■ Characteristics of landslide and failure with their Countermeasures in Kanmon-group Slopes, Yamaguchi prefecture, Western Japan

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1. Introduction

Lake deposit of the Cretaceous period of the Mesozoic Era (100 to 150 Ma), which is called Kanmon group, spreads over a large area in Yamaguchi Prefecture, which covers Hagi city and Shimonoseki city, or the northwestern and southwestern parts of the Prefecture. The layers mainly consist of tuffaceous sandstone, shale and conglomerates and alternative layers thereof. Reddish shale found in Shimonoseki district in the southwestern part of the Prefecture is relatively little weathered, and is used as bedrocks of dams and to produce inkstones, which are a local specialty. On the other hand, these rocks in the northeastern part (Hohoku-cho to Hagi city) are highly weathered down to deep underground, and are clayey. Fig.1 shows the area over which Kanmon group is distributed in Yamaguchi Prefecture. Weathering of rocks in deep Kanmon-group layers are believed to be caused by the following phenomena: The tuffaceous sandstone and shale were formed in relatively old times in the Cretaceous period of the Mesozoic Era. The layers are gently inclined and show folding structures of long fre-

quencies. Rain water has seeped into the anticlines of the folding structures, and, in the long course of time, has weathered the rocks not only physically but has also chemically and converted the rocks into clay.

Due to the weathering to deep underground, the area over which Kanmon group spreads shows a landform characterized by gentle hills. In recent years, large-scale cutting projects of hill slopes have been executed to construct and improve principal highways and bypasses and develop farmlands. Of so formed cut slopes, those that had an inclination of 1:1.0 and exceeded 15 m in height were found to be prone to landslide and slope failure during storms.

This paper summarizes site surveys of 10 landslides and slope failures of relatively large scales that were investigated by the authors in and after 1989. Stability analysis of cut slopes consisting of highly weathered Kanmon-group layers was also conducted using the ordinary Fellenius method, shear strength parameters, and data of underground water level. The stability was also analyzed by the finite element method for comparison. Six possible causes of the failure of cut slopes

Fig. 1 Locations of the surveyed landslides and slope failures in the Kanmon group in Yamaguchi prefecture

consisting of weathered Kanmon-group rocks were analyzed by investigating the relationships between the factors and failure for each slope collapsed.

2. Ten landslides and slope failures in Kanmon-group area

Relatively large-scale landslides and slope failures of Kanmon-group slopes were observed at ten sites in the northern and northwestern parts of Yamaguchi Prefecture in and after 1989. The locations are shown in Fig. 1. The following section first describes the conditions of collapse for each slide and then describes the countermeasures taken as contributed in Table 1. The characteristics of landslides and slope failure of cut slopes consisting of Kanmon-group rocks are also summarized. In this paper, landslides denote movement of soil masses on relatively gentle slopes. Slope failure denotes movement of relatively small soil masses on relatively steep slopes.

2.1 Landslides, slope failures, and repair methods

(1) Futami

A section of a mountain slope was cut for road improvement. The lower part of the cut slope was concrete block retaining wall of an inclination of 1:0.4 and a height of 8.5 m, on which the soil was cut to an inclination of 1:1.0 and for a height of 15 m. After cutting, cracks appeared on the slope surface and concrete blocks in 1982, and restraining strength of approximately 200 kN/m became necessary. As a countermeasure, two courses of concrete grating crib and anchoring works were performed at 3-m intervals. In August 1994, during a storm that continued for three days and

brought a total rainfall of 247 mm, a large-scale landslide occurred, and the entire slope extending from the top of the hill to the river collapsed. The landslide was 150 m in length, 67 m in width, and 18 m in depth. A schematic diagram of the landslide is shown in Fig. 2. As the figure shows, unlike other landslides of Kanmon-group layers, the sliding surface of the landslide was in the fracture rocks of CL-class. A relatively large-scale fault was estimated to exist on the head of the slope. To provide the necessary restraining strength of 2,090 kN/m, a drainage well of 3.5 m diameter and 16.5 m depth was constructed, and four courses of ground anchorage (860 kN each course) were installed at 3-m intervals. Precast concrete panels were used as the pressure plates of the ground anchorage.

