Int. J. of GEOMATE, March, 2014, Vol. 6, No. 1 (Sl. No. 11), pp. 800-805
Geotech., Const. Mat. and Env., ISSN:2186-2982(P), 2186-2990(O), Japan

STABILITY OF A ROCK SLOPE SUSCEPTIBLE TO SEASONAL MOVEMENTS

A. K. Alzo'ubi[1]
[1]College of Engineering, Abu Dhabi University, UAE

ABSTRACT: A rock slope, known as the Checkerboard Creek slope located in British Columbia, Canada, is moving under the effect of seasonal temperature changes. Freezing and thawing processes are causing the rock to move at rate of 13 mm/year. A 60 m deep weathered zone has been identified along the slope. This paper uses a discrete element numerical modeling approach to investigate the Checkerboard Creek slope, which is moving toward the reservoir, and to propose a support system to stabilize the slope. The numerical simulation conducted in this study shows that the failure is situated near the slope's toe where a road-cut has been made. To stabilize the rock slope, a support system comprising of cables and a shotcrete layer was proposed and installed numerically along the steepest portion of the weathered rock area. Comparison studies showed that the proposed support system could successfully stabilize the moving rock slope at a reduced tensile strength value. By using this support system, the risk of rock slope failure will be reduced to an acceptable limit through increasing the Factor of Safety of the slope to 1.15. Installing a support system for this problematic slope is highly recommended.

Keywords: Rock Slope, Seasonal, Support, Safety

1. INTRODUCTION

The Checkerboard Creek slope is located at the Revelstoke hydroelectric dam, British Columbia (BC), Canada (Fig. 1). A series of active and old tension cracks were discovered up to 150 m above the highway rock cut immediately after the end of construction of the dam. This finding prompted an intensive geotechnical and geological program carried out by BC Hydro to examine the moving slope, define the moving volume, and monitor the slope to guarantee the safety of the Dam.

Fig. 1 The general location of the Revelstoke Dam

In 1984, BC Hydro began an investigation of the moving slope under question. The thorough investigation was conducted to determine the possibility of any instability in the slope [1]. The slope was moving toward the reservoir at a rate of 10-13 mm/year. This movement is seasonal movement, beginning as the ground surface cools in October and stopping when the ground begins to warm in May. The investigation established that the moving volume is between 2 to 3 million m^3 concentrated in a weathered rock mass.

Monitoring of the rock slope has determined that the slope movement is triggered by the thermal cycle experienced by the slope. This thermal cycle, along with other environmental effects, weathers the rock mass and results in a weathered region along the slope face. Aydin and Basu [2] found that as the tensile strength of the rock material decreased, the material tends to behave with more ductility and the rupture strain increased as a function of the weathering degree.

Martin et al [3] found out that although total collapse of the slope is highly unlikely, the rock mass adjacent to the highway is susceptible to failure under the continuous weathering process. This study presents a supporting system consisted of a shotcrete layer and a cable system to stabilize the slope. The strength reduction factor method was used to assess the Factor of Safety of the supported slope, and was found to be larger than 1.

2. GEOLOGY AND CLIMATE

The geotechnical and geological settings of the site was discussed by [4], [5], and [6]. The rock mass is composed of igneous rocks, mainly foliated Granodiorite overlying the easterly dipping Columbia River Fault. Steeply dipping joints and shears were identified. These joints and shears dip in and out of the slope at 60°-90° from the horizontal. Toppling driven by shearing along these discontinuities is kinematically possible.

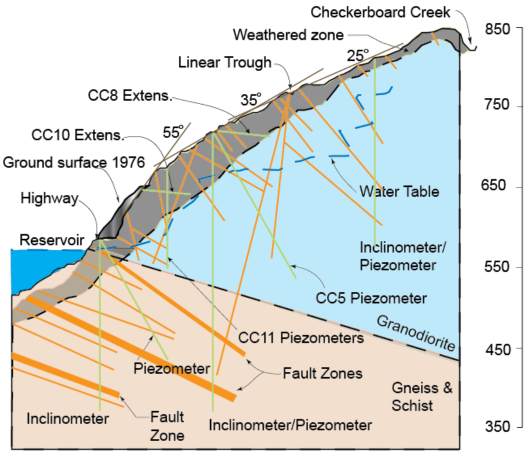

Fig. 2 Cross-section along the Checkerboard Rock Slope (modified from [7])

