Progressive Failure Mechanisms in a Slope Prone to Toppling

C.D. Martin  Dept. Civil & Environmental Engineering, University of Alberta, Edmonton, Canada
A.K. Alzo'ubi  Dept. Civil Engineering, Abu Dhabi University, Abu Dhabi, UAE
D. Cruden  Dept. Civil & Environmental Engineering, University of Alberta, Edmonton, Canada

Abstract
The Checkerboard Creek rock slope is located upstream of the Revelstoke hydroelectric dam, British Columbia, Canada. Annual surface movements of approximately 10 mm have been recorded over the past 25 years. The movement is occurring within a 60-m-deep weathered zone. A numerical modelling methodology based on a discrete element framework was used to investigate progressive failure of the slope that lacks a basal rupture surface but does contain discontinuous joints and shears. The methodology, which assigns internal micro-structure to the intact rock allows for both shear and tensile failure through intact rock bridges and for slip along existing discontinuities. The progressive failure modelling methodology was used to estimate the effect of continued weathering on the stability of the slope. A velocity boundary condition was also applied to the slope surface to estimate the time to failure. The simulations predicted that slope instability would localize at the steepest part of the slope but could take nearly 200 years to occur, assuming the current deformation rate of 10 mm/year, continues.

1 Introduction
The Checkerboard Creek rock slope is located upstream of the Revelstoke hydroelectric dam, British Columbia, Canada (Figure 1). Upon completion of the construction of the Revelstoke Dam in 1984, a series of ancient and active tension cracks were discovered up to 150 m above a highway rock cut, that had been constructed as part of the highway realignment made necessary by reservoir impounding. This discovery triggered an intensive geotechnical and geological program to investigate the moving slope, determine the moving volume, and monitor the slope to ensure the safety of the Revelstoke Dam (Stewart and Moore, 2002; Watson et al, 2004).

The monitoring of the slope from 1984 until now has determined that the down slope movement averages approximately 10 mm/year, and is primarily caused by the thermal cycle experienced by the rock slope. The movement begins as the ground surface cools in October and stops when the ground begins to warm in April-May. The investigations determined that the moving volume was between 2 to 3 million m$^3$ and concentrated in a 60 m thick weathered rock mass near the ground surface (Watson et al, 2004). Lorig et al (2009) examined the potential for catastrophic collapse of the Checkerboard Creek rock slope and concluded that even under an extreme event scenario, a rockslide into the reservoir would not impact Revelstoke Dam.

The geological investigations for the slide found joints and shears dipping into and out-of slope in the weathered rock mass between 60$^\circ$ and 90$^\circ$. There was no evidence of a downslope-dipping base of sliding. The fracture geometry of the rock slope suggested the potential for toppling. The development of a rupture surface in toppling slopes requires the progressive formation of new fractures as the sub-vertical layers deform. Hence the modeling of toppling slopes, involving discontinuous joints, should provide for both shear and tensile failure through intact rock bridges and for slip along existing joints. In this paper, we utilize a discrete element method that incorporates slip along naturally occurring joint sets as well as rupture of the intact blocks that separate these natural discontinuities. Rock mass yield can occur in both shear and tension and form new blocks that are free to move and rotate and completely detach. Assuming that the recorded deformation pattern of 10 mm/year continues, this paper presents an assessment of the time duration for the rock slope to evolve towards a potentially less stable condition, and where the instability could be expected to occur, which could guide future monitoring efforts. The numerical model is also used to estimate the impact of rock mass strength degradation on the stability of the Checkerboard Creek rock slope.
2 Geotechnical setting

The geotechnical and geological aspects of the Revelstoke Dam site and Checkerboard Creek rock slope have been presented and discussed by Moore et al (1982), Lane (1984), and Moore et al (1997). The rock mass is composed of igneous rocks, mainly foliated granodiorite overlying a gneiss and schist sequence containing the easterly dipping Columbia River Fault. Steeply dipping joints and shears were identified and traced. These joints and shears dip in and out of the slope at 60°-90° from the horizontal.

