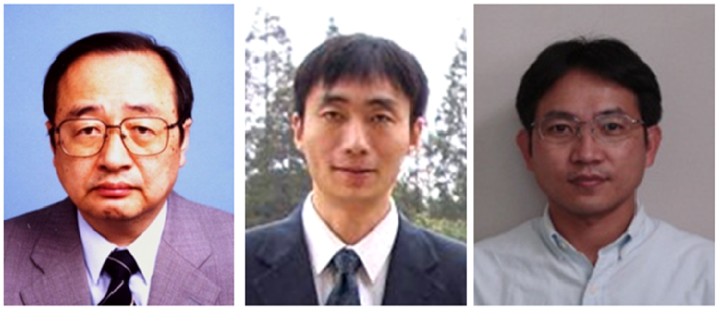
Failure of a Cut Slope and Deterioration of Shear Strength due to Weathering



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ABSTRACT: In this paper, strength distribution of a cut-slope of stiff clay and strength deterioration leading to a failure are investigated. A slope with a gradient of 1:1 measuring 10m in height and 50m in width was cut for a new local road through hills constituted of stiff clay. Four months after cutting the slope, a 25m wide portion of the slope failed. On the site, two borehole tests for sampling and SPT were done. In-situ sounding tests and sampling by the bench cut method were carried out. From results of sounding tests at the site, the strength distribution of the clay in the slope is found from the surface of the original slope to the depth, which is considered to be traces of weathering evident during a long period of time.

Then, five series of shear tests on the stiff clay, test series *A* to *E*, were carried out using the direct shear apparatus, which was improved by Mikasa (1965), in order to observe strength deterioration of stiff clay. The test conditions were programmed to simulate different degree of weathering conditions by controlling the water absorption into the samples. It was found that samples with a lager amount of water absorption showed lower shear strength. Therefore, it was concluded that deterioration of stiff clay at the site induced the slope failure. It should be noted that the most effective and useful method to simulate different degree of weathering is not by way of triaxial compression tests, but direct shear tests, and that this approach, which simulates weathering of stiff clay on samples in a shear box, follows the concept of '*Advanced Total Stress method*'.

1. INTRODUCTON

Terzaghi(1936) suggested that the cause of failure of stiff clay slopes with a gentle gradient arises from deterioration of the slope's strength when water is absorbed by the soil. Skempton(1964a) carried out an investigation into the failure of a stiff-clay cut slope which occurred more than 50 years after the slope was made. The delay of failure was explained as being due to slow equilibrium of pore water pressure in the slope, namely a phenomenon of progressive failure in a stiff clay slope. Bjerrum (1967) proposed a model which could predict propagation of the deterioration of shear strength in a slope. On the basis of these studies, Henkel et al. (1955), Sevaldson (1956), Skempton (1948, 1964b) presented case studies of slope failures which occurred at three locations in the United Kingdom.

In Japan, Mikasa et al. (1967), Okuzono (1976), and Nisigaki (1977) studied the failure of stiff clay slopes, and deterioration of strength due to absorption of water into stiff-clay was taken into account in the analyses. However, progression of a slope from a stable to an unstable condition has not been explained using shear test results up to date.

In this paper, a failure of a cut slope, which was constructed as part of a new local road through hills constituted of stiff clay of the Osaka Marine Clay group in the southern part of Osaka, Japan is investigated. The slope's gradient was 1:1 and had a berm at a height of 10m measuring 50m length for a new local road. Four months after cutting of the slope, in early spring, a 25m wide portion of the slope collapsed. Fig.1 shows a photo at the site of the slope failure. Site investigations using sounding method and laboratory tests were carried out in order to study the cause of this failure.

In order to observe degree of 'weathering', five series of shear tests on samples from the site were planned to be carried under *CD-condition* using Mikasa's direct shear apparatus. This discussion on deterioration of shear strength of stiff clay does not deal with change of pore water pressure in a sample, but with change of shear strength directly based on the concept of *'Advanced Total stress method'* (Mochizuki et.al., 1984, 1996).

