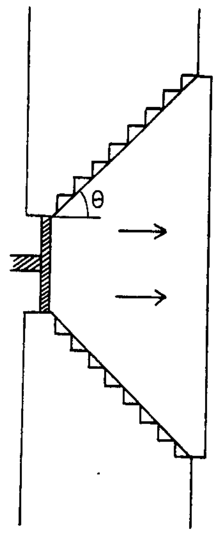


Fig. 4. Division of a shear plane to horizontal & vertical planes

Through the above mentioned process we can estimate the size of sheared mass. Next problem is on the upper and lower inclined failure planes, because the directions of failure planes are not the same with the direction of force given by the jack. Their planes look to be failured partly by shear stress, and partly by tensile stress. Then we divide their planes to the horizontal components failured by snear stress and the verticalco mponents failured by tensile stress like Fig.4 as an approximation. When regarding tensile strength of the layer as negligible and denoting shear strength on horizontal plane as τ_H, the area of a inclined failure plane as S, the total area of horizontal planes in upper half or lower half showed in Fig. 4 as S_H, the opening angle of failure plane as θ, the total resistance force T_H working on inclined upper and lower failure planes is represented by,

$$T_H = 2S_H \cdot \tau_H \qquad \text{here } S_H = S\cos\,\theta$$

The vertical failure plane shown in Fig. 3 is made almost by shear stress. It is only sligntly inclined to the force of jack. Then, its inclination has no practical effect. But it would be better to take the same deal with the former case. When denoting the projection of the area of one vertical failure plane to the direction of jack force as S_V and the shearing strength on vertical plane as τ_V, the total resistant force T_V working on two vertical planes is represented by

$$T_V = 2S_V \cdot \tau_V$$

Then the force of jack T is thus,

$$T = T_H + T_V = 2\ (S_H \cdot \tau_H + S_V \cdot \tau_V) \qquad \cdots\cdots\cdots\cdots(1)$$

In the case of Fig. 3

$$S_V = 211 \times \frac{510+140}{2} = 6.86 \times 10^4 \ (\mathrm{mm}^2)$$

$$S_H = 211 \times \frac{140+70}{2} = 2.14 \times 10^4 \ (\mathrm{mm}^2)$$

The portion of S_H to the total area $(S_H + S_V)$ is 23.7%.
When assuming the isotropic condition, $\tau = \tau_V = \tau_H$ in the ground, Eq. 1 is expressed by Eq. 2

$$\tau = \frac{T}{2\ (S_H + S_V)} \qquad \cdots\cdots\cdots\cdots(2)$$

The assumption of $\tau_H = \tau_V$ is also used in vane test.
In an anisotropic ground, Eq. 2 does not give right shear strength, but still give qualitatively right variation of shear strength in the ground. Hence we will use Eq.2 in this trial test to simplify problems.

However, as the subject of software of Pabijast, to calculate τ_H, τ_V and $\tan\phi$,c is to be studied in the future.
τ_H and τ_V may be measured by changing ratio of S_H and S_V as done in vane test.[4]
And when assuming K_O-state in the ground and using the Jaky's equation of Eq. 3,

$$K_O = 1 - \sin\phi, \quad \text{here } K_O \ : \text{lateral stress ratio at rest} \qquad \cdots\cdots\cdots\cdots(3)$$
$$\phi \ : \text{angle of internal friction}$$

$\tan\phi$, C can be calculated by changing the ground water level (pore pressure) in the broeholes.
Expressing the effective vertical stress and the force of jack before and after the change of water level as σ_{V1}, σ_{V2} and T_1, T_2,

$$T_1 = 2\ \{S_H(C + \sigma_{V1}\ \tan\ \phi) + S_V(C + K_O\sigma_{V1}\ \tan\ \phi)\}$$
$$= 2\ \{C(S_H + S_V) + (S_H + K_O S_V)\sigma_{V1}\ \tan\ \phi\} \qquad \cdots\cdots\cdots\cdots\cdots(4)$$

$$T_2 = 2\ \{C(S_H + S_V) + (S_H + K_O S_V)\sigma_{V2}\ \tan\ \phi\} \qquad \cdots\cdots\cdots\cdots\cdots(5)$$

We can obtain the values of three unknown C,ϕ,K_O by resolving three equations Eq.3, 4, 5. τ_H, τ_V are calculated from C, ϕ, K_O in use of the Mohr--Coulomb equation.

