Journal of Nepal Geological Society, 2002, Vol. 26, pp. 73–77

Environmental-friendly countermeasures applied to control instabilities in a high vertical rock slope: an example from Oita prefecture, Japan

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ABSTRACT

An analytical method for the selection of countermeasures to control instabilities in a high vertical slope composed of welded tuff underlain by gravelly palaeo-channel deposit has been described. Based on the engineering geological investigation, discontinuity features namely, orientation, spacing and persistence representing the slope were established as input data for the DEM (distinct element method) model construction. The model analysis reveals that the studied slope is prone to sliding (slumping) accompanied by toppling failure, which may cause disaster by blocking the river at the toe of the slope. Construction of a concrete base structure at the toe of the slope is suggested to restrict sliding (slumping). But toppling and sliding of individual rock blocks at the upper part of the slope are allowed to take place naturally. This arrangement ensures long-term safety as well as maintenance of natural environment to reasonable limits and will encourage the local authority to offer the site as a place of learning natural phenomena and its method of preservation.

INTRODUCTION

Design of slope stabilization measures is an important task in rock engineering. Rock slopes at a site may be composed of single or multiple types of rock having similar or dissimilar origin. Furthermore, they are likely to be dissected by a veriety of discontinuities such as beddings, joints, faults and fractures. Accordingly, rock masses of a slope are not continuum and their behavior is dominated by various discontinuities. Because of these features the mode of slope failure is likely to vary from simple plane failure to complex type consisting of a combination of plane, wedge and toppling failures. Consequently, the type of counter measures to be applied for slope stabilization works vary from place to place and from project to project.

Unstable rock slopes may be stabilized by various types of retaining walls as has been explained by Hunt (1986). Rockbolts and cablebolts may also be used to prevent jointed rock masses from plane and wedge failures (Stillborg 1994). Describing a limit equilibrium analysis for toppling failure, Goodman and Bray (1976) have presented a method of preventing toppling failure of rock blocks using cablebolts. However, these support and reinforcement works have a limitation that they can only be executed if slopes are accessible to machinery works. Rockbolt installation work in a steep, high slope is costly and also risky for the lives of workers. Besides, rockbolt reinforcement may not be effective in a progressive slope failure where weathering plays an important role in reducing rock block shear strength. Therefore, designing of preventive measures to control instabilities in a high, steep and weathering prone slope is still a subject of discussion in rock engineering .

In this study, the geological setting of a high, steep and weathering-prone slope located on the banks of a tributary of the Chikugo River in Kyushu Island, Japan, has been described. The rock mass of the site is then characterized and the characterization features have been used for DEM(distinct element method) model construction. The results of slope stability analysis carried out by DEM models was then used to suggest countermeasures to control the instabilities of this site.

GEOTECHNICAL DESCRIPTION

The rock slope (Fig. 1) is vertical, 400 m long and ranges in height from 30 to 60 m. On May 14, 1997 the area experienced rainfall of 33 mm/hr, which triggered a failure of the rock slope at this site resulting in 16, 150 m^3 of rock debris. The slide was approximately 100 m wide and 3 m deep. During sliding, some rock blocks bounced off to the left bank of the river and destroyed a window of a building. Besides, it also narrowed the river channel. In addition to this latest failure, the slope has failed twice in the past: first in 1920 and again in 1955 as revealed by available records.

Welded tuff derived from Yabakei pyroclastic flow forms a plateau at various parts of Oita Prefecture in Japan. The rock mass at the study site, which also constitutes a part of the plateau, is composed of welded tuff underlain by gravelly palaeo-channel deposit. The welded tuff has as many as four flow units (Fig. 2). The contacts between different flow units are marked by the presence of many closely spaced horizontal joints frequently containing soft infill material. Besides these horizontal joints, the welded tuff is also characterized by prominent vertical shrinkage joints developed during cooling of the lava. The gravelly palaeo-