Fig.1 Picture of the slope failure

2. OUTLINE OF SITE AND SOIL INVESTIGATION

Fig.2 shows a surface geological map of Osaka plain and the surrounding area. The site of the cut-slope failure, which is marked on the map, is located 5km east from Osaka bay and 23km west of the Ikoma-Kongo mountain range. From a view point of geological history, Osaka plain was formed over the last two million years as sedimented cyclic layers of clay, silt, sand or gravel due to cyclic geological movements and climatic changes. Depth of the sedimented plain ranges from 600 to 1000m.

The sedimented layers during the above mentioned term are named as the *'Osaka group layers'* in which 14 layers of marine clay are included, these are numbered *Ma-0* to *Ma-13* in ascending order. *Ma-13* is the most recent layer of Alluvial marine clay. On the other hand, the Ikoma-Kongo mountain range with height of 800 to 1,100m has been created actively during the past five hundred thousand years by heaving, folding etc.. Due to this the region, including the site, was also risen, and the *Ma-8* layer (the ninth layer in *Osaka marine clay layers* from the base layer, *Ma-0*) appeared in the surface of ground and hilly terrain with a height of 20 to 30m above sea level was formed.

Fig.3 shows a plan of the area around the landslide and the location of *in-situ* sounding tests. In addition to the site for sampling by the bench cut method and boreholes. To determine the stratigraphy of the soil and strength distribution, *the Swedish Sounding test* was carried out using both a standard size tester and a smaller-size tester.

The profile of borehole No.9, whose location is shown in Fig.3, is presented in Fig.4 as a representative soil profile of the hill. A layer, measuring almost 2m thick, of dense white-brown gravel with maximum particle size of less than 50mm covers *the Ma-8 stiff clay layer*, which is approximately 6m thick. This clay is grey-blue when fresh but changes to a white-brown color shortly after drying and also develops cracks and flakes propagating at the surface of the slope. Alternately there are thin silty-clay layers with a small amount of fine sand, and thin sandy-silt layers underlie the stiff clay. Although vegetation was planted, grass growth was poor due to the presence of sulfur in the clay.

Boreholes were later used as observation wells of the water level in the hill. A small increase in water level followed after rain fall as *G.L-12.6m* to about *G.L-12m* and it returned to the former level after about a day. From water level behavior, it is deduced that the large amount of water supply with high water pressure did not exist in the layers.

Laboratory tests were carried out in order to simulate the difference in the degree of weathering present in the stiff-clay samples. Direct shear tests were carried out in order to observe the deterioration of strength under several different conditions of water absorption into the samples.

