

INVESTIGATION AND STABILITY ANALYSIS OF SLOPE FAILURE AT EXTREMELY SHALLOW LAYER AFTER SNOWMELT IN SNOWY COLD REGIONS

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ABSTRACT: In snowy cold regions such as Hokkaido, slopes can collapse owing to the frost heave phenomenon, freeze-thaw actions in winter, or snowmelt in spring. Kitami City in Hokkaido recently suffered from such slope failures when the extremely shallow layer eroded and slid after snowmelt in the early spring of 2020. The depth of this slope failure was about 0.20 to 0.25 m, which was very shallow compared with general surface failures. In this research, we conducted a site investigation of the road slope and collected undisturbed soil samples from a non-collapsed area near the slope failure to conduct a constant pressure direct shear test. For the direct shear tests, specimens with natural water content at the time of sampling and forcibly saturated specimens were used. The results of the slope stability analysis with the strength parameters obtained from the direct shear test showed that the slope failure was caused by snowmelt penetration, mainly due to a decrease in cohesion.

Keywords: Snowy cold region, Slope failure, Direct shear test, Slope stability analysis

1. INTRODUCTION

In snowy cold regions such as Hokkaido in Japan, slopes can collapse owing to the frost heave phenomenon, freeze-thaw actions in winter, or snowmelt in spring. Many incidents of slope failure along roadways of Hokkaido were reported (e.g. Subramanian et al. [1], Mori et al. [2]). Such a slope failure also occurred in the early spring of 2020 on a road slope in Kitami City, Hokkaido. The depth of this slope failure was about 0.20 to 0.25 m, which was very shallow compared with general surface failures. In this research, we conducted a site investigation of the road slope and collected undisturbed soil samples from a non-collapsed area near the slope failure to conduct a constant pressure direct shear test. In addition, slope stability analysis of the infinite length assumptions without seepage was conducted using the strength parameters obtained from the direct shear tests, and the factors that led to the failure were discussed.

2. SITE INVESTIGATION OF THE ROAD SLOPE WHERE SURFACE FAILURE OCCURRED

2.1 Meteorological Conditions Before and After the Day When the Slope Failure Was Observed

The slope failure was observed on April 3, 2020.

Fig.1 shows the overall appearance of the road slope where the failure occurred. Since the failure

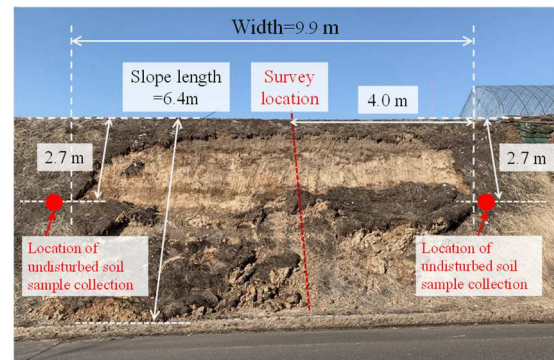


Fig.1 Overall appearance of the road slope where the failure occurred

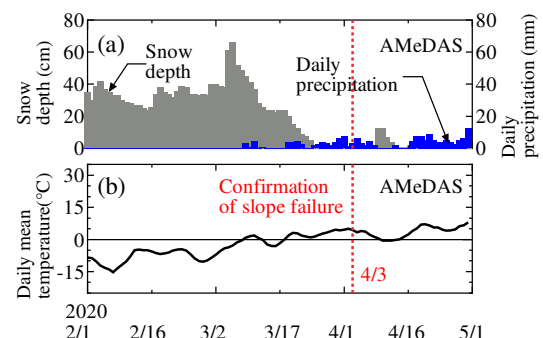


Fig.2 Daily mean temperature, snow depth, and daily precipitation around April 3, 2021

surface is a little dry, it is assumed that the slope failure occurred a little earlier than this date.

