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Engineering Application of Direct Shear Box Test for Slope Stability Problem

사면 안정 문제에 대한 직접 전단 시험의 공학적 적용

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요 지

현재 일본에서는 사면 안정 문제에 있어서 파괴면상의 전단강도는 일반적으로 점착력의 경험적 예측에 기초를 두고 있으며, 그 점착력은 표토상부에서 가정한 활동면까지 흙두께에 비례한다고 가정한다. 이 예측법은 안정해석 결과가 설계자에 의한 영향을 받지 않는다는 점이 이점이나, 이론적인 배경이 부족하고, 파괴면 상의 흙두께가 매우 두꺼운 경우에는 점착력이 과다평가되어 역으로 때때로 그로 인한 전단저항각이 과소평가되지기도 한다. 본 연구에서는 2007년 효고현의 자연사면 파괴사례를 대상으로 파괴면상의 현장전단응력의 효과를 고려한 일련의 직접전단시험을 실시하여 주의 깊게 조사하였다. 그 결과, 초기전단응력을 고려한 시험으로부터 구한 전단강도가 사면파괴의 역해석에 적용되었을 때의 안전률이 현장상황과 부합하고 있음을 알 수 있었다.

Abstract

In the current practice for slope stability problem in Japan, the shear strength, τ, mobilized along the failure surface is usually estimated based on an empirical approximation in which the cohesion, c, is assumed to be equal to the soil thickness above the supposed slip surface, d (m). This approximation is advantageous in that the result of stability analysis is not influenced by the designers in charge. However, since the methodology has little theoretical background, the cohesion may often be grossly overestimated, and conversely the angle of shear resistance, ϕ, is significantly underestimated, when the soil thickness above the supposed slip surface is quite large. In this paper, a case record of natural slope failure that took place in Hyogo Prefecture in 2007, is described in detail for the case in which the shear strength along the collapsed surface was carefully examined in a series of direct shear box (DSB) tests by considering the effects of in-situ shear stress along the slip surface. It is demonstrated that the factor of safety agrees with that of in-situ conditions when the shear strength from this kind of DSB test was employed for the back-analysis of the slope failure.

Keywords : Back-analysis, Initial shear stress, Laboratory test, Slope stability

1. Introduction

Slope stability problem requires the strength parameters of soil that is cited, i.e., c : cohesion, and ϕ : the angle of shear resistance. In a practical design, these parameters are generally estimated based on an empirical approximation

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in which the cohesion c (kN/m^2) is assumed equal to the soil thickness above the supposed slip surface, d (m). As seen in Fig. 1, the value of tan ϕ is then back-calculated by the stability equation typically using Fellenius method (1936). From the engineering point of view, this methodology is advantageous in that the result of stability analysis is not influenced by the designers in charge. Moreover, since Fellenius method has no concern for whether the shape of potential failure surface is circular or noncircular, it is quick to decide a countermeasure work in the case of an urgent slope failure such as a road disaster.

Since the methodology has little theoretical background, the cohesion may often be grossly overestimated, and conversely the angle of shear resistance, ϕ, is significantly underestimated, when the soil thickness above the supposed slip surface is quite large. In such cases, the countermeasure construction works to prevent slope failure from happening such as the drawdown of ground water level, constructing counter-weight fill etc. become unrealistically ineffective and involvs enormous cost.

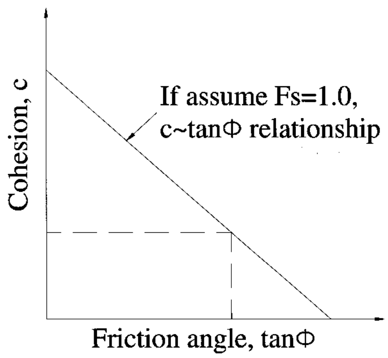


Fig. 1. Concept of determinate of strength parameter based on back-calculation

The fluent use of the empirical method may be attributed to the fact that the shear strength from the laboratory shear test is generally too small to account for the slope failures which occurred in the past. However, Shibuya et al (2007) has recently described the result of back analysis of a slope failure in which the factor of safety was far less than unity based on the undrained strength from conventional constant-volume DSB test, whereas it was close to unity when the shear strength along the collapsed surface was carefully examined in a series of constant-volume DSB test in which the effects of in-situ shear stress along the slip surface are properly taken into account.

In this paper, a case record of slope failure that took place in Hyogo Prefecture in 2007 was described in detail. The objective of this paper is to examine the applicability of the DSB test to slope stability problem, for which the soil samples retrieved from the slip surface were each subjected to in-situ shear stress under drained conditions, and it was sheared under constant-volume (i.e., undrained) conditions.