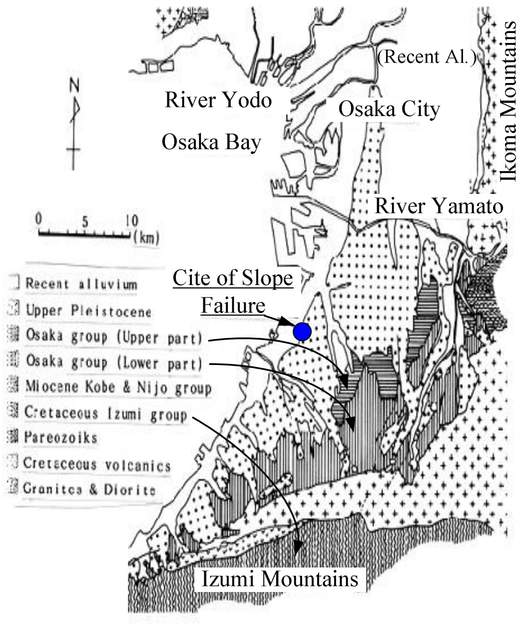


Fig. 2 Surface geological map of Osaka plane

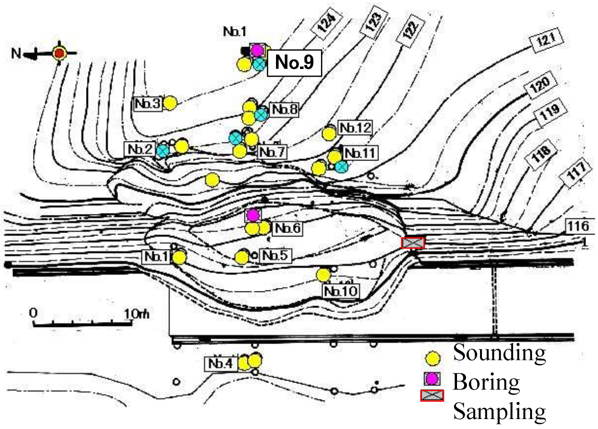


Fig. 3　Topographic map of failure site

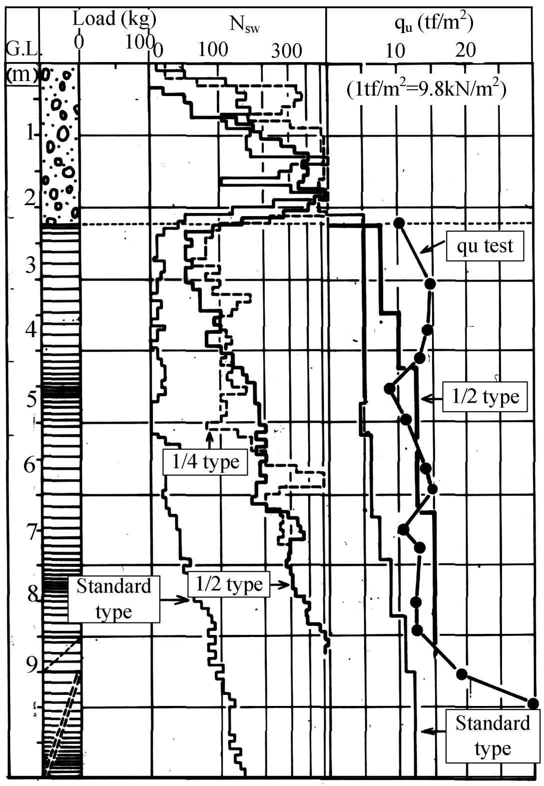


Fig.4 Drilling log at point No.9

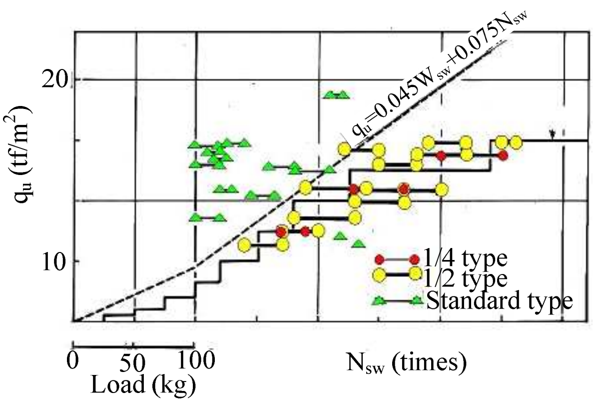


Fig. 5 Relationships of N_{sw} to q_u
: *Swedish sounding test*

3. SOUNDING TEST AND DISTRIBUTION OF STRENGTH IN THE SLOPE

A Swedish-sounding tester of the standard size was not suitable to use at the slope site due to its heavy weight. Therefore, a much lighter sounding tester (weighing half that of *the standard Swedish tester*) was developed with a reduced similar shape in order to impose the same stress as that of the standard tester *(1.13MN/m^2)* on the cross-section of the screwing point.

Fig.5 shows a correlation between strength of unconfined compression tests *(q_u)* and the half-rotation number obtained from the sounding test *(N_{sw})* at borehole *No.9*. The value of N_{sw} is plotted by a pair of symbol marks against q_u*-value* since N_{sw} was observed at every 10 cm, whereas q_u was measured every 50cm to 100cm.