A weathered layer exists within the top 60 m of the slope. This layer is composed of poor-quality rocks: highly weathered, weak, altered and disturbed rock with crushed zones and frequent shears. The movement of the slope is concentrated in this weathered region. Underneath this weathered layer, a more competent layer was found of fair-to-good-quality rock. Localized zones of poor-quality rock were found along the shear zones and joints. Figure 2 shows a cross-section through the slope. The weathering of the rock slope was extensive within the first 60 m from the ground surface (Fig. 3).

Fig. 2 Surface weathering of the slope adjacent to the highway cut

Between 1984 and 2000, slope instrumentation was installed in the slope area to monitor the displacement, pore water pressure, and the temperature change. For the details of the instrumentation and its distribution in the boreholes see [7]. Readings from the instruments were later processed and showed strong relations between the variation in the temperature and the movement of the slope.

Monitoring revealed that the slope is moving in a cyclic mode at a rate between 0.5 to 13 mm/year. The cyclic nature of the displacement was found to be associated with the seasonal temperature. Martin *et al* [3] shows the displacement pattern along with the temperature and water level variation in the slope. The slope movement resumes during the cold weather between early autumn and late winter.

The average air temperature ranges from 25 °C to 35 °C. Freezing occurs between November and March. Continuous readings were taken from the piezometers installed in the slope; the readings showed that the saturated conditions exist 50 m to 80 m below the ground surface. See Fig. 2 for the interpreted water table location. The water table was incorporated into the numerical model.

3. THE NUMERICAL MODEL

The discrete element method has been long recognized for its ability to model rock mass behavior in underground and near-surface rock applications. This method can simulate explicitly discontinuities inside the rock mass.

UDEC [8] was originally not designed for isolated joints or for joints terminating in intact rock. However, Lorig and Cundall [9] described the implementation of the voronoi joint option in UDEC for modeling reinforced concrete beams. The introduction of the voronoi joint provided a means for simulating joints that were discontinuous and terminated in heterogeneous intact rock. The randomly sized polygons can be considered analogous to flaws in the intact rock.

Fig. 4 Rock mass exposure showing the blocky nature of the rock mass

Figure 4 shows the blocky nature of the rock mass that forms the rock slope. As shown in the figure, the rock slope is highly heterogeneous with randomly shaped blocks. This justifies the use of the mosaic block tessellation to generate polygonal blocks in two dimensions that can simulate the rock slope behavior realistically.

A network of flaws was generated numerically inside the rock mass. These flaws have an average length of 1.3 m, formed random polygonal blocks. All the discontinuities and the geological units were included in the simulation along with the weathered region. Figure 5 shows the UDEC-DM used in this paper and fig. 6 shows the details of the UDEC-DM in the area of concern.