5. Result of the Pabijast in the Kamenose landslide.

Boring for the Pabijast was done without water supply to keep natural water content and not to disturb the borehole. The first boring hole was protected by a casing pipe so as not to be deformed and disturbed while the second hole was being bored. The second borehole was also protected by a casing pipe, in order to avoid the natural disturbance of wall before test, Then the Pabijast was carried out from the bottom to the ground surface, as drawing up both casing pipes step by step. Oil supply to the borehole jack was done by a hand oil jack slowly and step by step. Each step was followed after the complete settlement of pressure and deformation. This implies drained condition. The existence of two boreholes and the ground of granite is more suitable for drained test than undrained test.

Fig. 5 are examples of the Pabijast data. 7m is under the sliding surface, the structure of layer is intact, then the difference between peak strength and residual strength is big. 5.7m is the sliding zone, it is disturbed very much, then the peak strength seems to be the same with the residual strength.

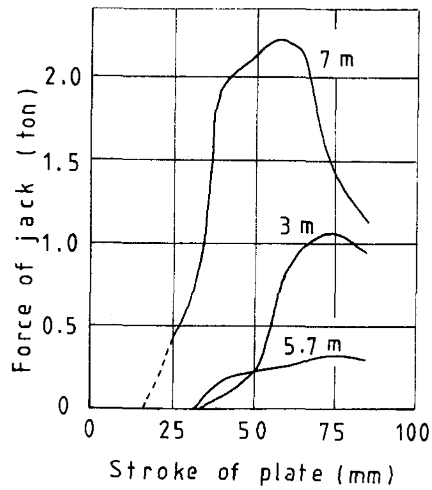
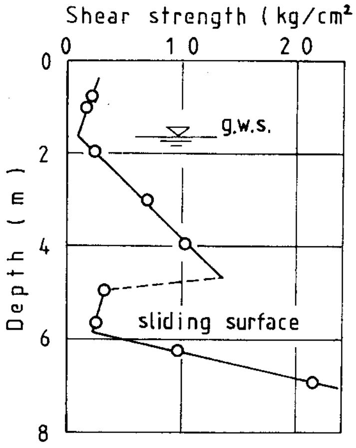


Fig. 5. Examples of data in the Pabijast

Pig.6-A is the variation of shear strength calculated from the measured data by equation (2), Fig. 6-B is the observation from boring sample. According to the boring, the natural ground water surface is 1.65m depth. 6 m is the clear border, because the layer above 6 m is brown and the layer below 6 m is blue, the difference of colour is very vivid. It is sometimes observed that the colour above the sliding surface is different from it below the surface. The colour above the sliding surface is usually reddish brown, and the colour below it is usually blue or dark grey. This is caused by the difference of the degree of oxidization of iron. The layer above the sliding surface includes trivalent iron because the layer is permeable and the ground water level changes in it, and oxygen can be supplied enough. The layer below the surface includes bivalent iron because it is tight and comparatively impermeable, then a small amount of ground water can saturate it and keep it from oxidization. Therefore, the border of colour at 6 m depth is supposed to be the sliding surface. This presumption has been confirmed by the fact that a small landslide occured on this border of colour, when we excavated up to about 3 m depth (just below the border) for sampling and drainage well in a place 15-20m from the point of the Pabijast.

The result of the Pabijast shows that shear strength decreases rapidly at the border of colour of 6 m, namely the sliding surface as mentioned above and the disturbed layer just



m	Colour	Lithology	Observation
	light brown ↓ dark brown	weathered granite debris	most weathered
2			heavily weathered slided mass
4			
6			very weak poor recovery
	blue grey	granite	disturbed
8		tuff breccia	undisturbed

(A) Result of the Pabijast (B) Result of boring

Fig. 6. Reuults of the Pabijast and boring

above it, and it increases rapidly below the sliding surface. These results are quite reasonable.

Shear strength seems to decrease at the ground water surface, too. This phenomenon is explained by the capillary tension. The capillary tension works as a kind of normal or negative pore pressure. (It takes important role for slides in a homogeneous layer. This is researched in reference 1,2,4) . The capillary tension works in unsaturated zone and increases shear strength. But it decrease rapidly above the ground water surface because the water content increases and the capillary tension disappears rapidly there. Therefore, the shear strength decreases above the ground water surface, though normal stress increases with depth. This phenomenon was observed also by vane test in model layer using standard sand (reference 3) .

6. Comparison of the Pabijast with Simple Shear Tests and a Small Landslide Caused by Excavation

we excavated a place 15-20 m apart from the point of the Pabijast to take samples for simple shear tests. If the geological section were the same with the point of the pabijast, it must be very effective in the numerical comparison between the Pabijast and the simple shear tests. However, unfortunately the sliding surface ascended to 3 m depth in this point, though it was at 6 m depth in the point of Pabijast. Then the comparison of shear strength in the same depth and the same geology was limited. Therfore, to examine the shearing characteristics of the same geology, we took 14 block samples (30×30×30cm) from 1 m, 2 m, 2.5 m (these three are of brown granite debris) , 3 m (the sliding surface, the samples include it in the center) , 3.5 m (blue weathered granite) and 4 m (tuff breccia, it was too hard to test) .