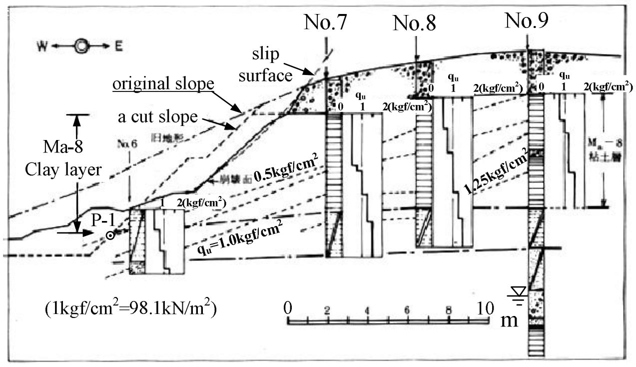
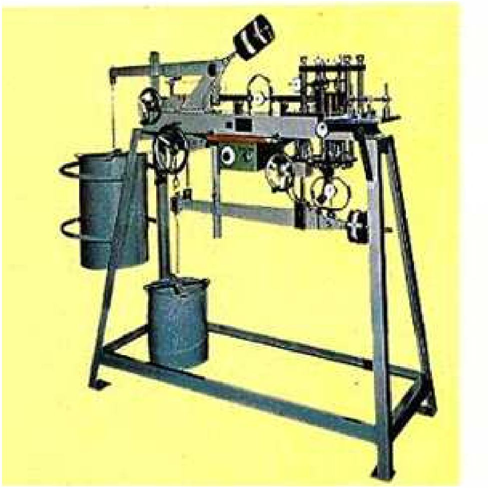


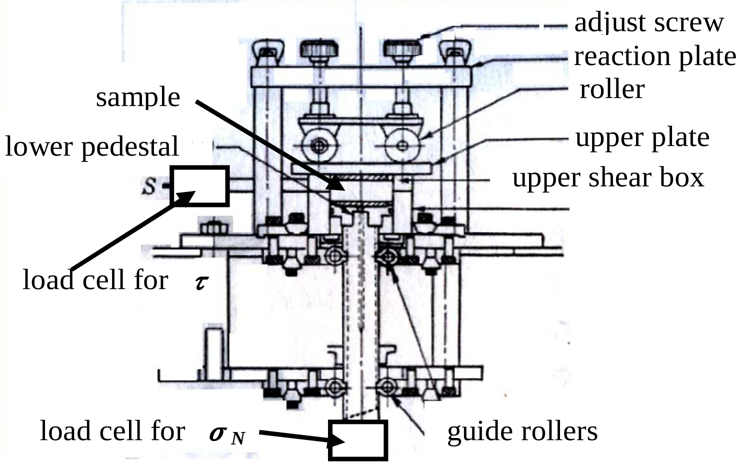
Fig. 6 Cross section of the slope and and strength distribution

A step-like relationship between N_{sw} and q_u for the sounding test with 1/2 of the standard weight, as shown in Fig.5, is adopted. The dotted line in the figure was presented by Inada (1954) for the standard tester, however, this equation gives comparatively higher q_u*-values* than results obtained from an unconfined compression test carried out on the clay.

Fig.6 shows a cross-section through the central part of the landslide and the strength distribution of the sounding tests. Broken lines with dots separate zones with an increment of *0.25 kgf/cm^2* (≒ *25kN/m^2*) of q_u. These show values observed by the sounding tests. It is found that strength contours in the clay layers are almost parallel to the original topography, though t*he Ma-8 clay layer* is located almost horizontally, namely, the parallel distribution of strength to the original topography is expected to be. It is supposed that some degree of weathering must have taken place as the decrease of the strength which should occur perpendicularly to the original hill surface.



(1) General view



(2) Main structure

Fig.7 Outline of Mikasa's direct shear apparatus

The slip surface observed was to pass through *P-1* crossing the lower boundary of the Ma-8 Clay layer, coming up towards the top of the slope as shown by a dotted line in Fig.4.