(2) Tashiro

Both sides of a ridge were cut in 1990 to 1992 to construct a road. A landslide occurred during the construction, and the land was cut again into a gentle slope of 1:1.8. However, in May 1998, when storms of daily rainfall of 80 mm resulted in a total rainfall of 313 mm for 20 days, a landslide occurred again. The landslide occurred in a highly weathered sandstone layer

Fig. 2 Geological section of Futami

Table 1 Landslides and slope failures occurred in Kanmon formation group from 1989 to 2002

No.	Location	Date of occurrence	Cause	Amount of rainfall	Exploit declivity, incline and height	Type of sliding surface	Size of landslide			Inverse analysis method				Countermeasure	Landslide or slope failure
							length (m)	width (m)	depth (m)	Safety factor	Condition of water	c (kN/m ²)	ϕ (°)		
1	Futami	Aug., 1994	Head fault	247mm/3days	Cut45° $\theta=21^\circ$ h=24m	Dual arc and line	150	67	18	0.98	Sliding Surface +5m	17	15.65	Ground anchor N=67, 1 Well ($d=16.5m$)	Landslide
2	Tashiro	May, 1998	Hard weathering	80mm/day 313mm/20days	Cut22° $\theta=25^\circ$ h=35m	Dual arc and line	39	43	5	0.98	Surface -2m	5	17.2	Bank Rock bolt	Landslide
3	Takibe nakamura	Jun., 2001	Weather and black soil	Heavy rain 184mm/3days	Cut37° $\theta=30^\circ$ h=18m	Straight line	35	80	5	0.95	Surface	4	30	Re-cutting slope (23°)	Landslide
4	Akanumata	Aug., 1994	Weather and black soil	247mm/3days	Cut31° $\theta=23^\circ$ h=23m	Arc	35	90	5	0.98	Surface -1.8m	4.4	25	Re-cutting slope (32°) Boring drain	Landslide
5	Oba	Jul., 2002	Weather, fault and black soil	Non observed	Cut42° $\theta=39^\circ$ h=12m	Dual arc and straight line	18	25	2	0.95	Non observed	2.3	28.3	Re-cutting slope (40°) Rock bolt Geofiber	Slope failure
6	Hatada	unknown	Weather and fault	Rainfall repetition	Natural slope 35° h=22m	Arc and straight line	20	17	3	1.00	Surface -2m	4.5	22	Ground anchor n=21, Re-cutting (43°)	Slope failure
7	Nobase	Oct., 2002	Weather, fault and black soil	55mm/3days	Cut44° $\theta=43^\circ$ h=18m	Arc and straight line	21	29	3	0.90	Surface -1m	2	33.9	Re-cutting (28°)	Slope failure
8	Hirawarabi	Jun., 1997	Weather and fault	195mm/day	Cut42° $\theta=40^\circ$ h=19m	Arc	16	18	3	0.95	Surface -2m	4.5	31.05	Re-cutting (42°) Concrete mesh	Slope failure
9	Takibe shiroyama south	Jun., 1989	Weather, fault and black soil	121mm/3days	Cut40° $\theta=32^\circ$ h=19m	Straight line	23	70	6	0.98	Sliding Surface +2m	1.1	14.9	Re-cutting (24°)	Landslide
10	Takibe shiroyama north	June, 1989	Weather, fault and black soil	121mm/3days	Cut40° $\theta=32^\circ$ h=17m	Straight line	31	60	7	0.98	Surface	15	30	Re-cutting (27°)	Landslide

whose N-value was smaller than 10. The underground water level continued to be high after the storms, and the slope continued to slide. Repair and control measures were performed, including counter weight fill work, drainage boring to lower underground water level, and insertion of steel bars into grating cribs to reinforce the slope.