Fig.2 shows the daily mean temperature, snow

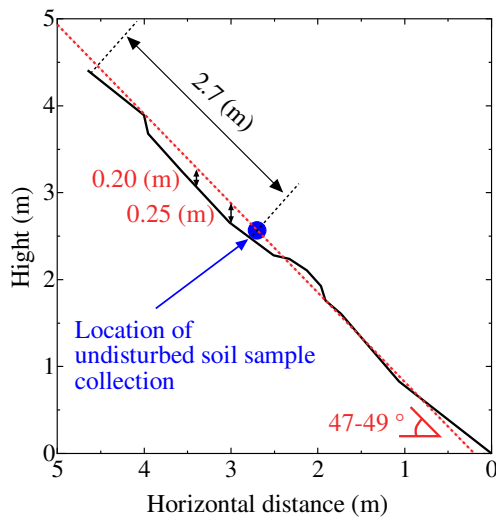


Fig.3 Cross-sectional view of the slope failure obtained by surveying

depth, and daily precipitation around April 3, 2020, when the slope failure was observed. The meteorological data were observed at the AMeDAS Kitami station [3]. From this figure, it can be seen that the snow depth has been decreasing rapidly since mid-March when the daily mean temperature became positive. On the other hand, daily precipitation before the date of confirmed slope failure is low. Based on the above meteorological data, the trigger for this slope failure is thought to be the infiltration of snowmelt.

2.2 Overview of the Investigated Road Slopes

The site investigation, including a collection of undisturbed soil samples, was conducted from May 15 to June 5, 2020. The road slope is a cut slope facing south. The height is about 4.4 m, the angle is 47 to 49 °, and the upper part of the slope is flat and used as cultivated land. The surface of the slope was covered with vegetation work to prevent erosion, and a metal net was laid on top of it.

Fig.3 shows a cross-sectional view of the slope failure obtained by surveying. As shown in the figure, the depth of the failure surface (vertical depth) was about 0.20 to 0.25 m. This figure also shows the locations where the soil samples were collected.

2.3 Soil test results of Collected Soil Samples

Fig.4 shows a vertical cross-section of a non-collapsed area near the slope failure (2.7 m below the slope top). From site observations, it was confirmed that the soil layer from the slope surface to a depth of 0.5 m was slightly weathered sandy soil. The soil samples were collected from this section at intervals of 0.1 m in the vertical direction, and the grain size distribution, soil particle density,

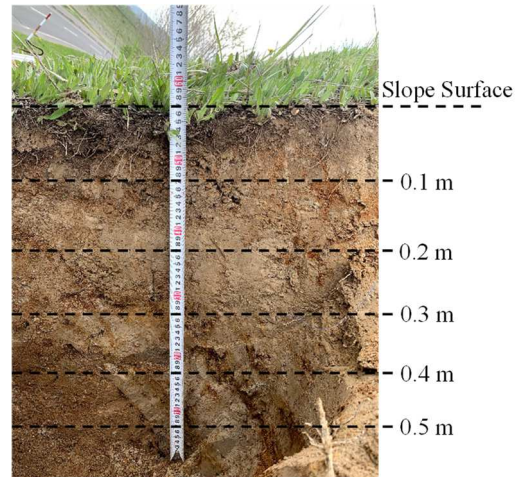


Fig.4 Vertical cross section of a non-collapsed area near the slope failure

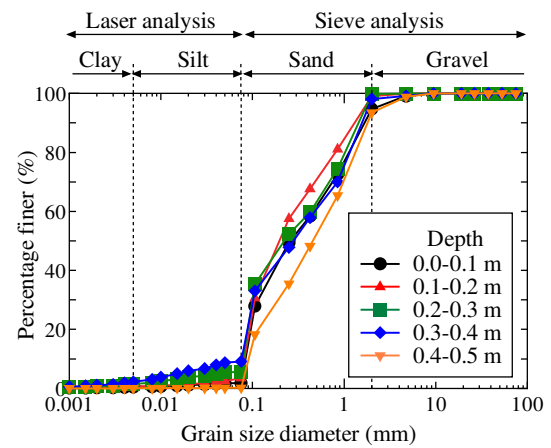


Fig.5 Grain size accumulation curve

and water content were measured. All of the respective soil tests were conducted by the Japanese Geotechnical Society Standards [4-6].

Fig.5 shows the grain size accumulation curve of soil samples collected from the non-collapsed area. In this study, the grain size distribution was determined by sieve analysis for grain sizes larger than 0.075 mm and by the laser diffraction method for smaller grain sizes.

Fig.6 shows the fine fraction content F_c , soil particle density ρ_s , and water content w at each depth. The soil samples from 0.1 to 0.4 m depth contained a small amount of fine-grained material, while soil samples from 0.4 to 0.5 m depth contained no fine-grained material and a small amount of coarse-grained material.