2. Outline of the Slope Failure

Fig. 2 shows the location of the examined slope located in northern part of Hyogo Prefecture in Japan. The site named "Ichiharakuroda" suffered from a catastrophic failure of cut slope in February 2007, resulting in close-down

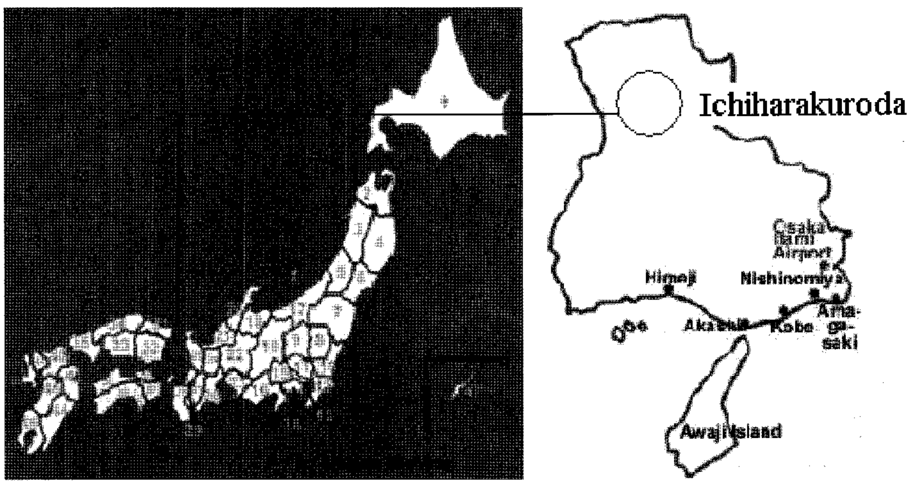


Fig. 2. Location of site

Table 1. Incidents to complete slope failure

Date	Observations	Events
Middle of October, 2007	Development of lateral tension cracks on the retaining wall	After continuous rainfall over a week. Field monitoring started.
Early February, 2008	The rate of ground surface displacement was 1.5 mm/hour	Heavy snowfall accumulated 48cm in the middle of January.
18 of February, 2008	The rate of displacement accelerated to show the value of 20-35 mm/hour.	Some increase of ground water level observed.
19 of February, 2008	Catastrophic slope failure	The failure took place within a few minutes.