4. SHEAR TESTS AND STRENGTH PARAMETERS

4.1 Apparatus, samples and drainage conditions in shearing tests

The direct shear apparatus used in this study is an improved version of the system developed by Mikasa (1960, see Fig.7(1)). The diameter of specimen is 6 cm and the thickness is 2 cm. The main feature parts of apparatus are shown in Fig.7(2). The new version has overcome several inadequacies inherent in standard apparatus such as tilting of the shear-box during a test, and the development of friction between the upper and lower half of the shear boxes. The other important feature is that a test under consolidated and undrained conditions, *CU*, can be performed by keeping the volume of a sample constant by increasing/de-creasing vertical stress, σ_N, on the sample during the test. From the results of *CU-test*, the effective strength parameters, c' *and* φ', and the total strength parameters, c_{cu} *and* φ_{cu}, are obtained. Here, parameters c' *and* φ' are regarded as equivalent values as those of *CD-test*, c_d *and* φ_d.
Un-weathered and undisturbed samples for a series of direct shear tests were taken from borehole *No.9* and others were taken from the horizontal surface of a bench cut near the failure zone by driving thin walled tubes measuring 100mm in diameter and 3000mm in length (see Fig. 8). Samples from the bench cut shown had a light-brown color suggest

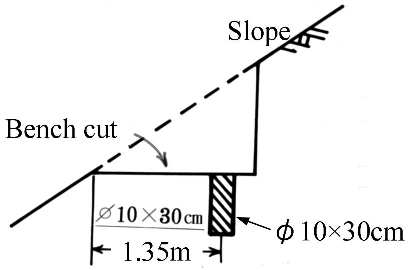


Fig.8 Bench cut and sampling tube

ing that it weathered more than those sampled from the borehole.

Samples obtained from the borehole were used for a series of *standard CU-tests*. This test is denoted by the letter *A*, such as *Test-A* or *Strength-A*. Other samples obtained by the hand trimming method at the bench cut were tested under consolidated and drained condition *(CD-condition)*, during which different degrees of weathering were simulated by controlling water supply into the sample. These tests are denoted by the letter *B, C, D and E* respectively, according to the water supply condition. Table-1 shows main physical properties of the sample. Testing procedures are given in section 4.2 and 4.3.

The reason for performing *CD-tests* for *Test-B to E* is explained as follows; The stiff-clay slope failed four months after construction. During that time, the soil experienced shrinkage by drying with the sun shining and dilation by absorption with wetting by rain falls many times. This condition corresponds to a drained condition. As a result, it was deduced that a decrease of soil density after dilation due to water absorption caused decrease of strength, which resulted in the slope failure. This process into the failure is regard as '*a failure mechanism of stiff clay slope with a long term*'.

Table 1 Main physical properties of clay

w_n, ρ_s	23%, 2.621g/cm^3
p_y, Cc	66.6kN/m^2, 1.15
S : M : C	27 : 41 : 32 (%)
LL, PL	46, 19.3 (%)
γ_d, e	1.634t/m3, 0.603

4.2 Standard CU-test (Test-A)

Table 2 shows the testing program, and procedures followed for *the JGS guide line* on *the CU-test* using the direct shear apparatus (Takada, 1993). Fig.9 shows the effective stress path of tests obtained in terms of shear stress, τ, and vertical effective stress, σ'. The strength parameters, *c' and* φ', are obtained from a strength envelope connecting τ_{max} *and* σ' on stress paths. The envelope line is divided into two parts at a stress of σ'_b, where the stress σ'_b is *the stress representing influence of consolidation yield stress* (Mochizuki, 1996). It is known that the envelope generally shows an upward curve for a stress range less than σ_b', whereas a higher strength stress range than σ_b' gives a straight line which passes through the origin of the graph.