2.4 Collection of Undisturbed Soil Samples

The undisturbed soil samples for the constant pressure direct shear tests were collected from the non-collapsed area. An acrylic cylinder with an inner diameter of 60 mm attached to a sharp-edged

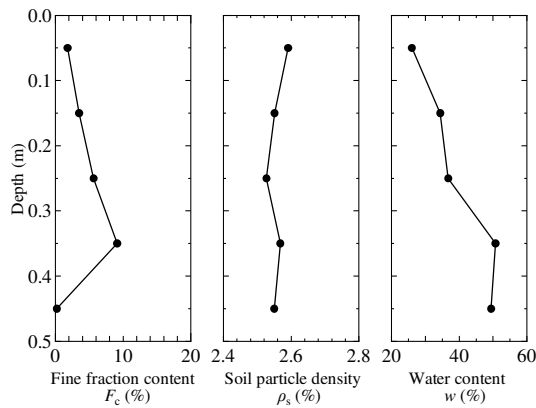


Fig.6 Fine fraction content F_c , soil particle density ρ_s and water content w at each depth

shoe was used. Soil samples were collected undisturbedly by pressing this cylinder slowly and perpendicularly against the slope surface.

3. CONSTANT PRESSURE DIRECT SHEAR TEST

3.1 Specimens

In this study, the constant pressure direct shear tests were conducted to obtain the strength parameters (cohesion c , internal friction angle ϕ).

Table 1 summarizes the experimental conditions for the four cases conducted in this study. Two types of soil samples were used in the experiments: one was collected from a depth of 0.2 m near the failure surface, and the other was collected from a depth of 0.4 m, deeper than the failure surface. And also, two types of soil moisture conditions were used in the experiments: one was a natural water content state (unsaturated state), and the other was a forcibly saturated state.

The specimens were prepared by the following procedure. First, the bottom surface of the undisturbed soil sample in the acrylic cylinder was cut and smoothed. Then, the sample was extruded from the acrylic cylinder using a piston and transferred directly into a shear box with the same inner diameter and a depth of 20 mm. Finally, the top surface of the specimen in the shear box was cut and smoothed. The size of the specimen was 60 mm in diameter and 20 mm in height, which is the Japanese Geotechnical Society Standards [7].

Some of the specimens were forcibly saturated

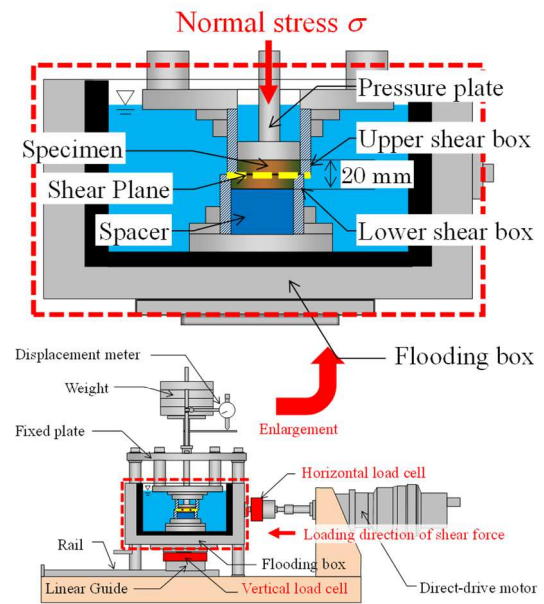


Fig.7 Schematic diagram of the constant pressure direct shear test apparatus

using the following procedure. First, the specimen was placed in an acrylic cylinder, sandwiched between porous plates at the top and bottom, and submerged in a container filled with distilled water. Then, the container was placed in a separate pressure-resistant container and degassed with a vacuum pump. This degassing process took about 96 hours. Finally, the specimen was placed in distilled water for more than a week to increase the saturation.

3.2 Test Apparatus And Test Method

Fig.7 shows the schematic diagram of the constant pressure direct shear test apparatus.

In this study, the change in normal stress on the shear plane due to frictional forces on the inner surface of the shear box was monitored by a vertical load cell. During the direct shear test, the constant pressure was controlled by increasing or decreasing the weight to reflect the change in normal stress.