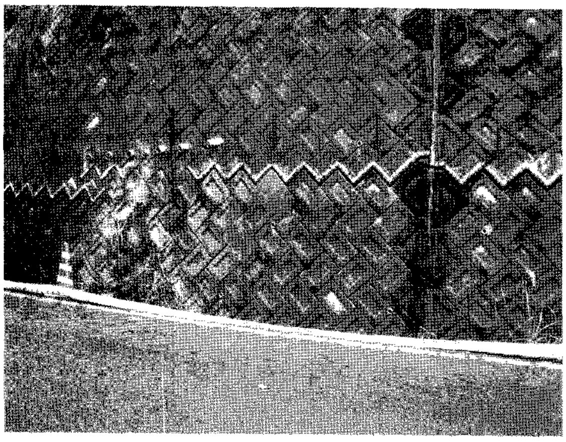


Fig. 3. Lateral tension crack on the retaining wall before snowfall

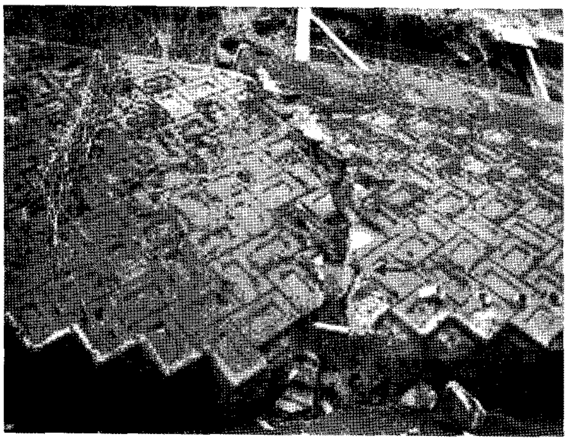


Fig. 4. Vertical crack immediately after slope failure

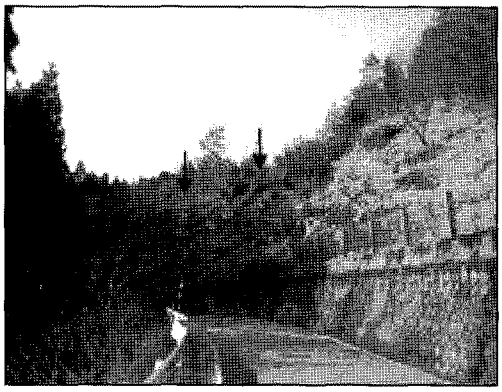


Fig. 5. View of slope failure of Ichiharakuroda site

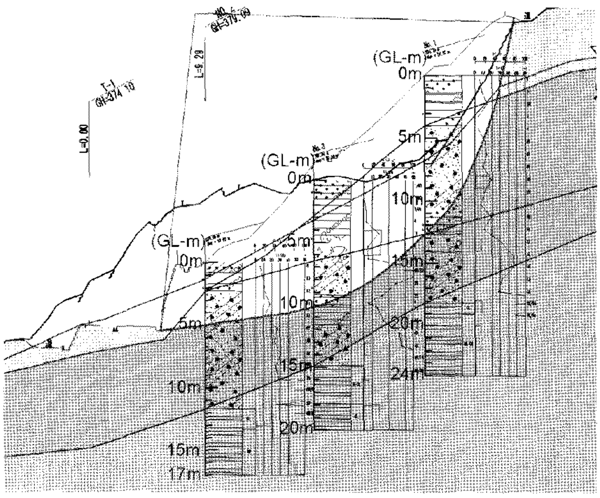


Fig. 6. Cross-section of the slope before and after the incident of slope failure

of the local road more than a month. Some observations into the behavior of the slope are summarized in Table 1.

Figures 3, 4 and 5 show some pictures of the site.

Fig. 6 shows the cross-section of the slope before and after the incident of slope failure. As seen in this figure, the natural slope can be classified into three zones, i.e., the upper clay layer with the SPT N-value ranging from 0 to 10, dense sandy soil layer with the N-value from 5 to 50, and the lower clay layer with the N-value of 3 on average, respectively. The shape of failure surface may be assumed to be either a type of non-circular composite failure surface or a circular one. In the former case, the angle of the slip surface relative to the horizontal was about about 9 degrees near the bottom and 57 degrees above it. In the stability analysis described later, these two types of failure surface are both examined.

3. Strength Parameters from Direct Shear Box (DSB) Test

3.1 Direct Shear Box Test Performed

In shear box of DSB apparatus Mikasa (1979) has pointed that an essential problem is sample and the classification by moving type of shear box has not important. Shibuya divided existing DSB apparatus as shown in Fig. 7 which shows three different types of shear box currently in use worldwide. These can be characterized as

Type 1: the top platen and the upper part of the box are each independently allowed freedom to move vertically and to rotate (Skempton & Bishop, 1950)

Type 2: The top platen is rigidly fixed to the upper box so that the two move vertically or rotate together (Jewell & Worth, 1987)

Type 3: the upper box is prevented from moving vertically or rotating; the top platen moves independently, but is can also be prevented from rotating (Mikasa, 1960; Takada, 1993)

In loading platen is fixed in the upper part of the box for the apparatus of Type3, DSB test can be performed under constant volume. In the shear boxes of Type1 and Type2, however, there is possible to arise some problem like below because the void ratio of soil sample changes in the process of shearing corresponded with its dilatancy property

1) In DSB test using normally consolidated clay or loose sand, the measured value of the vertical and shear stress has error because the both of shear box contacts due to minus dilatancy. For overconsolidated clay and dense sand, the soil particle comes out from the gab of the two shear boxes due to plus dilatancy.
2) the gab of the two boxes does not keeps its initial space due to rotation of the upper shear box Type1 so that the soil particle comes out from the gap or the two boxes contacts.