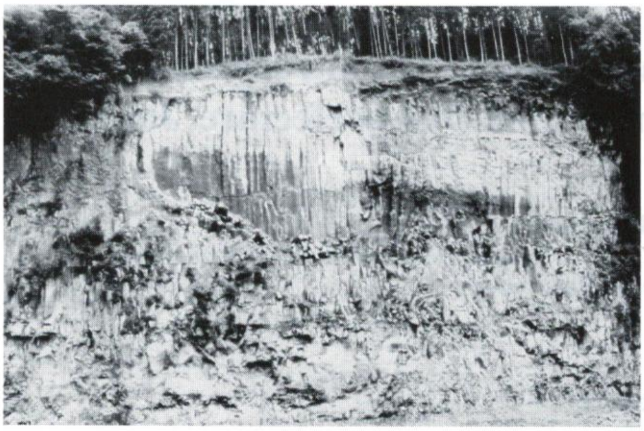

Fig. 1: Photograph showing the rock mass (welded tuff) of the slope.

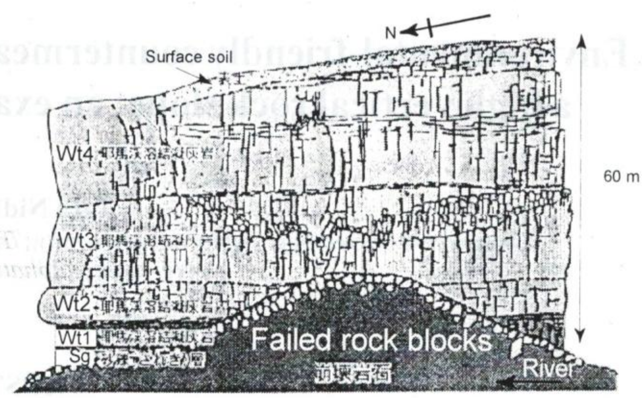

Fig. 2: Sketch (longitudinal section) of the slope showing joints and flow units (Wt1, Wt2 etc.) in welded tuff, which is underlain by gravelly palaeo-channel deposit (Sg). At the toe of the slope are failed rock blocks (after Esaki et al. 2000).

channel deposit is about 4 m thick and is covered by failed rock blocks including gravels and boulders deposited by floods. As shown in Fig. 2, the river is flowing on the gravelly palaeo-channel deposit.

Although all of the four flow units were involved in the latest failure, main failure zone is confined in the 3rd and 4th flow units (Fig 2). There are numerous cracks seen on the cliff face in flow unit 3 and 4. But flow unit two has only a few cracks, whereas the flow unit one has no such cracks. At the ground near the top of the slope, there are some open vertical cracks of 0.1m wide and 4 m deep. These cracks are not inter-connected with the cracks lying further below on the slope face. Besides these cracks, the rock mass at the slope face consists of loose rock blocks of pillar like shape. The field investigation also revealed a possibility that the underlying gravelly palaeo-channel deposit may be eroded away by the floodwater creating a situation in which the overlying rock mass may slump down. These situations coupled with the vertical angle of slope make the site prone to failure. the fauilure may cause a disaster by blocking the river, particularly, if it leads to large volume of failed materials.

This investigation aims at identifying the possible mode of failure and the likely volume of the failed materials. At the same time, it also attempts to select a suitable countermeasure to prevent the likely disaster. Considering the highly jointed nature of the rock mass, distinct element method (Cundall and Hart 1993) has been employed as an analysis tool.

ROCK MASS MODELLING

The persistence, spacing and orientation of the joints were measured on the exposed slope face. The collected data were also refined considering the dimension of failed blocks accumulated at the toe of the slope. In addition, the gap lengths for horizontal joints were also established by the DEM analysis in order to assess block dimension in the model. The input data prepared in this manner are shown in Table 1 and the same were used for a two-dimensional DEM model construction (Fig. 3). Although mechanical properties of rock and discontinuity have to be established from laboratory tests (Bhattarai et al. 2001) for a DEM model constructiion, in the present study only published data (Table 2 and 3) were used. as the published data pertains to the samples of the same rock type taken from the nearby area of the present study site. In addition, the DEM analysis carried out using the properties shown in Table 2 and 3 has also finally predicted practically acceptable results as described below.