(3) Takibenakamura

Both sides of a ridge were cut in 2000 to 2001 to construct a bypass of the National Highway Rout 435. The process and conditions of the slope failure are shown in Fig. 3 and Photo 1, respectively. As shown in Fig. 3(a), a small-scale failure occurred first on the southern slope. The slope was cut again into an inclination of 1:2.2, and was reinforced with ground anchorage that used pressure plates of precast concrete panels. Exploratory boring was conducted on the northern slope, and core samples collected by Denison sampling were used for a triaxial test and non-axial shear test. The test results are shown in Table 2. As shown in Fig. 3(b), the northern slope was cut to an inclination of 1:1.2, and steel bars(1,100 bars, 3-m long each) were inserted to restrain a model small-scale failure(Fig. 3(c)). However, in June 2001, during a three-day storm of a total rainfall of 184 mm, a large-scale landslide occurred(Fig. 3(d)). Near the head of the landslide section, thin black colored soil was observed adhering to

the sliding surface as shown in Photo 2(Yamamoto *et al.*(2000), Yamamoto *et al.*(2001), Yamamoto *et al.*(2002)). However, in the other parts, the sliding surface consisted of only white clay, suggesting that not the entire sliding surface passed though the thin black colored soil of small strength parameters(internal friction angle ϕ , cohesion c).

According to the fluctuations in strain, which was monitored by a pipe strain gauge installed within a bore pits, and the positional relationship between the head and toe of the slope, the sliding surface was con-

Table 2 The strength parameters of each soil sample (Takibenakamura)

	Sample No.	Test condition	c (kN/m^2)	ϕ ($^\circ$)
Thin black colored soil	①	Triaxis(UU test)	11.0	20.4
	②	Triaxis(CU test)	3.0	31.3
	③	Direct shear (Natural)	21.0	26.2
	④	Direct shear (Submerged)	8.8	22.5
Other clay	⑤	Triaxis(UU test)	24.0	28.4
	⑥	Triaxis(CU test)	18.0	37.7
	⑦	Direct shear (Natural)	37.3	26.4
	⑧	Direct shear (Submerged)	25.5	25.6

Fig. 3 Landslide and its countermeasure series at Takibenakamura

Photo 1 Whole view of Takibenakamura landslide

cluded to be linear, suggesting the existence of discrete surface. The failure occurred by the landslide mass slipping through the reinforcing bars. As shown in Photo 3, the bars were left behind with the ends bent downwards, and some were dragged into the soil mass. A cause of the failure was that the heads of the reinforcing bars were not joined by concrete grating crib works, etc. As an ultimate repair and control measure, the slope was cut again to a stable inclination of 1:2.2, which was calculated by conducting stability analysis of Fellenius slope critical circle method using the shear strength of the thin black colored soil when immersed ($c=8.8$ kPa, $\phi=22.5^\circ$) (Fig. 3(d)).

(4) Akanumata

The mid-slope of a hill was cut in 1990 to 1992 to develop farmlands. A landslide of this slope is shown in Fig. 4 and Photo 4. Although the slope was near the head of the hill, there was a thick layer of talus deposit. The talus deposit was the cause of a landslide that occurred during the construction of farmlands, and the slope was stabilized by cutting into a slope of 1:1.5.