The condition of test is consolidation, drain and constant strain, The condition of water content is both natural water content and saturation. When doing simple shear test of weathered granite, shear stress often continues to increase untill 25-30% strain. Such a big strain can not give right shear strength from the mechanism of the test. Furthermore peak strength often gives too big strength for slides. Then we used the upper yielding value instead of peak strength. The upper yielding value is measured as the first inflection point of the linear stress-strain relation plotted on both logarithm paper which was proposed by Murayama-Shibata in reference 5 & 6. They stated there clay will failure in the future if a stress lager than upper yielding value is permanently applied, though the value is always smaller than peak strength.

Fig. 7 and Fig. 8 are their results of simple shear tests. Fig. 9 is density and water content obtained from samples. All of the samples, namely the sample with the sliding

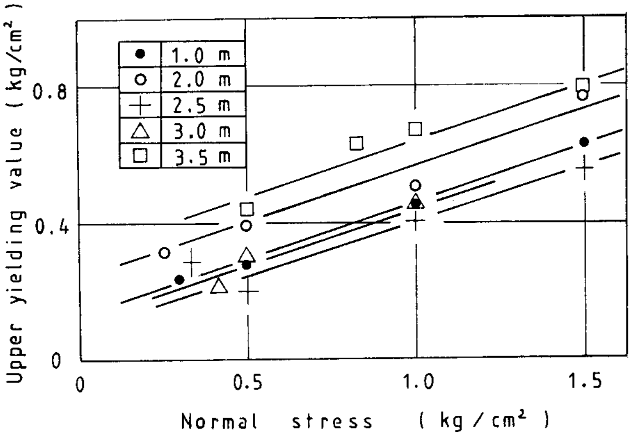


Fig. 7. Result of simple shear tests (natural water content)

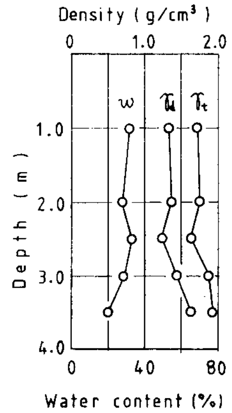


Fig. 9. Density and water content of samples

γt : wet density

γd : dry density

ω : water content in percent of dry weight

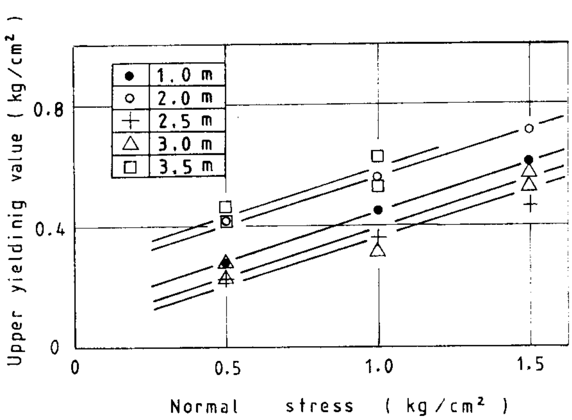


Fig. 8. Result of simple shear tests (saturated)

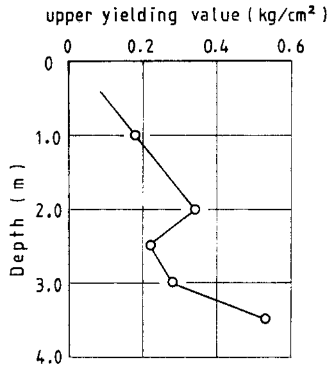


Fig. 10. Variation of upper yielding values obtained from Fig. 7 & 8

surface in it, the brown granite samples and the blue granite sample, have the same angle of internal friction. Only C is different. Next we will estimate the variation of upper yielding value with depth, using Fig. 7 and Fig. 9. Fig. 10 shows it. The shape of Fig. 10 is similar to Fig. 6. There is a zone of low shear strength with 0.5-1.0m thickness above the sliding surface. The zone is so disturbed as the sliding surface itself. Shear strength increases again below the sliding surface.