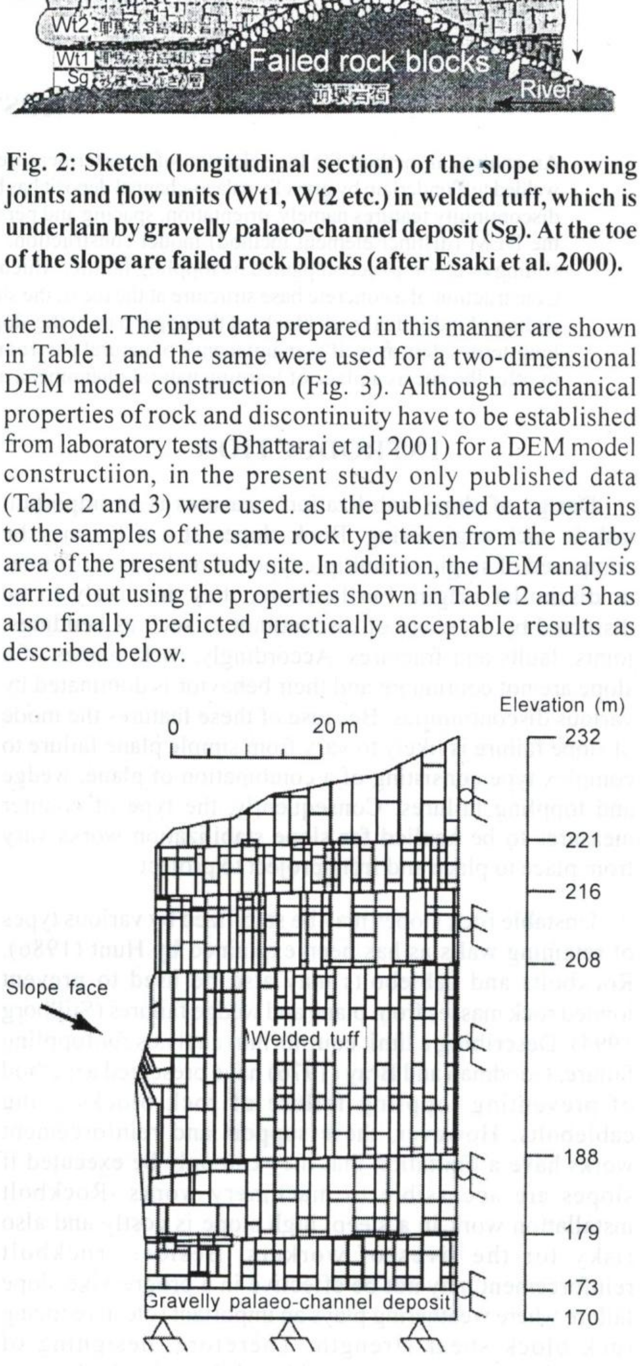

Fig.3: DEM analytical model of the slope also showing the boundary conditions employed during model run.

Table 1: Values of joint parameters established for DEM model construction.

Joint type \ Parameters	Trace length (m)		Gap length (m)		Spacing (m)		Elevation range (m)
	Mean	Standard deviation	Mean	Standard deviation	Mean	Standard deviation	
Vertical joint	6.0	-	-	-	1.66	0.93	173-179
	9.0	-	-	-	1.8	0.99	179-188
	20.0	-	-	-	1.23	0.47	188-208
	9.0	-	-	-	0.98	0.39	208-216
	5.0	-	-	-	1.16	0.41	216-221
	12.0	-	-	-	2.31	0.44	221-232
Horizontal joint	3.99	3.03	0.5	0.1	2.27	2.72	173-179
	5.58	2.87	0.8	0.1	1.78	1.19	179-188
	2.72	1.33	0.7	0.1	1.89	1.04	188-208
	2.68	1.21	0.5	0.2	1.74	1.18	208-216
	2.51	1.28	0.6	0.1	1.49	0.45	216-221
	3.59	1.04	0.5	0.2	3.0	1.30	221-232

Table 2: Properties of joints in welded tuff established by direct shear tests (after Esaki et al., 1998).