Photo 2 Black colored thin soil on sliding surface
(At the Head cliff of Takibenakamura)

Photo 3 Reinforce bar folding at sliding cliff
(At Takibenakamura)

However, during the storm that triggered the landslide in Futami District in August 1994, a landslide occurred (Photo 4). The geological structure of the bed rock was a stratum of opposite dip, and was different from the dip slope structures in the other landslides. The underground water level after a rain was monitored and was found to be high and near the ground surface. Countermeasures were taken, including drainage boring, cutting of the slope near the head, and insertion of H-steels at 1.5 m intervals for a length of 11.5 m. (5) Oba

During a road improvement project in 2003 and 2004, which involved cutting of a hill slope by the National Highway Route 435 to an inclination of 1:1.0, a slope failure occurred at the lower part of the slope and developed into a relatively large slope failure during subsequent cutting. As shown in a lateral profile in Fig. 5, the soil layers are inclined almost parallel to the cut slope. The sliding surface was generated along the border between shale and highly continuous tuff, which contained layers of white clay of weathered shale. A fault was observed on the top of the slope along the direction of the failure's head. Unlike other landslides and slope failures, this landslide was not triggered by rain. The underground water level was 2 to 5 m under the sliding surface and was so deep that it should not have affected the stability of the slope. To provide the neces-

Fig. 4 Geological section of Akanumata

Photo 4 Whole view of Akanumata landslide

sary restraining strength of 66 kN/m, 13 courses of steel bars were inserted for 3.0 to 4.5 m at 1.5-m intervals into reinforced fiber soil, were clamped at the surface of the reinforced fiber soil, and were joined at the heads.

(6) Hatada

The mid-slope of a narrow ridge was cut to construct a spillway for an earth dam. The narrow ridge extended along a fault, and traces of two slope failures were observed on the slope to be cut. The traces and a geological profile of the slope are shown in Photo 5 and Fig. 6, respectively. The time of the failures and the amount of rainfall during the failures are unknown. The backland of the slope that fell was near the top of the ridge, and the supply of underground water should have been little. Thus, the risk of underground water affecting the stability of the slope was judged to be small. To provide the necessary restraining strength of 186 kN/m, 21 ground anchors (238 kN per anchor) and steel pressure plates were installed.

(7) Nobase

A slope was cut during a road improvement project in 2003. Shale that was exposed at the roadside was fresh. The inclination of the cut slope was planned to be 1:0.4 for the lower section, and 1:1.0 for the upper section. However, as shown in Fig. 7, a relatively large-

scale fault was found in the middle of the slope, and had caused the upper section of the slope to weather completely, where no non-weathered rocks were found. The fault also causes the groundwater level to rise and maintained it high although the site is on a ridge and there is no source of underground water. The slope failure occurred when it rained little, or 55 mm in three days. As a countermeasure, the slope was cut again into a gentler inclination of 1:1.7.

(8) Hirawarabi

Slice-cutting of a slope was planned for a road improvement project. However, in July 1997, before the project started, the slope collapsed during a storm of 195 mm in daily precipitation. The slope failure is

Fig. 6 Geological section of Hatada

Fig. 5 Geological section of Oba

Fig. 7 Geological section of Nobasa

Photo 5 Whole view of Hatada slope collapse

Photo 6 Whole view of Hirawarabi slope collapse

shown in Photo 6. A fault was estimated to exist near the head of the fallen section, and the strata showed a dip slope structure, which is prone to destabilizing the slope. The groundwater level monitored was deep enough to not affect the stability of the slope. As countermeasures, the slope was cut into the initially planned inclination of 1:1.0, the landslide mass was removed, and spray concrete grating crib works were applied to protect the slope.

(9) Takibeshiroyama South

A new farm road construct project was executed by cutting both sides of the ridges in 1989. During a storm that brought a total rainfall of 121 mm in three days, large-scale landslides occurred on the both slopes. The landslide on the southern slope is shown in Photo 7. The soil of the slope was highly weathered tuff of N-values smaller than 15. The sliding surface was almost linear, suggesting it passed through a discrete surface. The land behind the top of the slope was reversely inclined, thus the groundwater level monitored was low and was judged to have little effects. As countermeasures, drainage boring was conducted, and the slope was cut again into a stable inclination of 1:1.8.