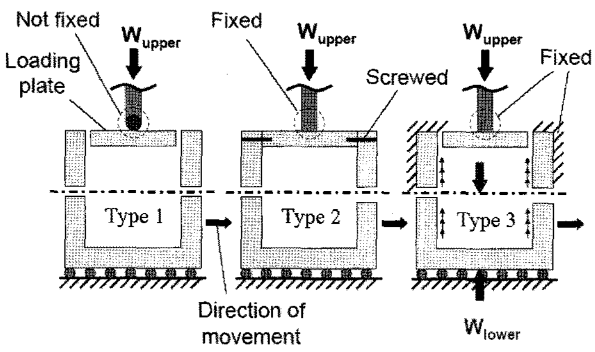
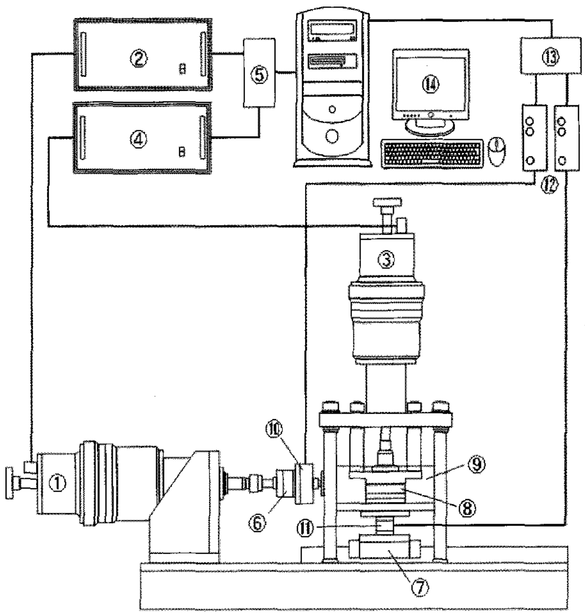


Fig. 7. Direct shear box apparatus employed (Shibuya et al., 1997; 2001)

The true vertical stress on shear surface is difficult to measure because the friction between the soil sample and shear boxes exists for Type3 apparatus. However, if the vertical stress is measured under lower shear box, the true vertical stress without the friction is evaluated. In this study, every DSB test results are based on newly developed Type3 apparatus.

Fig. 8 shows the DSB apparatus developed at Kobe University. In this fully automated apparatus, the vertical stress as well as the horizontal displacement can each be controlled by using a direct-drive motor (refer to Shibuya et al., 2001). The features of the DSB apparatus, together



①:direct drive motor for lateral loading ②:drive init for lateral loading ③:direct drive motor for vertical loading ④:drive unit for vertical loading ⑤:serial correspondence board ⑥:slide unit ⑦:linear roller way ⑧:shear box ⑨:outer box ⑩:load cell for lateral loading ⑪:load cell for vertical loading ⑫:strain amplifier ⑬:AD transformation board ⑭:personal computer

Fig. 8. Direct shear box apparatus employed (Shibuya et al., 1997)

with data acquisition system are in detail described by Shibuya et al. (1997, 2005). Constant-volume conditions can be readily achieved by maintaining the vertical movement of the loading ram zero during the shear. The vertical load is measured at the bottom of the lower shear box, implying that the vertical stress is free from any frictions between the soil and the shear box wall (refer to Shibuya et al., 1997). In all the tests performed in this study, the clearance between the upper and lower shear boxes was maintained constant at 0.2 mm.

3.2 Tests Performed

Soil sample tested was retrieved adjacent to the supposed slip surface at the position of Boring No.3 (see Fig. 6). The soil on the estimated slip surface was distinctive in that it was very soft sandy clay by showing relatively high water content and comprising many cracks. The soil slurry having the initial water content twice the liquid limit was prepared in the laboratory. It was one-dimensionally consolidated in a chamber using the vertical stress of about 80 kPa. Fig. 9 shows the grain size distribution curve of the sample tested.

In this study, two kinds of DSB test were performed. One is a sort of conventional DSB test in which the saturated soil samples each having the dimension of 60 mm in diameter and 40 mm in initial height were one-dimensionally consolidated in the shear box by increasing the vertical stress to the prescribed value, and the sample was afterwards subjected to undrained shear under constant-volume conditions. Note that the undrained constant-volume shear was employed, since the slope collapsed over a relatively short period of about a minute as observed using a video. The other type of DSB test was carried out in a similar manner, but the soil sample after consolidation was subjected to the application of horizontal shear stress under drained conditions. The initial shear stage was employed in order to simulate the in-situ shear stress in soil element along the slip surface.

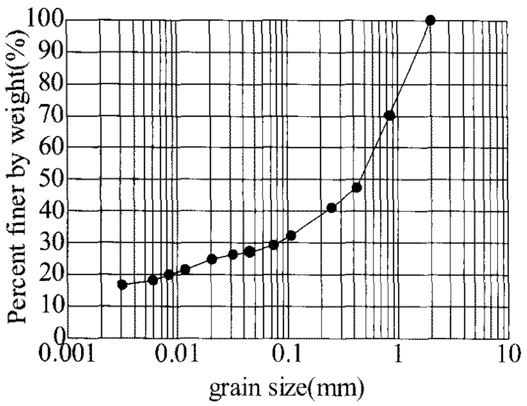


Fig. 9. Grain size analysis

Fig. 10 shows the stress states of soil element corresponding to these two tests. In the conventional test (see Fig. 10 (a)), the sample is consolidated and sheared undrained without the application of initial shear. However, this kind of test cannot simulate properly the stress history of the soil element along the slip surface. As seen in Fig. 10 (b), the slip plane has long been subjected to in-situ initial shear stress, τ_i. The τ_i value, together with the in-situ vertical stress on the slip plane can be expressed in the following form;