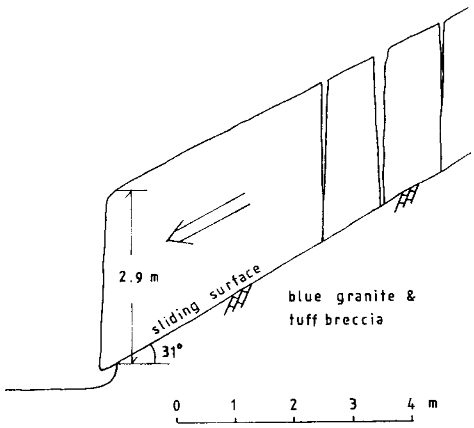


Fig. 11. Landslide caused by the excavation

Now we have two data (1 m and 2 m) to compare between the Pabijast and the simple shear test in the same geology and the same depth. Before comparing them, we will check the upper yielding value. The upper yielding value is not so popular as the peak strength and the residual strength. Fortunately we observed a small landslide by excavation for sampes of the simple shear tests and also a drainage well. Then we will examine whether the upper yielding value is corresponding to the actual shear in the field or not. The small landslide is shown in Fig. 11. The sliding surface was on the border between the brown granite and the blue granite. The velocity of the landslide is 0.1-0.2 cm/sec, when the author noticed and measured.

This landslide is not rotational type, but slab slide, The sliding surface was very straight in this part, which was confirmed by dittch excavation on a longitudinal line in its center. The movement of landslide was observed to be straight also. Therefore. when considering tensile strength negligible, it is possible to neglect the resistance at the tension crack shown in Fig. 11. The main tension crack was formed at about 5 m from the end of landslide, the other tension cracks were also observed before and benind it. The width of landslide was 13.5 m which is enough big comparing with the length. Therefore, we could neglect the side friction without a big error, as usual analyses of slope stability do so.

When regarding the resistance at the tension crack and the sides as zero and the safety factor of this slope as 1.0, we can calculate the shear strength on the sliding surface using its depth (2.9 m) , wet density (1.75 g/cm^3) and the slope angle of sliding surface (31 degrees) , we obtain 0.26 kg/cm^2 for it.

Water content of the mass of landslide is nearly the same with the natural water content of samples, because sampling was done just before and after the landslide. Fig.10 shows that the shear strength of sliding surface at 3.0 m is 0.28 kg/cm^2, and we get 0.27kg/cm^2 from Fig.7 as the shear strength of the sliding surface under the condition of natural water content and the normal stress for 2.9 m overburden pressure. Both values of the shear strength calculated from landslide (0.26kg/cm^2) and the strength obtained from simple shear test ($0.27 kg/cm^2$) are practically equal. From the examination above, the upper yielding value of this simple shear test can be deduced to have given right value for the actual shear in this field, namely the landslide.

Let us compare the values of the Pabijast with the upper yielding values of simple shear tests. We can compare the data of 1.0 m and 2.0 m depth under the same conditions on depth and geology. The shear strength obtained from the Pabijast at 1.0 m is $0.17 kg/cm^2$. We will decide vertical shear strength τ_V and horizontal shearing strength τ_H at 1.0 m depth from the data of 1.0 m in Fig.7, then calculate the shear strength due to be given by the Pabijast when Fig. 7 is right.

From Eq. 1 & 2, the shear strength obtained from the Pabijast is shown by

$$\tau = \frac{1}{(S_V + S_H)} \cdot (S_V \cdot \tau_V + S_H \cdot \tau_H) = 0.76\ \tau_V + 0.24\ \tau_H \qquad \cdots\cdots\cdots(6)$$

regarding the coefficient of lateral stress at rest as 0.5 and using Fig, 9,

$$\sigma_V = 0.17\ kg/cm^2,\ \sigma_H = 0.09\ kg/cm^2$$

then we get τ_V and τ_H from the data of 1.0 m in Fig. 7.

$$\tau_V = 0.16\ kg/cm^2 \qquad \tau_H = 0.19 kg/cm^2$$

Uning Eq. 6, we have $\tau = 0.17 kg/cm^2$

Therefore tne value of the Pabijast is just equal to the shear strength obtained from the simple shear tests.

In the case of 2.0 m depth, the shearing strength by the Pabijast is $0.26 kg/cm^2$. 2.0 m is 0.35m below the ground water surface. Then,

$$\sigma_V = 0.31\ kg/cm^2,\ \sigma_H = 0.16\ kg/cm^2$$

We can get τ_V and τ_H from the data of 2.0 m in Fig. 8 Dy using σ_V and σ_H

$$\tau_V = 0.30\ kg/cm^2,\ \tau_H = 0.34 kg/cm^2$$

Putting them into Eq. 6, we have $\tau = 0.31\ kg/cm^2$

Therefore the value of the Pabijast is very similar to the value obtained from the simple shear tests, too.

From these examinations, it can be said that the machine of Pabijast worked properly and the Pabijast gave reasonable shear strengths in this trial test. Hence, how and where to use the Pabijast is a subiject of the following research.

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