Photo 7 Whole view of Takibeshiroyama south landslide

Photo 8 Whole view of Takibeshiroyama north landslide

(10) Takibeshiroyama North

A landslide also occurred on the opposite slope of the aforementioned slope, when it rained 121 mm in three days during the road improvement project. The landslide is shown in Photo 8. The soil of the slope was highly weathered tuff of N-values smaller than 15. Since the backland was deep, the groundwater level was high even after the landslide. The sliding surface was linear as in the aforementioned landslide, suggesting it passed through a discrete surface. The slope was cut again into an inclination of 2.0, and spray concrete grating crib works and reinforcing micro piles were installed to stabilize the slope.

2.2 Characteristics of the landslides and slope failures in the Kanmon-group area

The characteristics of large-scale landslides and slope failures that occurred in the Kanmon-group area were investigated by summarizing the time, factors, causes, scale, configuration, strength parameters of the soil determined by inverse analysis, and countermeasures taken for each landslide and slope failure (Table 1). As the table shows, all except the one in Futami (nine out of ten) occurred at highly weathered soil layer, and all except the one in Oba and the one in Hatada, whose rainfall was unknown, (eight out of ten) were triggered by storms.

The shape of the sliding surface was linear in (3) Takibenakamura, (9) Takibeshiroyama South, and (10) Takibeshiroyama North. In the other seven districts, the sliding surfaces were composite of circular and linear shapes. Fig.8 shows the relationship between the length and width of the landslides and slope failures. The result of (1) Futami is excluded since it was a rockslide. The average relationship for ordinary landslides is also shown in the figure with a dashed line

Fig. 8 Relation between length and width in landslides and slope failure

(Express Highway Research Foundation of Japan (1985)). Compared to this line of 1:1.2 in length:width, most of the landslides and slope failures in this area were wide and shallow compared to their length. Their mean length was 16 to 39 m, the mean width was 17 to 90 m, and the mean depth was 2 to 7 m. From these mean values, the length:width relationship was calculated to be 1:1.8, length:depth was found to be 1:0.16, and width:depth was 1:0.04. The inclination of the slopes was 22 to 45°, and was 32° on average, suggesting that landslides also occurred on gentle slopes. The results of (2) Tashiro and (9) Takibeshiroyama South were quite different from the others, but the causes are not understood. Fig. 9 shows

Fig. 9 Relation between slope height and slope inclines of landslides

Fig. 10 Relation between $\tan \phi$ and c obtained by the inverse analysis method for landslides and slope failures

the heights and inclinations of slopes that collapsed and the height and inclination of the cut slopes after countermeasures were taken. The relationship between the height and inclination of slopes was found to closely correlate to failure, and a border was likely to exist at the critical failure line shown in the figure. Shear strength values determined by inverse analysis are shown in Figure 10. The values are in the range of $c=1.1$ to 15 kN/m^2 and $\phi=14.9$ to 30° . Although the values varied, c tends to increase as $\tan \phi$ decrease. This indicates that c increases and ϕ decreases as rocks are weathered and become clayey. The values for (1) Futami, (9) Takibeshiroyama South, and (10) Takibeshiroyama North were quite different from the others likely because they differed in landslide type. The slide in Futami was a rockslide, and the slides in Takibeshiroyama South and North had gently inclined linear sliding surfaces. The fluctuations in value was likely attributable to differences in various conditions, such as groundwater levels varying from saturated to no groundwater and different rainfall conditions. Another characteristic of landslides and slope failures in this area is that black colored thin soil is frequently observed along the joints that form discrete surfaces as indicated by Yamamoto *et al.* (2000, 2001, 2002).