The site investigations carried out for the Checkerboard Creek slope revealed a variation in rock mass quality (Watson et al, 2004). A weathered layer is present to a 60m depth below the ground surface. This layer composes poor-quality rock: highly weathered, weak, altered and disturbed rock with crushed zones and frequent shears. The movement of the slope is concentrated in this weathered region. Beneath this weathered layer lies fair-to-good-quality rock. Localized zones of poor-quality rock were found along the shear zones and joints. Many of the tension cracks are filled with colluvium to a depth of about 20m. Figure 2 shows a plan and cross-section through the slope, and the location of the cross-section. The cross-section shows the weathered region, the complex geological structures in the area subjected to movement, the instrumentation, and the water table.
The average annual precipitation at Checkerboard Creek ranges from 1.5m to 2.0m. The highest total monthly precipitation occurs in December. The average air temperature ranges from \(-25^\circ\) to \(35^\circ\)C with freezing occurring between November to March. In 1993, packer tests were conducted to establish the ground water condition (Moore et al 1997). In addition, continuous readings were taken from the piezometers installed in the slope. The piezometric data indicated saturated conditions approximately 50m to 80m below the ground surface (see Figure 2 for the interpreted water table location). A downward pressure gradient exists, and the main recharge source is from the infiltration of the surface water.

The monitoring revealed that the slope is moving in a cyclic mode at a displacement rate between 0.5 to 13 mm/year, depending on the location of monitoring points on the slope. The cyclic nature of the displacement was found to be associated with the seasonal temperature. Figure 3 shows the location of extensometer CC10 and Figure 4 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 to late winter. During the rest of the year, limited to no displacement is recorded. Numerical analyses confirmed that the down-slope movement pattern defined by the monitoring system could be attributed to the seasonal temperature variation and the associated thermally induced strains acting on the steeply dipping joints (Watson et al 2006, see Figure 3).

Figure 3. The rock joints exposed on the steep portion of the Checkerboard Creek rock slope above the highway. Extensometer CC10 is housed in the box and its location is shown on Figure 2. Also shown in the right figure is the mechanism suggested by Watson et al (2006) contributing to the movement mechanism.

Watson et al (2006) conducted extensive numerical and physical modelling to determine the likely size and impact of a failure of the Checkerboard Creek rock slope under various loading scenarios. They utilized the geological model in Figure 2 and the Universal Distinct Element Code (UDEC). According to Watson et al (2006) the numerical analyses indicated that there is a large reserve of rock mass strength against collapse of the entire slope under both static and seismic conditions and confirmed that there is negligible likelihood of a sudden single collapse larger than about 0.5 million m\(^3\). The geometry and the joint distribution along with the joint's properties from the 2006 analysis were used in this current study to explore the long-term effect of the continuing annual movements on the stability of Checkerboard Creek rock slope.
3 Simulation of progressive failure

Analysis of a rock slope typically requires an evaluation of the rock structure, an assessment of the mode of movement and the strength of the material. Once a critical mode is determined, both continuum and/or discontinuum analyses can be readily applied and the analyses are relatively straightforward. However, in the Checkerboard Creek rock slope, no obvious surface of rupture is present and in such situations continuum and, in many cases discontinuum analyses, are not appropriate. Eberhardt et al. (2004) showed that a continuum/discontinuum hybrid approach based on fracture mechanics could be used to simulate slope failure in massive brittle rocks. However, the suggested yield mechanism illustrated in Figure 3 relies on discrete discontinuities and for those situations a discontinuum approach is desirable as it can readily accommodate discrete fractures as well as provide for the growth of new fractures.

The discrete element method has long been recognized for its ability to model rock mass behaviour in underground and near-surface rock applications. This method can simulate explicitly discontinuities inside the rock mass. The Universal Distinct Element Code (UDEC) was originally not designed for isolated joints or for joints terminating in intact rock. However, Lorig and Cundall (1987) described the implementation of the Voronoi joint option in UDEC for modelling reinforced concrete beams. The introduction of the Voronoi joint provided a means for simulating joints that were discontinuous and terminated in intact rock. The Voronoi tessellation generator command in UDEC can be used to create randomly sized polygonal blocks between continuous joints. The randomly-sized polygons can be considered analogous to microstructure in the intact rock that can represent grain boundaries or larger scale internal structure. Failure is
simulated by progressive breakage of these micro-structure. By developing a rock mass with both discontinuous joints and intact rock between these joints filled with polygonal shaped micro-structure a rupture surface can develop without constraints, i.e., the rock can yield by shearing along the existing joints, and/or rupture of intact rock by either tension or shear. This jointed and polygonal block model will be referred to as the UDEC damage model (UDEC-DM) in this paper.