Hence, the strength parameters, c_{cu} *and* φ_{cu}, are obtained from an envelope passing through stresses characterized by the maximum shear stress, τ_{max}, and the consolidation stresses of soil, σ'_0. The envelope for *CU-condition* is also divided into two parts at a stress of σ'_c, showing an upward curve for $\sigma>\sigma'_c$, and a straight line for $\sigma'_c>\sigma$. Here, stress, σ'_c, is named as '*yield stress from a CU-shear test*, and is regarded as being equivalent to the consolidation yield stress, σ'_y. This test denoted as *Test-A*, and the strength obtained is termed as *Strength-A*, which will be dealt with as an index strength for *Strength-B to E*, which will be shown in the following section. 2.4

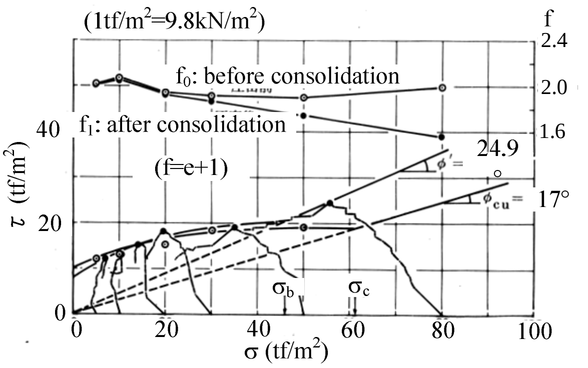


Fig.9 Relationships of shear stress to vertical stress and strength curves: Test-A (CU-test)

It is understood '*qualitatively*' that weathering causes deterioration of the strength of stiff clay, which can lead to failure of the slope. Therefore, four series of direct shear tests were carried out on stiff clay samples from the bench cut near the slip surface, in which the four degrees of 'weathering' were simulated by changing the amount of wetting into the samples during the tests. These tests are denoted *Test-B, C, D and E* respectively.

The test conditions of *Test-B, C, D and E* are shown in detail in Table 3; In *Test-B*, no water was introduced into the sample during the shear tests in order to simulate a *'lightly weathered'* condition. In contrast, in *Test-C* the samples were allowed to absorb water during only the consolidation phrase so as to simulate a *'medium weathered'* condition. In *Test-D* enough water was flooded into the sample during the consolidation and shearing phrase to simulate a *'heavily weathered'* condition. In order to measure residual strength of clay, the sample in *Test-E* was sheared horizontally at a displacement of *'+5mm and -5mm'* by repeatedly until 45mm in a total displacement. During the test, the sample was allowed to absorb water.

Fig.10 shows stress-strain curves for *Test-B* to *Test-D*. Fig. 11 shows shear strength, τ_f, plotted against consolidation stress, σ_0, and the strength envelopes from all of the tests. Results of unconfined compression tests, c_u *(=qu/2)* are also plotted. These envelopes show a decrease in strength from *Test-A to Test-E*. At stresses higher than σ_b', the φ_d defined by *Test-B, -C and -D* (φ_d=27.6°) almost coincide with φ' of *Test-A* (φ'=24.9°), whereas φ' is slightly smaller. It is found that curves in a stress level less than σ_b' are almost parallel corresponding in *Strength-B to D*. *Strength-B* shows almost half shear strength of *Strength-A*, and *Strength-D* is almost a quarter of *Strength-A*.

Residual strength is found to be smaller than *Strength-D*, and shows much smaller inclination of the strength curves. It is apparent that the residual strength does not belong to the scheme of strength curves in other tests.

5. STRENGTH PARAMETERS FOR EACH TEST AND DETERIORATION OF STREGTH

It is known that strength in the stress range less than σ_b is generally shows relationships of an upward convex curve. In such a case, it may be a practical way to connect stresses by a straight line between an intercepting stress on the *τ-axis*, c_1, and its corresponding shear strength, τ_b.