$$\tau_i = \frac{W \sin i}{A/\cos i} = \sigma_v \cos i \sin i \quad \sigma_i = \frac{W \cos i}{A/\cos i} = \sigma_v \cos^2 i \tag{1}$$

where the angle i denotes the slope angle. In this study, the stress history on the slip surface was closely simulated.

Figs. 11 and 12 show an example of the DSB test with the initial shear loading. As shown in these figures, the sample was first consolidated to the effective vertical

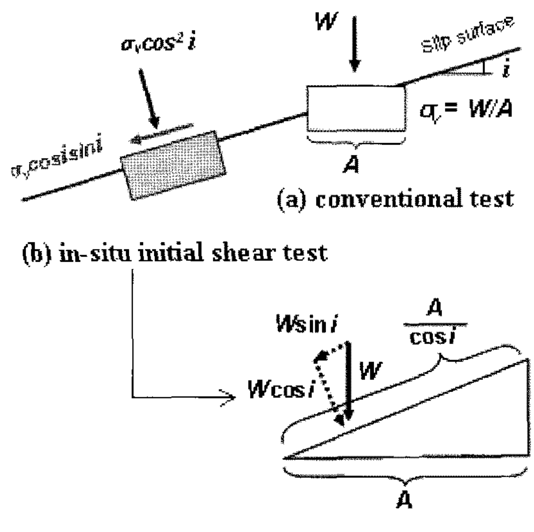


Fig. 10. Stress conditions along the slip surface (Shibuya et al, 2007)

stress, σ_v of 246 kPa, and the initial shear stress of 115 kPa was applied under drained conditions by using the rate of shear displacement of 0.002 mm/min, during this stage the sample exhibited a considerable amount of horizontal displacement more than 1 mm. The sample was after a common period of time of 1,000 mins subjected to shearing under constant-volume conditions by using the rate of horizontal shear displacement of 1.0 mm per minute.

In consolidation stage, the vertical stress, σ_v, was increased to three fixed levels of 100 kPa, 200 kPa, and 400 kPa by using a common rate of increase of 10 kPa/min. The angle i during the initial shear stage was varied at 0°, 15° and 30°.

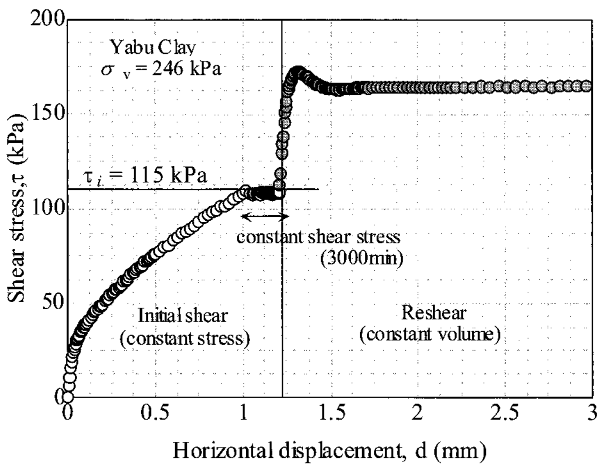


Fig. 11. Example of relationship with Horizontal displacement and shear stress in initial stress loading test

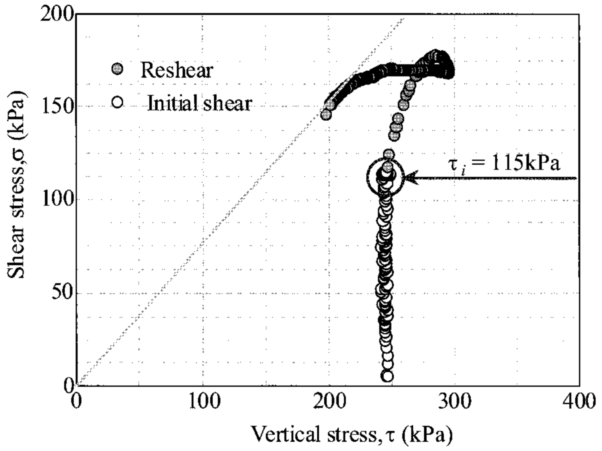


Fig. 12. Example of relationship with vertical stress and shear stress in initial stress loading test