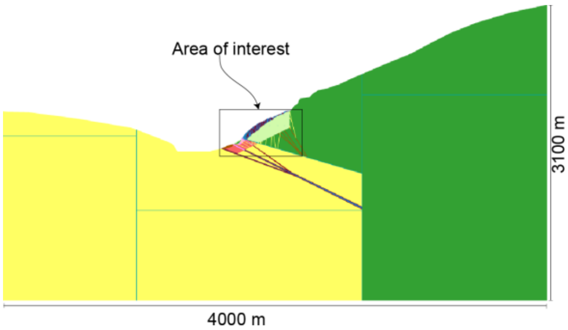

Fig.5 The numerical model of the Checkerboard rock slope

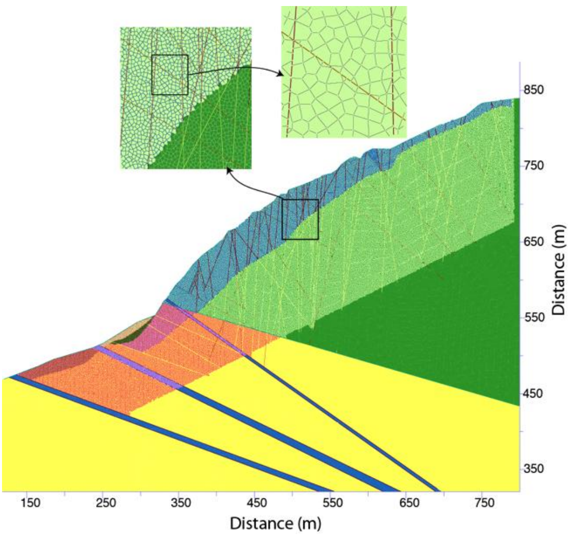

Fig.6 The details of the joints and flaws generated by using UDEC-DM

The Hoek-Brown failure criterion [11] was used to determine the rock mass cohesion, friction and tensile strength. The discontinuities' properties were taken from [7]. Table 1 shows the material and Voronoi microstructure properties used in this numerical study. The normal stiffness of the flaws and the shear stiffness were assumed to be equal and then varied to investigate their effect on the slope. The shear stiffness was varied between 1/10 and 4/10 of the normal stiffness. This range was found to have insignificant effects on the displacements from the numerical model measured at the slope surface.

4. FAILURE PROCESS AND LOCATION

The progressive failure of geo-materials is a gradual and time-dependent failure process at localized areas, followed by the redistribution of the stresses in the slope. These new stresses build up and cause the propagation of the failure. If this fracturing continues, a rupture surface connecting all the failed parts will form. Many factors can cause progressive failure, such as the time-dependent degradation of the strength parameters, increasing pore pressures, and/or thermal stresses. In addition, the presence of non-persistent joints and heterogeneity of the rock mass might cause tensile stresses inside the rock mass and also the initiation and propagation of cracks that might lead to failure.

To study the effect of weathering on the Checkerboard Creek slope, the cohesion and friction of each geological unit was estimated using the Hoek-Brown criterion [11] and the GSI index for each rock formation. The GSI index used in this study was adapted from [7] (see Table 1). Martin *et al* [3] showed that failure will be concentrated near the road cut and tensile strength degradation might cause this failure. Although failure occurred, it was limited to the steepest part of the slope (Fig. 7). As a result of the analysis the region that is prone to movement might be stabilized.

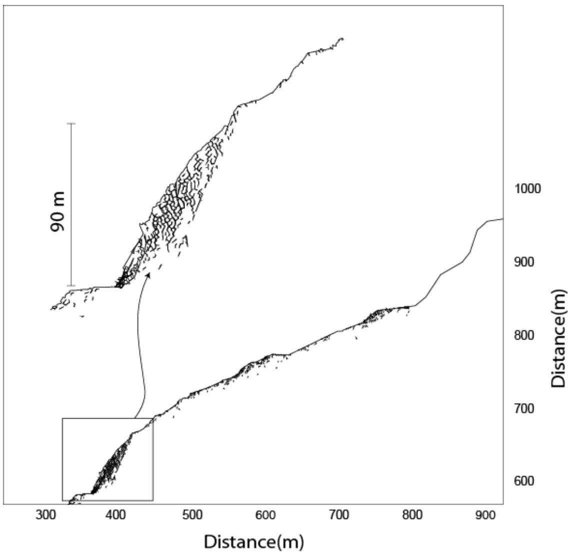

Fig.7 Failure at the rock slope, notice the concentration of the moving rock mass

It is noteworthy to mention that the tensile strength predicted by [11] ranged between 9 to 20 kPa, while the back-calculated tensile strengths in this analysis range was 25 to 650 kPa depending on the assumed unconfined compressive strength. According to this present study, Hoek-Brown criterion underestimates the tensile strength of rock mass.

Table 1: Properties of the rock mass and the voronoi microstructure used in the UDEC-DM analyses

	Gneiss & Mica Schist		Granodiorite		Columbia River Fault & Shears
	Fresh	Weathered	Fresh	Weathered	
UCS (MPa)	78	76	133	75	
GSI	45	35	60	35	
mi	26	20	29	26	
E (GPa)	6.5	3.6	17.7	3.6	0.5
ν	0.25	0.27	0.23	0.27	0.3
Voronoi micro-structure					
ϕ (deg)	36	36	48	36	18
Coh. (MPa)	2.4	2.4	5	2.4	0.1
σ_t (MPa)	0.02	0.02	0.22	0.016	0
kn (GPa/m)	18	5	22	6	1.5
ks (GPa/m)	1.8	0.5	2.2	0.6	0.15