The development of a rupture surface in rock slopes that contain discontinuous joints requires the progressive formation of new fractures as the existing fractures deform. Modelling of this progressive process in the vicinity of the Checkerboard Creek highway excavation was simulated by Lorig et al (2009) using discrete elements with an artificial regular through-going fracture system that formed a conjugate joint set (Figure 5). To estimate the volume of displaced material from the steepened portion of the slope, Lorig et al (2009) degraded the material properties of the joints until the deformations were sufficiently large that individual blocks were able to translate and rotate and collapse as a pile of blocks. In their approach the portion of the slope that collapsed was constrained by the major shear zones and was simply used to explore the volume of rock mass that could enter the reservoir. Figure 5 shows the area of the slope modeled by Lorig et al (2009) and the slope volume as it collapsed. This approach led Watson et al (2006) to conclude that the Checkerboard Creek rock mass had significant strength capacity.

In this paper, we utilize a similar discrete element approach to that described by Lorig et al. (2009) but in our approach the artificial conjugate joints are neglected. Only the joints and shears shown in Figure 2 are utilized to develop the numerical model. Consequently failure is constrained to naturally occurring joint sets as well as failure of the intact blocks that separate these natural joints.

The properties for the main geological rock mass units (Hornblende Gneiss and Mica Schist, and Granodiorite) and the discrete discontinuities and shear zones were taken from Watson et al (2006). Table 1 is a summary of the properties in the UDEC-DM model for simulating the Voronoi polygonal microstructure within the intact blocks. The values for the input parameters that are needed for the Voronoi microstructure cannot be measured and have to be determined through calibration to the laboratory values for the small scale samples, e.g., uniaxial compressive strength (UCS), Brazilian tensile strength and Young’s modulus (Lan et al 2010).
Table 1. Properties of the rock mass and the internal Voronoi micro-structure used in the UDEC-DM numerical analyses.

<table>
<thead>
<tr>
<th></th>
<th>Gneiss &amp; Mica Schist</th>
<th>Granodiorite</th>
<th>Columbia River Fault &amp; Shears</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fresh</td>
<td>Weathered</td>
<td>Fresh</td>
</tr>
<tr>
<td>UCS (MPa)</td>
<td>78</td>
<td>76</td>
<td>133</td>
</tr>
<tr>
<td>GSI</td>
<td>45</td>
<td>35</td>
<td>60</td>
</tr>
<tr>
<td>mi</td>
<td>26</td>
<td>20</td>
<td>29</td>
</tr>
<tr>
<td>E (GPa)</td>
<td>6.5</td>
<td>3.6</td>
<td>17.7</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.25</td>
<td>0.27</td>
<td>0.23</td>
</tr>
<tr>
<td>Voronoi micro-structure</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Friction (deg)</td>
<td>36</td>
<td>36</td>
<td>48</td>
</tr>
<tr>
<td>Coh. (MPa)</td>
<td>2.4</td>
<td>2.4</td>
<td>5</td>
</tr>
<tr>
<td>Tens. str. (MPa)</td>
<td>0.02</td>
<td>0.02</td>
<td>0.22</td>
</tr>
<tr>
<td>kn (GPa/m)</td>
<td>18</td>
<td>5</td>
<td>22</td>
</tr>
<tr>
<td>ks (GPa/m)</td>
<td>1.8</td>
<td>0.5</td>
<td>2.2</td>
</tr>
</tbody>
</table>