4. Test Results

The DSB test results are shown in Figs. 13 through 16. Some results are also summarized in Table 1. The undrained stress path for a series of conventional test without initial shear loading (i.e., i=0) exhibited a typical of the behavior of normally consolidated soil by showing continuous development of positive pore pressure. Conversely, the samples with the initial shear loading exhibited a type of over-consolidated soil behavior by showing the initial sharp rise followed by accumulation of negative pore pressure

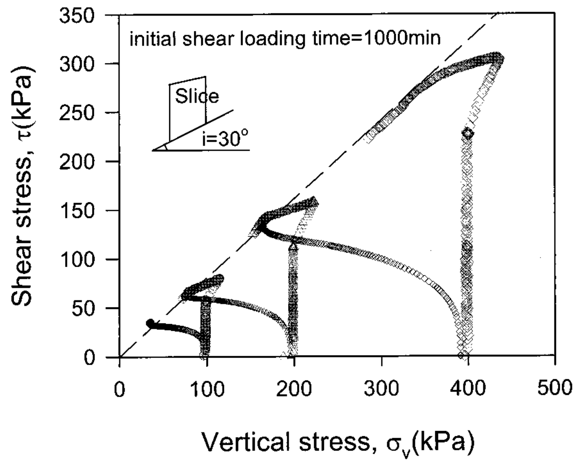


Fig. 13. Relationship with vertical stress and shear stress in the case of angle of slice bottom, i=30°

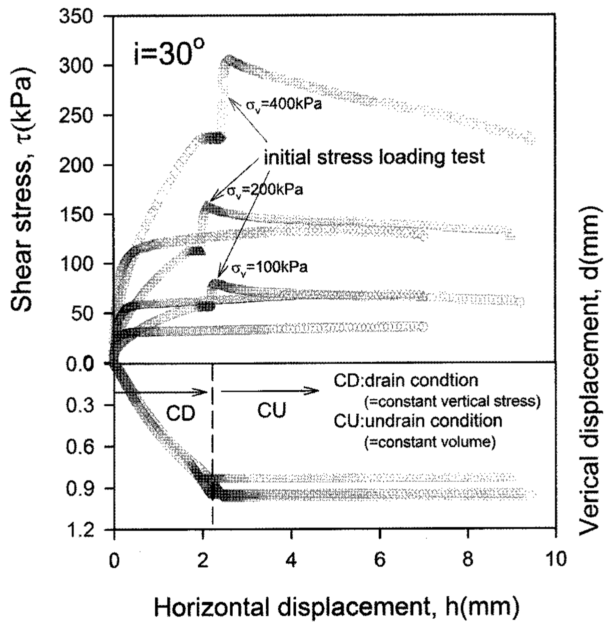


Fig. 14. Relationship with h ~ τ and h ~ d in the case of angle of slice bottom, i=30°

(refer to Figs. 13 and 15). As seen in Table 1, the ratio of undrained shear strength, S_u (i.e., the peak value of mobilized shear stress) to the relevant vertical stress σ_v was 0.34 on average for the conventional test without initial shear loading (i.e., i=0), while it was 0.79 on average for the samples with initial shear stress loading (i.e., i=15 and 30 degree tests). It is manifested that the initial shear stress loading brings about substantial increase in the shear strength.

Fig. 17 shows the relationship between the strength the S_u / σ_v value and the duration of applied vertical stress before undrained shear. Unlike the soil tested by Shibuya

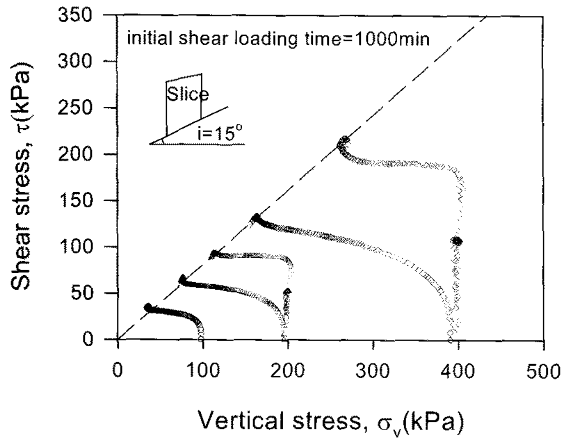


Fig. 15. Relationship with vertical stress and shear stress in the case of angle of slice bottom, i=15°