Martin *et al* [3] showed the progressive movement of the failed material toward the reservoir as a result of tensile strength degradation. Another attempt was made to simulate the rock slope and to predict the failure mechanism and extent, if any, by assuming the same properties for the entire slope. The flaws in the weathered and un-weathered geological units were assumed to have the same unconfined compressive strength of 60 MPa, and the tensile strength was back calculated. Failure was initiated at a tensile strength of 0.2 MPa, which is the same back-calculated tensile strength when the weathered and un-weathered rocks have different properties. The rock mass tends to topple and this toppling was restricted to the steepest slope portion near the highway cut (Fig. 8).

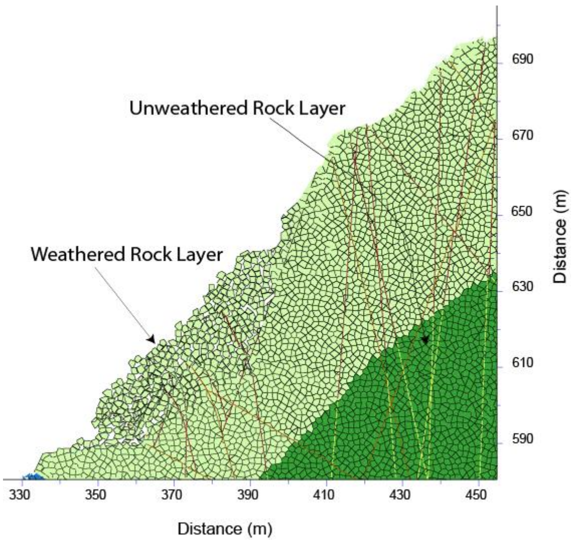

Fig.8 Progressive movement and toppling of the slope toward the reservoir

5. SUPPORTING SYSTEM

The results of the analysis presented in [3] and in this paper showed that the weathering process might result in failure of the slope near the highway cut. This might lead to retrogressive failure up the slope as a result of steepening of the slope as the failed material moves toward the reservoir. To further investigate this possibility, the moved material at the toe of the slope was excavated. As anticipated, the numerical modeling showed that the previously stable part of the slope started to move toward the reservoir. The newly formed surface resulting from the excavation was a 60° slope, which is very, close to the original slope surface prior to movement.

To avoid the consequences of this scenario or any retrogressive failure, a support system composed of a shotcrete layer and a cable system was designed and tested numerically. Passive anchors were used to stabilize the Marble Shear Block slope, which is another slide at the Revelstoke Dam project [12] and the block was successfully stabilized.

In this paper, a cable system was installed inside the weathered rock region and back into the stable rock mass, as shown in Fig. 9. Many configurations of the cables and the shotcrete layer have been modeled numerically to optimize the supporting system. Table 2 shows the properties of the shotcrete layer and the cables.

The best results were achieved when the cables were spaced at an average of 8 m apart, and 10 cm of shotcrete layer was applied at the surface, as shown in Fig. 9. Based on the analysis conducted, the weathered rock mass was assumed to have weak properties: UCS of 30 MPa and tensile strength of 0.8 MPa to put the slope under marginally stable condition. The tensile strength was then decreased gradually to 0.5 MPa with no significant movement observed. The velocity, displacement and unbalanced forces in the numerical model were watched. According to the numerical analysis of unsupported slope, the model would fail at 0.5 MPa of tensile strength. However, no excessive movement was detected in the numerical model.

Table 2 Support System properties

Property	Shotcrete	Cable
Area (mm^2)		625
Thickness (mm)	100	
E (GPa)	20	98
σ_t at yield	3 MPa	0.55 MN
UCS (MPa)	30	10^3
Coh (MPa)	2.0	
Tensile bond (MPa)	1.0	
Grout k_s (GN/m/m)		6
Grout τ (MN/m)		0.32