Properties	Horizontal joints	Vertical joints
Shear stiffness (Mpa/m)	2880	449
Normal stiffness (Mpa/m)	10800	8670
Cohesion (Mpa)	0.039	0.011
Friction angle (degree)	54	50

SLOPE STABILITY ANALYSIS

As already mentioned above, the rock mass of the site consists of welded tuff underlain by gravelly palaeo-channel deposit. Accordingly, the lowest band in the model (Fig. 3) is composed of gravelly palaeo-channel deposit and overlying it is the jointed mass of welded tuff. During DEM analysis, the right boundary of the model was assumed to have roller boundary condition, whereas the base of the model was assumed to have fixed boundary condition.

Analysis was performed to assess the response of the slope to a situation when an exposed portion of the underlying gravelly palaeo-channel deposit is eroded away. This situation is intended to simulate a flooding case when floodwater may wash out some portion of the poorly cemented gravelly palaeo-channel deposit. The model analysis shows that the rock mass lying above the gravel deposit slumps down, but the failure extends only up to about the mid-height of the slope (Fig. 4a). Afterwards, the failed rock blocks were removed and the model was run again using reduced strength properties. Fig. 4b shows the response of the model when the strength properties were reduced to 60% of the original value. This time, sliding of two blocks occurs in the upper part of the slope but the failure still appears to be confined locally. In the next case, to simulate an earthquake situation, a horizontal body force (based on static seismic coefficient = 0.1) was applied and the model was run using the original shear strength properties. In this case, the model predicts instabilities (Fig. 4c) of the whole slope, but the depth of failure does not exceed 3 m.

Table 3: Intact rock properties established by the laboratory tests (after Esaki et al. 1998)

Properties	Welded tuff	Gravel deposit
Density (N/m^3)	22300	18000
Bulk modulus (Mpa)	16800	155
Shear modulus (Mpa)	6460	51
Cohesion (Mpa)	2.45	-
Friction angle (degree)	45	46
UCS (Mpa)	74.3	-
Tensile strength (Mpa)	1.96	-

UCS: Unconfined Compressive Strength

COUNTERMEASURES FOR INSTABILITY CONTROL

The results of the DEM analysis and field observations both, suggest that sliding (slumping) and toppling are the prominent failure modes of rock slope failure at the studied site. Preventive measures such as rock slope trimming, excavation for reducing the slope height and/or slope inclination may not be effective, because joint spacing is, in general, very small and thus a designed slope geometry will go on changing further inviting toppling of rock blocks. Rockbolt reinforcement may also not be a good choice, because the slope is very high and inaccessible to machinery works. Besides, the rock blocks are small in size, loosely packed and their strength will further reduce as weathering continues. This may lead to further reduction in the size of rock block and, consequently to reduction in shear strength as well. Consequently, rockbolt reinforcement may not be effective, as the smaller rock blocks tend to fall down and thus lead to progressive failure.

Based on the field study and the DEM analysis it may be concluded that the sliding (slumping) can be restricted by constructing a concrete base structure, provided adequate provision for draining the slope behind the structure, is made. The concrete base structure protects the underlying palaeo-gravel deposit from being eroded away due to floodwater and thus prevents major instability, but as already discussed