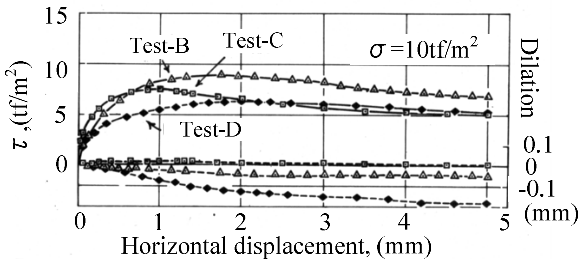


Fig.10 Relationships of shear stress to displacement: *Test-B, C, D*

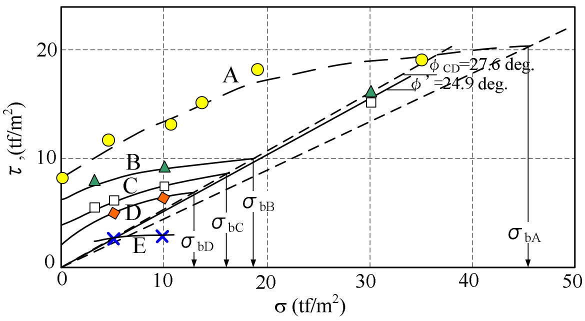


Fig.11 Strength curves obtained from *Test-A to E*

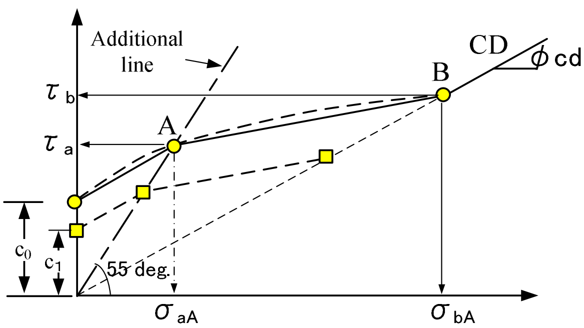


Fig.12 Substitute method of strength by two straight lines for a convex curve

However, in a case of heavily overconsolidated stiff clay, namely a case of $\sigma \ll \sigma_b$, strength parameters must be expressed more accurately than that method mentioned above as a practical way.

Fig.12 shows a method schematically to obtain strength parameters in a stress range of $\sigma < \sigma_b$, which is developed; First, an additional line with a appropriate degree against the horizontal axis, 55 degree in this case, is put into a diagram of $\tau \sim \sigma_N$ *relationships*.

Then, we find a intersecting point on the strength lines to the additional line, *point A* in the figure. A convex curve is expressed by two straight lines connecting an intercepting stress on the vertical axis of τ-σ relationships, namely c_1, to *point A* (σ_a, τ_a), and *point A* to *point B* (σ_b, τ_b) in the diagram, here τ_b is shear stress corresponded to σ_b.

Table 3 shows strength parameters for each test. It is clear that deterioration of strength due to absorption during the tests is expressed as decrease of parameters, c_1, σ_a and σ_b, though ratio of decrease of parameters on φ_1 or φ_2 in Test-B to D against that of Test-A is not clear.

Table 4 shows ratios of parameters on c_{1i}, σ_{ai}, c_i, *and* σ_{bi}, against those of *Test-B*. Here, suffix *'i'* of each strength parameter expresses *B, C, D or E* respectively. It is found that *Strength-D* is almost half strength of *Strength-B*.

In this study, it should be noted that change of strength does not explained using change of pore water pressure or pore air pressure, and that the effective stress method is not the only approach in order to solve failure mechanism of stiff clay slope.

Table 2 Test conditions

	Imaged condition at the site	Test condition-1	Test condition-2
A[*1]	Weathering is not progressed; almost intact clay condition.	CU-test regulated by JGS.	σ_N=50, 100, 200, 300, 500, 800 kN/m^2
B[*2]	Just after a cut slope was constructed; lightly weathered condition.	CD-test with no-water supply during the test.	σ_N=30, 50, 100, 300 kN/m^2
C	Only several instances of rainfall have been experienced; medium weathered condition.	CD-test with water supply for 20hrs only during consolidation. process.	σ_N=30, 50, 100, 300 kN/m^2
D	Many instances of rainfall have been experienced; heavily weathered condition.	CD-test with water supply both during consolidation. and shearing processes.	σ_N=50, 100 kN/m^2
E	Lower bound strength of stiff clay at the site, though such a condition is not expected.	Cyclic shear test under CD-condition with full water supply condition for both processes.	σ_N=50, 100 kN/m^2

*1:Undisturved samples for test-A were obtained from bore-hole No.9.
*2:Undisturbed samples for test-B, C, D and E were obtained by hand from the bench-cut near the surface of the failed slope.