The polygonal block pattern used to simulate micro-fractures provides a method for simulating tensile and/or shear fracturing through intact rock. Cohesion, friction, and tensile strength can be assigned to the boundaries of these polygonal blocks such that the strength is the same as intact rock. Local variation in strength and stiffness can also be applied if required. The surface weathering of the Checkerboard Creek slope suggests representing the intact rock as polygonal blocks (see Figure 6). However, the size of these blocks is unknown and can only be estimated. Inspection of the weathering observed in the rock outcrops on the Checkerboard Creek rock slope suggests that the largest dimension of the blocks can vary from tens of centimetres to one or two metres (Figure 6). In the UDEC-DM model, the micro-structure had an average length of 1.3 m and formed the random polygonal blocks shown in Figure 7.
4 Effect of weathering and slope movements on stability

There are two characteristics of Checkerboard Creek rock slope that are not observed in the other large rock slides found in the Columbia valley of Western Canada (e.g., Downie Slide, Dutchman’s Ridge, Little Chief
Slide): (1) slope movements are distributed throughout an intensely weathered zone without a through-going basal rupture surface, and (2) annual movements are triggered by thermal changes at the slope surface. In this section the UDEC-DM model in Figure 7 is used to explore the stability of Checkerboard Creek rock slope if the strength of the weathered zone decreases and if the slope movements continue for the foreseeable future at the current rate of 10 mm/year.

4.1 Rock mass weathering

In the Checkerboard Creek rock slope, the weathering of the granodiorite rock mass is extensive and extends to a depth of approximately 50 to 60 m. Within this zone the effect of weathering on the rock strength is illustrated in Figure 8, where the uniaxial compressive strength decreases from approximately 135 MPa at depth to approximately 50 MPa near the ground surface. As shown in Figure 2 this weathering is deepest in the steeper parts of the slope and gradually decreases up the slope. Figure 9 illustrates the appearance of the weathering observed in rock outcrops of granodiorite.

Figure 8. Effect of weathering on intact granodiorite uniaxial compressive strength.

![Figure 8](image)

Figure 9. Example of: a) weathered and b) fresh outcrops of granodiorite observed on Checkerboard Creek rock slope.

![Figure 9_a](image)

a) Weathered granodiorite (location 1, Fig. 2)

![Figure 9_b](image)

b) Relatively fresh granodiorite (location 2, Fig. 2)
It is generally accepted that the progressive failure of a slope with discontinuous joints and shears induces tensile stresses inside the rock mass. Because rock is weakest in tension, the tensile strength can control its shear resistance (Cho et al, 2008, Alzo'ubi et al, 2010). Aydin and Basu (2006) conducted a series of Brazilian tension tests to evaluate the tensile strength of weathered igneous rocks. They found that the weathering accompanying microstructural weakening could decrease tensile strength by an order of magnitude depending on the extent of weathering. They also found that as the tensile strength of the material decreased, the stiffness of the rock decreased and the rupture strain increased as a function of the weathering, i.e., the rock mass became more ductile.

The effect of weathering on the Checkerboard Creek rock slope was simulated by gradually reducing the tensile strength of the rock mass. By assuming that the weathering only affected the tensile strength of the intact rock while the friction and cohesion remained constant or reduced at a very small rate, the tensile strength is destroyed long before the frictional strength is mobilized. In other words, weathering may be a significant factor in controlling the behaviour of rock slopes, if the stability of the slope is controlled by the tensile strength. Alzo'ubi et al (2010) showed that in flexural toppling failures the tensile strength was found to be a key factor in this failure mechanism, and the collapse load was controlled by the tensile strength. Moreover, they found that the friction angle of the intact rock did not have a significant effect on the toppling failure mechanism.

To simulate the weathering process, the tensile strength in the weathered zone was reduced in increments in the UDEC-DM model until unstable deformations developed. Failure occurred when the tensile strength of the intact rock was decreased to 0.2 MPa. Although failure occurred, it was limited to the area adjacent to the highway cut at the toe of the slope (Figure 10). Note that the volume of collapsed rock was significantly less than that found by Lorig et al (2009), and shown in Figure 5. Figure 10 also shows the initiation of fractures that occurred in the complete slope when the tensile strength reached 0.2 MPa. The sensitivity of the solution to this value of tensile strength was evaluated by assigning the weathered rock mass properties to the entire slope, irrespective of the geology. The microstructure in the weathered and unweathered geological units was assumed to have an unconfined compressive strength of 60 MPa, and the tensile strength was back calculated. Failure again initiated when the tensile strength was reduced to 0.2 MPa, which is the same as the back-calculated tensile strength when the weathered and unweathered rocks have different properties. The mechanism of failure involved toppling and was also restricted to the steepest slope portion near the highway.
cut. While this approach illustrates the role of tensile strength in the stability of a slope containing joints and shear zones but lacking a through-going rupture surface, the approach cannot be used to estimate the time taken for the weathering and the reduction in tensile strength to occur.