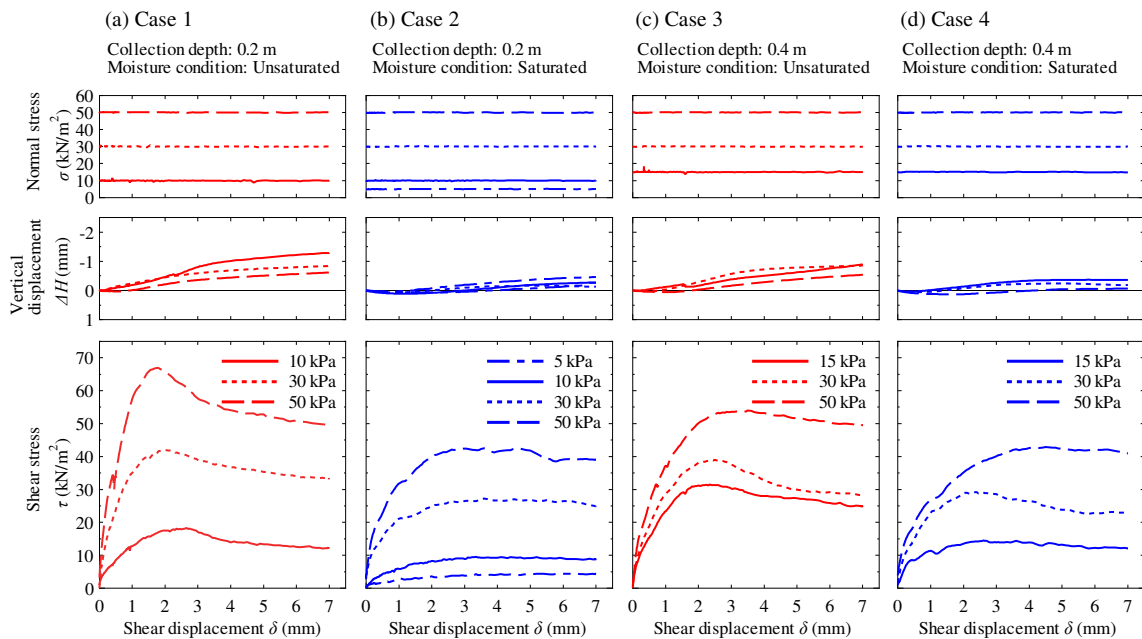
Table 1 also summarizes the vertical stresses established in the direct shear test. Normal stresses were applied to the specimens from the consolidation process until the end of the direct shear test. Among the specimens collected near the failure surface, the saturated specimens were tested with the small normal stress of 5 kN/m² because the

Table 1 Experimental conditions for the four cases conducted in this study

	Case 1	Case 2	Case 3	Case 4
Collection depth (m)	0.2	0.2	0.4	0.4
Moisture condition	Unsaturated	Saturated	Unsaturated	Saturated
Normal stresses (kN/m ²)	10, 30, 50	5, 10, 30, 50	15, 30, 50	15, 30, 50

Table 2 All direct shear test results conducted in this study

	Case 1				Case 2				Case 3			Case 4		
Normal stress σ (kN/m ²)	10	30	50	5	10	30	50	15	30	50	15	30	50	
Water content w (%)	30.1	31.6	32.3	64.4	49.4	51.3	44.3	39.0	38.1	36.0	55.9	49.4	50.1	
Dry density ρ_d (g/cm ³)	1.04	1.03	1.00	1.09	1.18	1.12	1.20	1.07	1.12	1.05	1.08	1.11	1.08	
Wet density ρ_t (g/cm ³)	1.35	1.35	1.33	1.78	1.76	1.70	1.73	1.49	1.60	1.42	1.68	1.66	1.62	
Void ratio e	1.45	1.48	1.55	1.35	1.17	1.27	1.15	1.39	1.22	1.46	1.38	1.31	1.37	
Degree of saturation S_r (%)	52.8	54.5	53.3	100.0	100.0	100.0	99.1	71.8	80.2	63.5	100.0	96.9	93.8	
Maximum shear stress τ_{max} (kN/m ²)	18.3	42.0	66.9	4.4	9.3	26.1	42.4	31.5	39.0	54.0	14.4	29.2	42.7	


 Fig. 8 Comparison of normal stress σ , vertical displacement ΔH and shear stress τ with shear displacement δ for each specimen under constant pressure direct shear test

change in normal stress due to an increased friction force was small (Case 2). On the other hand, the minimum normal stress for the specimens collected deeper than the failure surface was 15 kN/m² due to the large change in normal stress due to the increase in frictional force (Case 3&4).

Shearing of the specimens was performed at a constant speed by a direct-drive motor installed outside the flooding box. The volume change during the test was measured by a displacement meter attached to the vertical loading axis. The top and bottom shear box spacing were 0.2 mm, the shear displacement rate was 0.02 mm/min, and the maximum horizontal displacement was 7 mm. And the shearing position was set at 10 mm above the

bottom of the specimen.