3. Stability analysis of collapsed slopes

3.1 Modified Fellenius method

Although black colored thin soil was observed not in all the districts, discrete surfaces that were very low in shear strength need to be considered for all landslides and slope failures except for the one in Futami since the mechanisms of collapse are difficult to explain just by storms and extensive weathering of rocks. The shear strength at the discrete surface varied by site. As an example, the shear strength determined for the analysis of the landslide slope of Takibenakamura is here described. The shear strength is a strength parameter of the soil along the discrete surface, and the strength of the black colored thin soil that had been immersed in water for one day was cohesion $c=8.8 \text{ kPa}$ and internal friction angle $\phi=22.5^\circ$. (Yamamoto *et al* 2002)

Safety factor F_s was calculated using the modified Fellenius method shown in Fig. 11 and the following equation provided by the Highway Earthworks Series: Manual for Slope Protection. Analysis was conducted using Power SSA ver. 1.15 of GODAI Development Corp. The resultant safety factor values are shown in Table 3.

Fig. 11 Safety factor analysis by Fellenius method

Table 3 Safety factor obtained using modified Fellenius method ($c=8.8\text{ kN/m}^2$, $\phi=22.5^\circ$)

No.	Location	Safety factor		
		Total stress	Effective stress	
		Safety factor	Safety factor	Ground water condition
1	Futami	1.43	0.98	Sliding Surface +5m
2	Tashiro	1.52	0.97	Surface -2m
3	Takibe nakamura	1.19	0.96	Surface
4	Akanumata	1.46	0.98	Surface -1.8m
5	Oba	0.94	Non observed	Non observed
6	Hatada	1.16	0.98	Surface -2m
7	Nobase	1.07	0.98	Surface -1m
8	Hirawarabi	0.97	0.95	Surface -2m
9	Takibe shiroyama south	1.54	0.98	Sliding Surface +2m
10	Takibe shiroyama north	1.06	0.98	Surface

$$Fs = \frac{\sum \{c \cdot L + (W \cdot \cos \alpha - u \cdot L \cdot \cos^2 \alpha) \cdot \tan \phi\}}{\sum W \cdot \sin \alpha} \quad (1)$$

where, c is cohesion, ϕ is internal friction angle, u is pore water pressure, W is the dead load of the soil in the divided section, L is the length of the sliding surface, and α is the horizontal angle of the sliding surface. In this stability analysis, groundwater levels that are almost saturated were used in principle except for the slides in (1) Futami, (5) Oba, (10) Takibeshiroyama North. The groundwater levels were not assumed to be saturated for these three cases due to the following reasons. The case of (1) Futami was a slide of fractured

Fig. 12 Schematic details of finite element mesh (Takibenakamura)

Table 4 Material parameters used Finite element method (Takibe nakamura)

Parameters	1st layer	2nd layer	3rd layer
	Strongly weathering part	Partial weathering part	Weathering part
Young modulus E(Mpa)	100	200	20000
Poisson's ratio ν	0.3	0.3	0.3
Cohesion c (kPa)	8.8	24	100
Internal friction angle ϕ ($^\circ$)	22.5	28.4	35
Dilation angle ϕ ($^\circ$)	0	0	0
K_0 ※	0.6	0.5	0.4

※ K_0 : Coefficient of earth pressure at rest

rocks, and it is difficult to assume a high groundwater level. In (5) Oba, no groundwater level was detected during the slope failure. In (10) Takibeshiroyama North, rises in groundwater level is difficult to occur in such a landform. As shown in Table 3, the safety factors were $Fs=0.95$ to 0.98 , and were similar to the safety factors during landslides proposed by Fujiwara *et al.* (1979).

The results suggest that this method, which uses cohesion $c=8.8$ kPa, internal friction angle $\phi=22.5^\circ$, and the possible maximum underground water level for the corresponding topographic conditions, was appropriate for analyzing landslides and slope failures in highly weathered Kanmon group.