4.2 Effect of long-term movements on slope stability

The movement pattern observed in the Checkerboard Creek rock slope was discussed in Section 2 and illustrated in Figure 4. The movements recorded by the inclinometers in the slope showed that the largest movements occurred at the ground surface and gradually decreased to zero at the base of the weathered zone. The UDEC-DM model shown in Figure 7 was used to investigate the effect of this continuing movement on the stability of Checkerboard Creek rock slope. To simulate this movement, a constant velocity boundary condition was applied horizontally to the slope surface in the region of maximum movements (Figure 11). A velocity of 10 mm/year was simulated which approximates the upper limit of widespread annual movements recorded at Checkerboard Creek. After subjecting the slope surface to the velocity boundary condition, the UDEC-DM was cycled to allow deformation to occur throughout the slope until equilibrium or collapse, if any, was achieved. This procedure was repeated until a portion of the slope showed obvious signs of instability. This modelling approach allowed the slope to deform freely and resulted in extensive fracturing within the slope (Figure 11).

A total horizontal displacement of 1.8 m was required at the slope surface for the slope to become unstable. The insert in Figure 11 shows the measured displacement profile in the weathered region at inclinometer CC11. The resulting displacement profile was similar to that recorded by the inclinometer, with the largest displacement at the slope surface gradually reducing with depth. Comparison of Figure 11 with Figure 10 shows more intense and widespread fracturing within the weathered zone for the surface displacement model than for the tensile strength degradation model. Although the fracturing and joint opening were more extensive with this surface displacement approach, only a portion of the slope collapsed. Again, the portion of the slope that failed was limited to the steep slope above the highway and in this case the slope failure would be classed as a multiple small falls (Figure 12). In contrast to the falls observed in this model, the tensile strength degradation technique resulted in a toppling failure. Despite the application 1.8 m of horizontal displacement to the slope surface, no large collapse of the slope occurred. Assuming the 10mm/year movement rate remains constant, the 1.8 m of displacement in the simulation is equivalent to the displacement that would occur over a 180-year period.
5 Conclusions

There are two characteristics of Checkerboard Creek rock slope that are not found in other large rock slides in the Columbia valley of British Columbia between Revelstoke and Mica (e.g., Downie Slide, Dutchman’s ridge, Little Chief Slide): (1) slope movements are distributed throughout an intensely weathered zone without a through-going basal rupture surface, and (2) annual movements are triggered by thermal changes in the slopes boundary conditions. Predicting the stability of a moving slope that lacks a through-going rupture surface is challenging. In this paper the UDEC-DM was used to explore the stability of Checkerboard Creek rock slope as the strength of the weathered zone decreases and the slope movements continue at the measured rate of 10 mm/year.

The weathering process was simulated by degradation of the intact rock tensile strength, and the ongoing measured movements were simulated by applying a constant velocity boundary condition to the slope surface. Both methods predicted that failure is localized at the steepest part of the slope, at the slope toe near the highway cut. The tensile strength degradation resulted in a toppling failure while the continued surface slope movements resulted in a multiple small falls. The total movement that was required to cause the localized failure was approximately 1.8 m, equivalent to nearly two hundred years of annual movement of 10mm/year. The methodology developed in the UDEC-DM appears capable of simulating complex rock slope behaviour and is useful for exploring the processes associated with progressive failure in rock slopes.

6 Acknowledgements

We 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 authors also acknowledge the support and review provided by Tom Stewart, John Psutka and Andrew Watson of BC Hydro, and Dennis Moore.
7 References