The shearing process was conducted after a consolidation process of about 24 hours. For the saturated specimens, the flooding box was filled with distilled water before consolidation.

4. RESULTS AND DISCUSSION OF CONSTANT PRESSURE DIRECT SHEAR TEST

Table 2 summarizes the all test results conducted in this study.

Fig.8 shows the comparison of normal stress σ , vertical displacement ΔH and shear stress τ with shear displacement δ for each specimen under

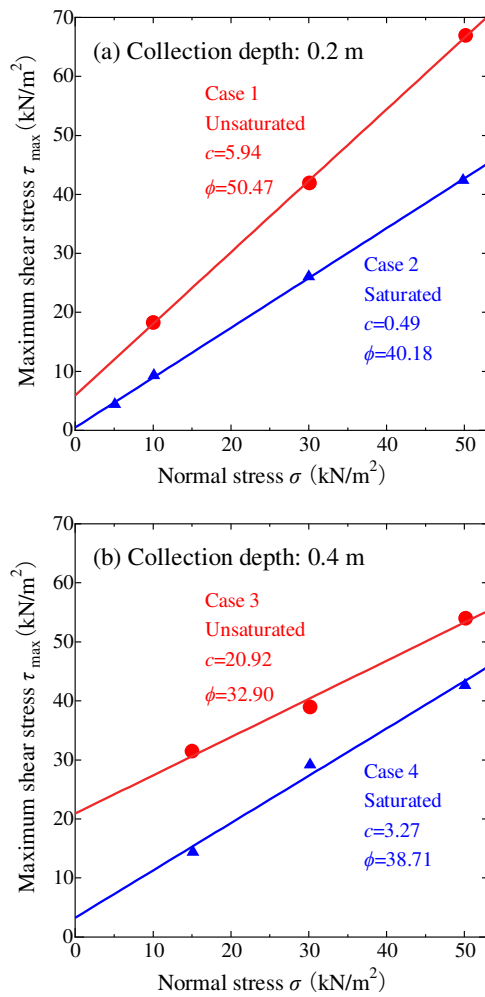


Fig. 9 Relationship between the maximum shear stress τ_{max} and the normal stress σ

constant pressure direct shear test. It is confirmed that the normal stress σ during the direct shear test can be controlled to be almost constant in all tests.

The comparison of the shear stresses τ in the unsaturated and saturated states of the specimens collected from near the failure surface (Fig.8 (a), (b)) shows that the maximum shear stress τ_{max} decreases as the specimens become saturated. This is because the suction decreases and disappears as the specimen becomes saturated. Even for specimens collected from a depth deeper than the failure surface (Fig.8 (c), (d)), the maximum shear stress τ_{max} decreases as the specimen saturates.

Comparing the shear stress τ in the saturation state of the specimens collected from near the failure surface and those collected from a depth deeper than the failure surface (Fig.8 (b), (d)), it can be seen that the maximum shear stress τ_{max} of the specimens collected from a depth deeper than the failure surface is higher.

Fig.9 shows the relationship between the maximum shear stress τ_{max} and the normal stress

Table 3 Input conditions for the stability analysis

	Case 1	5.94
Cohesion	Case 2	0.49
c (kN/m ²)	Case 3	20.92
	Case 4	3.27
Internal friction angle	Case 1	50.47
ϕ (°)	Case 2	40.18
	Case 3	32.90
	Case 4	38.71
Unit volume weight	Case 1	13.17
in a wet condition	Case 2	16.91
γ_t (kN/m ³)	Case 3	14.72
	Case 4	16.23
Assumed		
slip surface angle	All cases	48
α (°)		

σ at that time for each test shown in Fig.8. The figures also show cohesion c and the internal friction angle ϕ obtained from each specimen. For the specimens collected from near the failure surface, both cohesion c and the internal friction angle ϕ decrease with increasing saturation (Fig.9 (a)). On the other hand, for specimens collected from a depth deeper than the failure surface, the internal friction angle ϕ increased slightly with saturation, but cohesion c decreased significantly (Fig.9 (b)).