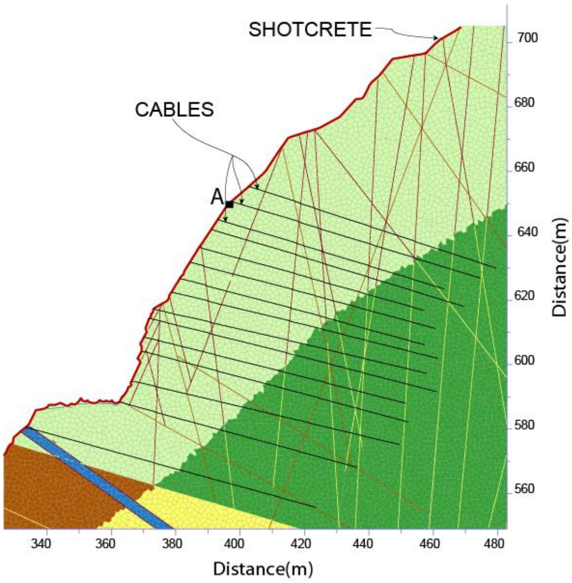

Fig.9 Suggested support system contains of 10 cm shotcrete layer and cables at an average distance of 8 m

Using the same strength properties for the intact material and the Voronoi micro-structure, two models were examined. The first model was with no support, while the other model was with a support system. The results were compared. As observed, the suggested support system was able to limit the displacement and stabilize the rock slope. Figure 10 shows the effect of a support system on the displacement pattern of the slope.

The Figure shows a comparison between the two models. As shown in the Figure, the support system was able to limit the displacements in the model, while the unsupported model continued to deform. To further investigate the model, the tensile strength of the supported rock mass prone to movement was reduced to 0.2 and 0.02 MPa, respectively. The results were similar to those obtained from the previous models. Supporting the rock slope stabilized the movement, no excessive fracturing or failure propagation was observed, and such fracturing as was observed was very minimal.

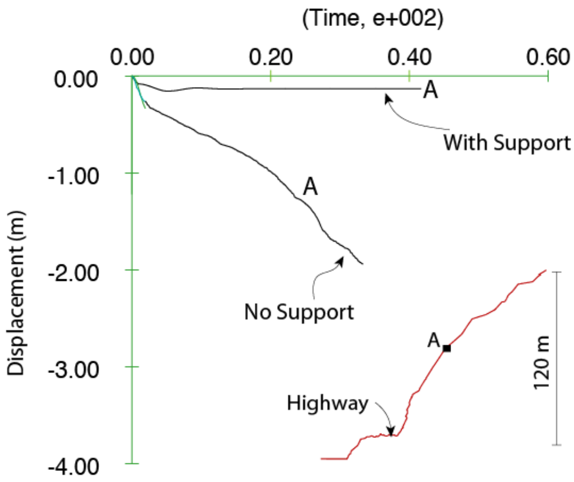

Fig.10 Effect of supporting system on the displacement of surface point A

The cable-shotcrete system proposed and modeled in this paper would hinder the cyclic movement observed at the Checkerboard rock slope. The question remains if this system would perform on site in the same way as in the numerical model. The only way to know is to install the system and monitor the actual performance in the field.

6. FACTOR OF SAFETY

The strength reduction factor was used to evaluate the traditional factor of safety. The proposed support system showed significant improvement on the displacement pattern as shown on the previous section. The strength reduction factor (SRF) method was used to estimate the factor of safety (FS) of the slope prior to the support installment. The FS was less than one, which indicate that the slope is failing. Sjoberg [13] noticed that in rock slopes a factor of safety greater than one is required.

The SRF technique was again utilized to estimate the FS of the supported slope. The FS of the supported slope was 1.15 at the same support system arrangement provided in the previous section. As the spacing between the cables was reduced to an average of 6 m, the factor of safety increased to 1.22. The reduction of the distance between successive cables improved the stability of the slope as concluded from the numerical simulation. Although this Factor of Safety is relatively low, it is acceptable as long as the slope is being monitored [13].

7. CONCLUSION

The discrete element damage approach was applied to simulate the Checkerboard Creek rock slope movement and a supporting system. The weathering process at the slope site caused the rock mass properties to degrade. The rock slope undergoes a seasonal movement that might jeopardize the dam's safety. According to the analysis, the failure is concentrated at the slope toe near the road cut at the steepest part of the slope.

To avoid the possibility of retrogressive failure, a support system consisting of 10 cm shotcrete and cables spaced at 8 m, was used successfully to stabilize the rock slope movement toward the reservoir at a reduced tensile strength value.