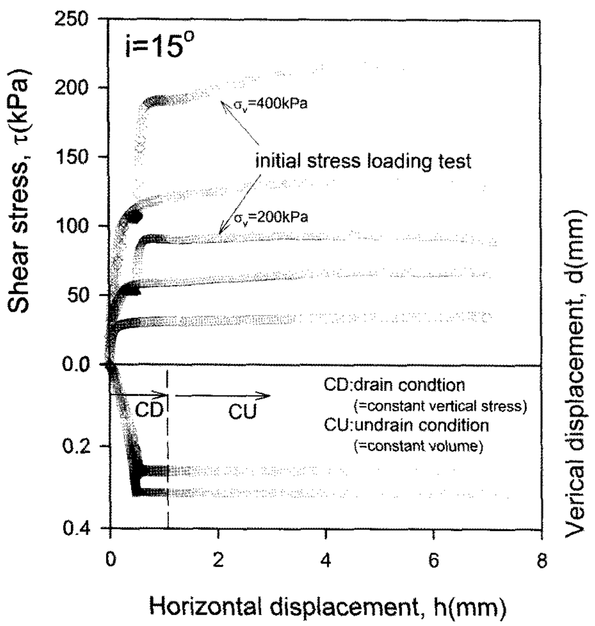


Fig. 16. Relationship with h ~ τ and h ~ d in the case of angle of slice bottom, i=15°

et al (2007), the time effects on S_u / σ_v seem negligible for the soil tested. Fig. 18 shows the effects of initial shear loading on the strength parameters. The S_u / σ_v (=arc tan ϕ) increases according to the degree of initial

Table 2. Comparison of the rate of strength increase in practical direct shear test and initial stress loading test

Practical direct shear test under constant volume condition (i=0)		
σ_v (kPa)	τ_{max} (kPa)	S_u / σ_v
100	35.77	0.36
200	66.97	0.33
400	133.55	0.33
	average	0.34
initial stress loading test (i≠0)		
σ_v (kPa)	τ_{max} (kPa)	S_u / σ_v
400	305.15	0.76
200	161.02	0.81
100	79.47	0.79
	average	0.79

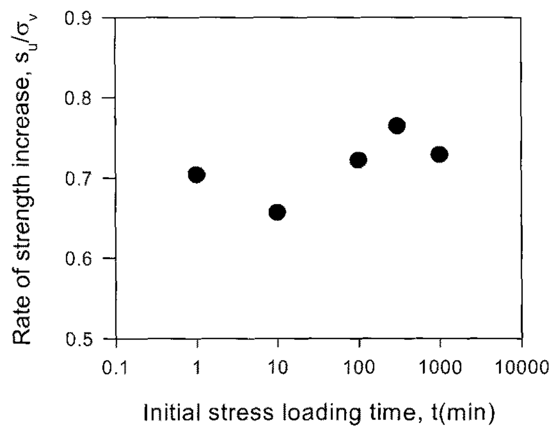


Fig. 17. Relationship with initial stress loading time and rate of strength increase

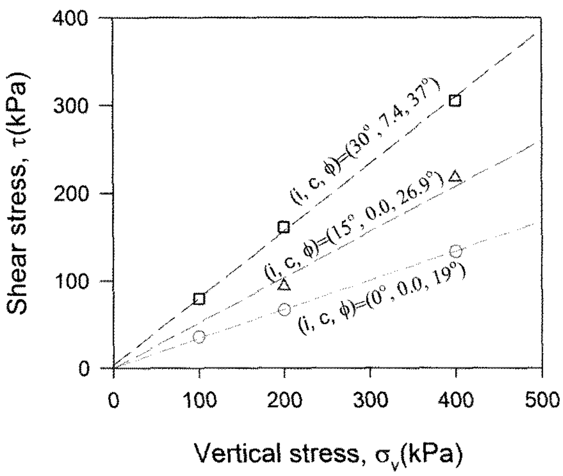


Fig. 18. Strength parameters and relationship with vertical stress and shear stress

shear loading, noting that (i, c, ϕ)=(0°, 0, 19°), (i, c, ϕ)=(15°, 0, 26.9°) and (i, c, ϕ)=(30°, 7.4, 37°).