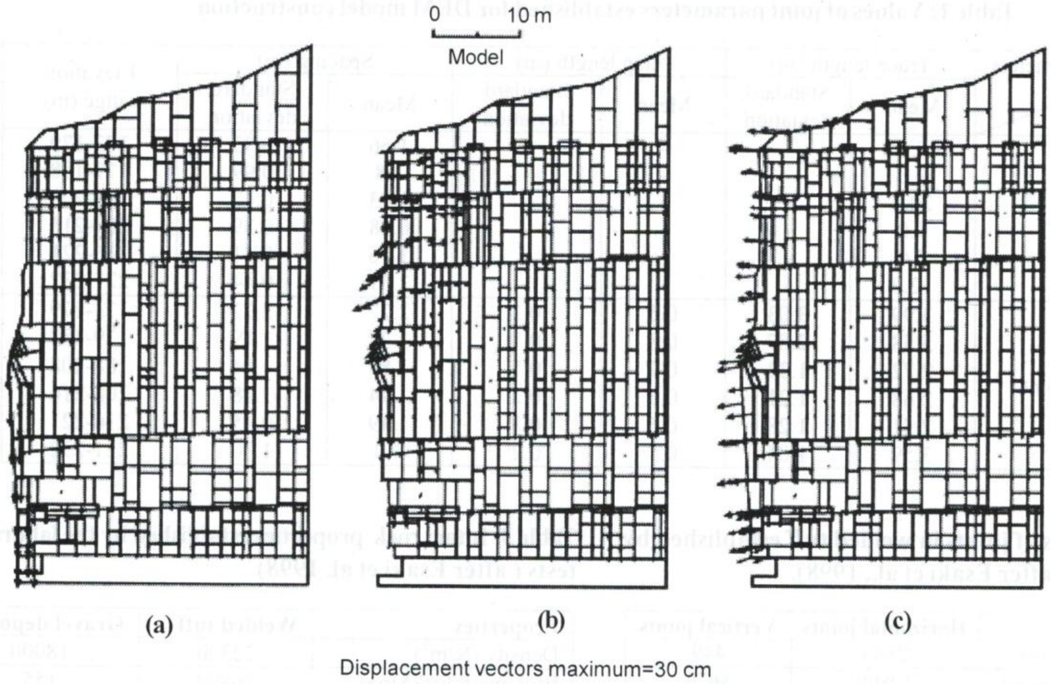

Fig. 4: Displacement vectors developed in DEM models for different situations. (a): Erosion of gravel deposit, (b): Reduction of joint strength, and (c): Development of horizontal body force.

above, toppling and sliding of individual rock blocks may still take place naturally in the upper part of the slope. As a result, the volume of the failed materials will be reduced considerably. A simple rock mass monitoring system may help to understand the critical condition of individual rock block that may fail and thus a swarning may be issued to safeguard public lives and properties including restoration works in the event of disasters.

DISCUSSIONS

As already mentioned above the DEM analysis results and field observations both, suggest that the sliding (slumping) and toppling are the prominent failure modes of rock slope failure at this site. It has also been already discussed that the wash out of the gravelly palaeo-channel deposit due to floodwater may cause slumping of overlying rock mass, whereas toppling of individual rock blocks, at the upper part of the slope, may take place due to reduction in the shear strength of discontinuities or due to an earthquake. The circular failure, which may lead to comparatively larger volume of failed materials, may not take place as indicated by the result of DEM analysis and as also dictated by the dimension of the rock blocks and geometrical network of the discontinuities. In the prevailing conditions, slumping is the only failure process that may result in comparatively larger volume of failed materials, although it will also depend on the extent of erosion of the gravelly palaeo-channel deposit. Thus, attempt of restricting slumping by constructing concrete base structures at the toe of the slope seems to be a viable option to prevent blockage of the river, and consequently to prevent the likely disaster. Similarly, monitoring of critical rock blocks allows one to take possible measure in time to prevent loss of lives and public properties. Further, these proposed countermeasures not only consider long-term safety but also maintain natural environment of the site and do not warrant any extensive construction works. At the same time, it encourages local authority to offer the site as a place of learning natural phenomena and methods of its preservation. Based on these facts the local government is proceeding ahead to implement the recommendations.

CONCLUSIONS

In this paper, an attempt has been made to identify likely failure mechanisms in a high, vertical and weathering prone slope of welded tuff using the DEM models. The model analysis indicates that the sliding (slumping) and / or toppling are the prominent modes of rock slope failure at this site. Construction of a concrete base structure at the toe of the slope is suggested to restrict sliding (slumping). But toppling and sliding of individual rock blocks, at the upper part of the slope, are allowed to take place naturally. The proposed countermeasures not only consider long-term safety but also maintain natural environment of the site without any extensive construction works.

ACKNOWLEDGEMENTS

This research work was carried out by the author while pursuing doctoral course in Kyushu University, Japan under the supervision of Prof. Dr. Tetsuro Esaki. The author received Monbusho Scholarship from Japanese Ministry of Education for his higher studies at Kyushu University. Dr. Y. Mitani, Associate Professor, Kyushu University, Japan provided valuable suggestions during field investigations and laboratory works.

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