This hybrid numerical modeling approach (UDE-DM) adapted in this study was used successfully to model this complex rock slope supporting system and showed high potential of success. The strength reduction factor techniques showed that the factor of safety of the supported slope is 1.2. This Factor of Safety is acceptable in such rock slopes, as long as the rock slope remains under close monitoring to avoid sudden failure of the rock slope.

8. ACKNOWLEDGEMENTS

The author would like to acknowledge the financial contribution of the Railway Ground Hazard Research Program sponsored by CP Rail, CN and transport Canada, and the Natural Sciences and Engineering Research Council of Canada. The author would like also to extend his thanks to Dr. C. D. Martin for his contribution during the initial stages of this research.

9. REFERENCES

[1] Watson A, Moore D, Stewart T "Temperature influence on rock slope movements at Checkerboard Creek", in Proc. International Symposium on Landslides, Evaluation and Stabilization, Lacerda, Ehrlich, Fotoura, Sayao (eds) Rotterdam, Balkema, Rio de Janeiro, Vol 2, 2004, pp. 1293-1304.

[2] Aydin A, Basu A "The use of Brazilian test as a quantitative measure of rock weathering", Rock Mechanics and Rock Engineering, Vol. 39(1), 2006, pp. 77-85.

[3] Martin C.D, Alzo'ubi A.K., Cruden D.M. "Progressive failure mechanisms in a slope prone to toppling", in Proc. International Symposium on Rock Slope Stability in Open Pit Mining and Civil Engineering, Vancouver, Canada, 1st ed., Vol. 1, 2011, pp. 12.

[4] Lane L., "Brittle deformation in the columbia river fault zone near Revelstoke, southeastern British Columbia, Canada", Canadaian Journal of Earth Sciences, Vol. 21, 1984, pp.584-598

[5] Moore D, Imrie A, Enegren E "Evaluation and management of Revelstoke dam reservoir slopes", in Proc. The 19th ICOLD Congress, Florence, Italy, Vol 3, 1997, pp. 1-22.

[6] Moore DP, Lewis MR "Rock slope reinforcement with passive anchors", in Proc. 23rd United States Symposium on Rock Mechanics, University of California, Berkeley, CA, 1982, pp. 945-952.

[7] Watson A, Martin C, Moore D, Thomas W, Lorig J., "Integration of geology, monitoring and modelling to assess rockslide risk", Rock and Soil Engineering, Felsbau Vol. 3, 2006, pp. 50-58.

[8] Itasca "Udec-Universal Distinct Element Code", Version 4.0. Itasca Consulting Group,Inc., Minneapolis. 2004.

[9] Lorig LJ, Cundall PA "Modeling of reinforced concrete using the Distinct Element Method", in Proc. Fracture of Concrete and Rock, SEM-RILEM international Conference, Shah SB, Swartz SE (eds), Springer-Verlag, New York Inc., Houston, Texas, USA, 1987, pp. 276-287.

[10] Alzo'ubi, A.K., C.D. Martin, and D.M. Cruden. "Influence of tensile strength on toppling failure in centrifuge tests", International Journal of Rock Mechanics and Mining Sciences Vol. 47(1), 2010, pp. 974-982.

[11] Hoek E, Carranza-Torres C, Corkum B "Hoek-Brown failure criterion-2002 edition", in Proc. The 5th North American Rock Mechanics Symp, Toronto, Canada, Vol. 1, 2002, pp. 267-273.

[12] Martin CD, Kaiser PK "Analysis of rock slopes with internal dilation", Canadian Geotechnical Journal, Vol. 21(4), 1984, pp. 605-624

[13] Sjoberg, J., "Analysis of large-scale rock slopes". Ph.D. thesis, Lulea University of Technology, Lulea, Sweden, 1999.

Int. J. of GEOMATE, March, 2014, Vol. 6, No. 1 (Sl. No. 11), pp. 800-805.
MS No. 3279 received on July 17, 2013 and reviewed under GEOMATE publication policies.

Pertinent discussion including authors' closure, if any, will be published in the March. 2015 if the discussion is received by Sept, 2014.
Corresponding Author: A. K. Alzo'ubi