5. Results of Slope Stability Analysis

The stability analysis was performed by using the modified Fellenius method, in which the factor of safety Fs is given by

$$Fs = \frac{\sum\{c \cdot l + (W - U \cdot b)\cos\alpha \cdot \tan\phi\}}{\sum W \cdot \sin\alpha} \quad (2)$$

where,

c : cohesion [kN/m^2]

l : length of each slice (m)

W : weight of slice [kN/m]

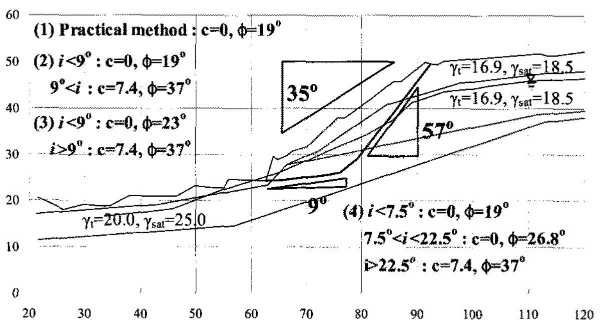
U : pore water pressure [kN/m^2]

b : width of slice (m)

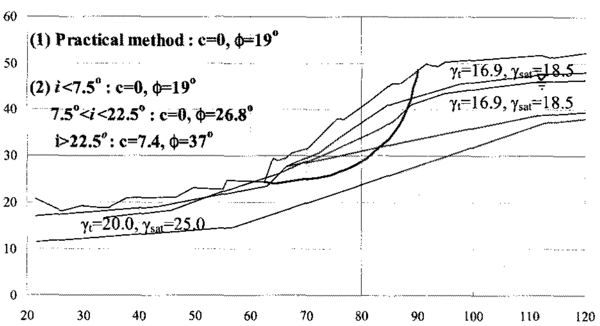
α : tangential angle of slice relative to the horizontal

ϕ : the angle of shear resistance

Fig. 19 shows the details of analysis by assuming non-circular and circular slip surfaces. In each case, the result of analysis by using the strength parameters from the



(a) non-circular slip surface



(b) circular slip surface

Fig. 19. Results of slope stability analysis

conventional DSB test with i=0 is compared to that using the strength parameters by taking the effects of initial shear loading into account. In the latter case with the non-circular slip surface (see Fig. 19 (a)), the angle i was taken as 9° and 57° for the bottom and upper slip surfaces, respectively. In the case of circular slip surface, the linear relationship between i and ϕ is postulated based on the measurement. It should be mentioned that the ground water level at the instant of slope failure was assumed to exhibit the same profile as that immediately measured after the slope failure.

Table 2 summarized the results of stability analysis. The factor of safety from the results of the conventional DSB test yielded unrealistic values far less than unity, 0.53 and 0.56, for both the cases assuming non-circular and circular slip surfaces. Conversely, the Fs is close to unity, 1.05 in the cases of (2) and (3) applied to the composite slip surface. The Fs value was slightly larger than unity, Fs = 1.15 when the circular slip surface is postulated. These results strongly suggest that the stability analysis using the strength parameters from the DSB test with initial shear loading provides more reasonable result in depicting the actual slope failure. In so doing, a series of constant-volume DSB tests with different degree of initial shear loading (i.e., different i values) may be carried out by using soil retrieved adjacent to the slip surface.

6. Conclusions

Case record of slope failure that took place in Hyogo Prefecture, Japan in February 2007 is described in detail in this paper. According to the careful observations, the slope failure was seemingly triggered by increase in ground water level induced by heavy snowfall in January. Continuous ground movement was observed over a few months before

Table 3. Results of stability analysis

Analysis Condition	Composition critical surface model	Circular critical surface model
(1)	0.53	0.56
(2)	***1.05***	–
(3)	1.10	–
(4)	***1.05***	1.152

the catastrophic failure occurred. Conversely, the slope failure took place over a short period of about one minute.

To back analyze the slope failure, a series of constant-volume direct shear box tests were performed by using the reconstituted soil sample retrieved on the slip surface. In order to simulate closely the stress history of soil element on the slip surface, the soil sample before undrained shear experienced in-situ shear stress under drained conditions. The sample was then sheared under constant-volume (i.e., undrained) conditions by using a fast rate of shear displacement of 1mm per minute.

In the stability analysis using Fellenius method, the factor of safety by using the strength parameters from the conventional DSB test (i.e., no initial shear loading was applied before undrained shear) yielded unrealistically low values being far less than unity. Conversely, the stability analysis by using the strength parameters from the DSB test with initial shear loading provided most reasonable result with the factor of safety being close to the unity. In the design of slope stability problem, it may therefore be suggested to perform a series of constant-volume DSB tests with different degree of initial shear loading so as to obtain properly the profile of undrained shear strength along the slip surface.

Similar case histories should be further carried out in order to generalize the perspective described in this paper.

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