Table 3 Strength parameters for each test

	c_1 kN/m^2	φ_1 deg.	σ_a kN/m^2	c_2 kN/m^2	φ_2 deg.	σ_b kN/m^2	c_3 kN/m^2	φ_3 deg.
A	85.9	25.0	89.3	109.9	11.2	418.5	0	24.9
B	59.6	20.1	56.1	73.3	6.9	182.5	0	27.6
C	34.1	37.4	51.4	67.6	6.5	165.2	0	27.6
D	21.3	35.6	29.9	34.2	15.7	141.7	0	27.6
E	21.3	7.0	—	—	—	—	—	—

Table 4 Strength ratio against strength-A

	c_{1i} /c_{1B}	σ_{ai}/σ_{aB}	c_{2i} /c_{2B}	σ_{bi}/σ_{bB}
B	1	1	1	1
C	57	91	93	90
D	35	54	47	78
E	(35)	—	—	—

6. Conclusion

The cause of the failure of a stiff-clay cut slope, which occurred after few months later of the construction, was studied in this paper. The man conclusions are as follows;

(1) From the results of *Swedish sounding tests* and borehole tests; it was found that the strength distribution of *the Ma-8 clay layer* of slope had almost the same inclination as that of the original topography at the site. It is considered to be evidence of weathering from the surface of original slope to the depth during a long period of time.

(2) Regarding results of laboratory tests; as deterioration of shear strength of the stiff clay was estimated to have occurred in the the slope, five series of shear tests on undisturbed stiff clay from the site were carried out using *Mikasa's direct shear apparatus*. The test conditions were set to simulate different degree of weathering conditions by controlling water supply into the samples. It was found that a sample with a lager amount of wetting showed lower shear strength, showing that weathering due to wetting and drying was simulated by controlling water supply on a rather dry sample from the site in a shear box. It was concluded that weathering of stiff clay causes deterioration of the shear strength, which causes the failure of a cut slope.

(3) As the last comment, it should be noted that the approach in discuss of a cause of the slope failure is based on the concept of '*Advanced total stress method*', which was presented by the first author (Mochizuki, 1984, 1996). In order to find deterioration of shear strength of undisturbed stiff clay, commonly triaxial compression tests under *CU-condition* may be used, in which the main objective of the tests is to observe behavior of pore water pressure in a test sample during shear tests. However, in this study several series of direct shear tests, mainly under *CD-condition*, were carried out in order to observe deterioration of shear strength directly due to weathering without observing change of pore water pressure or pore air pressure in the samples. The principle of *'Effective Stress method'* may consider that the change of strength comes from change of pore water/air pressure even in stiff clay, though that of the the *'Advanced Total stress method'* takes it as a deterioration strength problem of a stiff clay in stress range of severely overconsolidation. Therefore, it is fundamental to take *CD-condition* for shear tests as a drainage condition of shear tests. Thus, test technique to control water supply condition was developed in the shear box. In this study, it is obvious that the direct shear tester is much easier to control such conditions when compared to doing so using a triaxial compression tester.

●Additional comment

Results of five series of shear tests on over consolidated stiff clay proved a cause of deterioration of shear strength due to weathering.

Howevere, only to show decrease of shear strenght is not enough to understand a failure mechanism of the cut slope due to weathering. Then, three series of analyses of slope stability on the cut slope, namely in order to obtain safety factor, *Fs*, were carried out using *the standard slope stability analysis method (or Swedish stability analysis method)*. It was found that safety factor of a slip surface, which was observed at the site, becomes *0.82 of Fs* when *Strength-D* is used, though results of analyses on the slip surface using *Strength-C* gives *1.23 of Fs*. From this, it is supposed that the cut slope failed when shear strength of the slope deteriorates from *Strength-C* to *Strength-D* due to weathering.

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