5. SLOPE STABILITY ANALYSIS OF THE INFINITE LENGTH ASSUMPTIONS WITHOUT SEEPAGE

To investigate the effect of changes in strength parameters due to snowmelt infiltration on the stability of the slope, stability analysis of the infinite length assumptions without seepage shown in Eq. (1) was conducted using strength parameters obtained from the direct shear tests.

$$F_s = \frac{c}{\gamma_t \cdot z \cdot \sin \alpha \cos \alpha} + \frac{\tan \phi}{\tan \alpha} \quad (1)$$

where F_s is the factor of safety, c is cohesion, ϕ is the internal friction angle, z is the assumed slip surface depth, and γ_t is the unit volume weight in a wet condition, α is the assumed slip surface angle. The infinite length assumptions are widely used to predict the slope failure probability (e.g. Haefeli. [8], Skempton and DeLory. [9]).

Table 3 shows the input conditions for the stability analysis. In this analysis, the average angle of this slope 48 ° was used as the assumed slip surface angle α .

Fig.10 shows the results of the slope stability

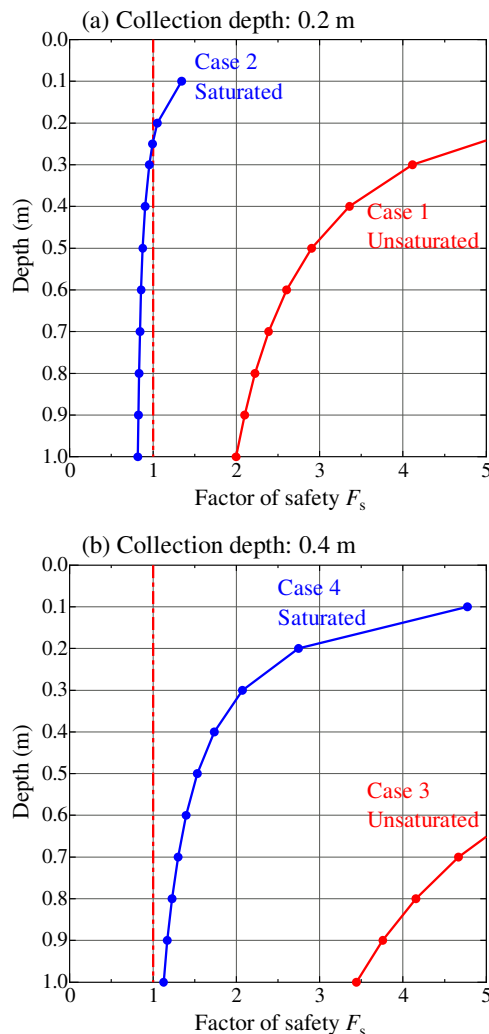


Fig.10 Results slope stability analysis of the infinite length assumptions without seepage

analysis of the infinite length assumptions without seepage. It can be seen that the factor of safety F_s in the saturated state is lower than that in the unsaturated state for both specimens collected from the failure surface and specimens collected from a depth deeper than the failure surface. In particular, for the specimens collected from the failure surface in the saturated state, the factor of safety F_s is below 1 when the depth was deeper than 0.25 m (Fig.10 (a)). This suggests that the surface layer of the slope was extremely unstable.

It can be understood that the decrease in cohesion c contributes significantly to the decrease in the factor of safety F_s due to the increase in saturation. These results suggest that the main cause of slope failure in this study is the decrease in cohesion c caused by the infiltration of snowmelt.

On the other hand, the factor of safety at a depth of 0.4 m for specimens collected from a depth deeper than the failure surface in the saturated state

is always greater than 1 (Fig.10 (b)). This reason could be that the soil deeper than 0.4 m was not subjected to freezing and thawing history during the winter of 2019. Kawaguchi et al. [10] conducted the constant pressure direct shear tests on soil samples subjected to freezing and thawing and reported that freezing and thawing history reduces the shear strength of the soil. Because the slope failure occurred during the snowmelt season, the freezing and thawing history is considered to have had a significant influence. In the future, we would like to clarify the effects of frost heaving and other phenomena.

6. CONCLUSION

In this research, we conducted a site investigation of the road slope and collected undisturbed soil samples from a non-collapsed area near the slope failure to conduct a constant pressure direct shear test. For the direct shear tests, specimens with natural water content at the time of sampling and forcibly saturated specimens were used. The results of the slope stability analysis with the strength parameters obtained from the direct shear test showed that the slope failure was caused by snowmelt penetration, mainly due to a decrease in cohesion.

We will continue to investigate slope failures and conduct soil tests to study the mechanism of slope failures at the extreme surface layer that occurs in snowy cold regions.

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