3.2 Finite element method

Elasto-plastic finite element analysis was conducted on the case of (3) Takibenakamura to compare with the results of the modified Fellenius method. The analysis was conducted using Shamen@FE ver. 1.30, which is a computer program for saturated and unsaturated seepage analysis of slope stability that uses the non-linear finite element method. Finite element meshes and soil division are shown in Fig. 12. This is

the state before the large-scale landslide occurred on the cut slope of 18 m high and 1:1.2 in inclination. Material's constants are shown in Table 4. The shear strength of the first highly weathered layer was the value of black colored thin soil that had been immersed in water for one day as in the modified Fellenius method, and was cohesion $c=8.8$ kPa and internal friction angle $\phi=22.5^\circ$.

The maximum shear strain distribution and partial safety factor are shown in Fig. 13(a) and 13(b), respectively. As shown in Fig. 13(a), the maximum shear strain exceeded 0.2 at the highly weathered sections in the middle and at the toe of the slope. Strain concentrated near the sliding surface shown in Fig. 13(c).

Fig. 13(a) Distribution maximum shear strain value

Fig. 13(b) Partial safety factor

Fig. 13(c) Sliding surface and groundwater condition using for modified Fellenius method (Takibenakamura)

4. Risk factors for landslides and slope failures in the Kanmon Group area

Six risk factors may have been involved in the landslides and slope failures described in Section 2.1: 1) degree of weathering, 2) existence of black colored thin soil, 3) cut slope higher than 15 m, 4) existence of folding (especially syncline), 5) fault at the head, and 6) storm of over 100 mm in daily precipitation. Involvements of these factors are shown in Table 5 for each landslide and slope failure.

When there was no fault on the slope head, four or more risk factors were found to have involved in the collapse. When there was a fault on the slope head, two more risk factors were sufficient to cause the slope to collapse.

5. Conclusion

Following knowledge was obtained by site survey, stability analysis, and risk factor investigation on the landslides and slope failures that occurred on the Kanmon-group soil of the Cretaceous period of the Mesozoic Era in Yamaguchi Prefecture:

- 1) In 15 years after 1989, ten relatively large-scale landslides and slope failures occurred on weathered Kanmon-group soil. Especially, cutting of gentle slopes is prone to causing landslides, and careful survey and planning are necessary.
- 2) The landslides in the Kanmon-group area are wide and shallow compared to their length ($2.3L=W$, and $0.04W=d$), and have occurred even on gentle slopes.
- 3) A certain relationship was found between the

Table 5 Cause of landslides and slope failures in Kanmon group area

No.	Location	Strongly Weathered	Thin black soil	Cutting over 15m	Folding	Fault	Heavy rain
1	Futami	×	×	○	×	○	○
2	Tashiro	○	△	○	○	×	○
3	Takibe nakamura	○	○	○	○	×	○
4	Akanumata	○	○	○	○	×	○
5	Oba	○	○	×	×	○	×
6	Hatada	○	×	○	×	○	△
7	Nobase	○	○	○	×	○	×
8	Hirawarabi	○	×	×	×	○	○
9	Takibe shiroyama south	○	○	○	○	×	○
10	Takibe shiroyama north	○	○	○	○	×	○

○ : Related × : Not related △ : Unknown

height and inclination of slopes that have collapsed, and a critical failure line was obtained, which divides slopes into those that collapse and not collapse.

- 4) Stability analysis of the modified Fellenius method was conducted using strength factors of black colored thin soil (cohesion $c=8.8$ kPa and internal friction angle $\phi=22.5^\circ$), which were obtained by a direct shear test, and the possible maximum groundwater level for the corresponding topography. The resultant safety factor values were 0.95 to 0.98, and were adequate as safety factors during landslides and slope failures.
- 5) Sliding surface can be estimated by determining the ground section where the maximum shear strain of over 0.2 concentrates by conducting finite element analysis.
- 6) Weathered slopes of the Kanmon group were surveyed on six risk factors. Slopes that have a fault on the top and two more risk factors and slopes that have no faults but four or more risk factors are prone to collapsing. The results should be effective to determine the inclinations of cut slopes and slope restraining works while cutting highly weathered Kanmon-group slopes.

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