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Identification of seismically-induced rock-slope failure mechanisms using the discrete element method

L. Arnold & J. Wartman  
*University of Washington, Seattle, Washington, USA*

D. Keefer  
*University of Maine, Orono, Maine, USA*

M. MacLaughlin  
*Montana Tech, Butte, Montana, USA*

**ABSTRACT:** Seismically-induced rock-slope failures represent a significant hazard in many regions worldwide. Much of the current understanding of rock-slope behavior in earthquakes is based on observed historical failure modes. This study presents analysis of dynamic rock-slope failures using the particulate discrete element code PFC2D. The discrete element model is able to simulate intact rock strength, the development of damage in the rock mass, and the transmission and reflection of dynamic waves. Using this model, four primary failure mechanisms and the dynamic conditions that lead to their development are identified. These failure mechanisms are shown to result in three distinct failure modes corresponding to well-documented dynamic rock-slope failure modes observed in natural slopes. The connection between the dynamic failure mechanisms and the resulting modes of failure in the model provides insight into the ground motion characteristics, specifically frequency content and amplitude, that control the behavior of rock-slopes subjected to dynamic loading.

1 INTRODUCTION

Seismically-induced rock-slope failures represent a significant hazard in many regions worldwide. They are among the most common, dangerous, and still today, least understood of all seismic hazards. Keefer (1984) performed a study of 40 historical worldwide earthquakes and the resulting landslides. The types of earthquake-induced landslides involving rock-slopes can be broadly categorized into two groups - deep and shallow rock-slope failures. Keefer notes the seismicity associated with the two groups: “Rock falls [and] rock slides...are initiated by the weakest shaking. In particular, these shallow, highly disrupted landslides from steep slopes are probably susceptible to the short-duration, high-frequency shaking characteristic of small earthquakes. Coherent, generally deep-seated landslides are initiated by stronger and probably longer-duration shaking.” While the observational data presented by Keefer is critically important, it does not provide a detailed understanding of the fundamental mechanisms that drive dynamic rock-slope behavior.

The discrete element method (DEM), and particularly a subset of DEM called the bonded particle method (BPM) has become widely used for modelling complex rock mechanics problems. BPM, first introduced by Potyondy and Cundall in 2004, has been shown to accurately reproduce several complex rock behaviors. This study presents analysis of dynamic rock-slope failures using the particulate discrete element code PFC2D. Although infrequently used for dynamic applications, BPM is capable of simulating the transmission and reflection of dynamic waves and the associated stresses involved.
By building a simple, homogeneous (i.e., contains no pre-existing joints, uniform strength and stiffness) model and subjecting to multiple dynamic inputs, the stresses induced throughout the rock-slope can be observed. Stress measurements under non-destructive testing reveal different critical locations and mechanisms for different loading frequencies. This allows for prediction of the locations and nature of initiation of damage in steep rock-slopes, which are then confirmed with destructive dynamic testing in BPM with both harmonic and recorded ground motions.

The combined (destructive and non-destructive) results show four distinct primary (initiating) failure mechanisms leading to three distinct failure modes. When compared to the results of Keefer (1984), the failure modes and the circumstances under which they are observed in the PFC model show strong agreement with the historical record of seismically-induced rock-slope failures.

2 MODEL DETAILS

The BPM rock-slope used in this study is 80-meters tall with a slope angle of 80 degrees. The full model domain is 280 meters wide and 160 meters tall. Note that in the figures contained in this paper, the model domain is truncated to better show the details affecting slope behavior. The natural frequency of the homogeneous slope is 2.9 Hz, as determined by the empirical formula for slope frequency \( f_n = \frac{V_s}{5H} \) (Ashford and Sitar, 1994) and confirmed by frequency analysis of the BPM model.

The model boundary conditions are defined by viscous stress-controlled boundaries after the method developed by Lysmer and Kuhlemeyers (1969). The stress-controlled boundaries on the model absorb s-waves at the base of the model and s- and p-waves on the sides of the model, where free-field conditions are imposed using a viscous connection to 1-D response columns. The implementation of boundary conditions in BPM is described by Arnold et al. (2014).

Typical engineering properties cannot be applied directly to the bonded-particle model. An extensive calibration process which identifies the micro-properties required to achieve the desired macroproperties is required. BPM inputs include particle size, particle friction, particle and bond stiffnesses in the normal and shear directions, and bond normal and shear strength. During the calibration process, these microproperties are systematically varied until the desired macroproperties are achieved. Table 1 shows the BPM microproperties used and the common engineering properties that emerge from the model as a result of the microproperties assigned. The interparticle and bond shear stiffnesses is equal to 0.57 times the normal stiffnesses, and the bond shear strength is equal to the bond normal strength. The interparticle friction coefficient is 0.6.

3 LOADING DETAILS

The models were subjected to a suite of harmonic motions with constant velocity amplitudes as well as recorded strong ground motions. The harmonic motion suite was applied to the models first in an elastic simulation, where no bond breakage was allowed, and then again in a destructive simulation where bonds were allowed to break as a result of stresses in the slope.

<table>
<thead>
<tr>
<th>BPM Microproperties</th>
<th>Normal Bond Stiffness</th>
<th>Normal Bond Strength</th>
<th>Particle Radius (avg.)</th>
<th>Porosity</th>
<th>Particle Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2E+10 N/m²</td>
<td>2.5E+04 N/m²</td>
<td>0.45 m</td>
<td>0.14</td>
<td>2166 kg/m³</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Engineering Macroproperties</th>
<th>Compressive Strength</th>
<th>Tensile Strength</th>
<th>Shear Wave Velocity</th>
<th>Elastic Modulus</th>
<th>Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>25.0 MPa</td>
<td>3.0 MPa</td>
<td>1200 m/s</td>
<td>6.5 GPa</td>
<td>1900 kg/m³</td>
<td></td>
</tr>
</tbody>
</table>
model. The recorded ground motions were applied at multiple levels of amplitude. Table 2 provides the details of the applied motions.

### Table 2. Loading details.

<table>
<thead>
<tr>
<th>Event</th>
<th>Dominant Frequency (Hz)</th>
<th>Scaled $a_{\text{max}}$ (g)</th>
<th>Scaled $v_{\text{max}}$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Harmonic</td>
<td>0.5 to 5</td>
<td>0.25 to 2.5</td>
<td>0.78</td>
</tr>
<tr>
<td>Northridge</td>
<td>2.2</td>
<td>0.94, 0.62, 0.43, 0.31, 0.25</td>
<td>0.18, 0.78, 0.5, 0.35, 0.25, 0.20, 0.15</td>
</tr>
<tr>
<td>Loma Prieta (G1)</td>
<td>2.9</td>
<td>0.9, 0.58, 0.40, 0.29, 0.23</td>
<td>0.17, 0.78, 0.5, 0.35, 0.25, 0.20, 0.15</td>
</tr>
<tr>
<td>Richmond Hill</td>
<td>4.5</td>
<td>1.82, 0.47, 0.35</td>
<td>0.78, 0.20, 0.15</td>
</tr>
</tbody>
</table>

4 RESULTS

4.1 *Dynamic stresses in steep rock-slopes*

The slope was subjected to harmonic loading in non-destructive simulations in order to measure and observe the stress conditions at different parts of the slope due to different loading frequencies.

The simulations generally indicated that frequencies at and below the slope’s natural frequency, the slope tended to behave as a unit with an upslope and downslope extreme for motion and associated stresses. For frequencies above the natural frequency, more complex wave behavior was observed. The vertically propagating body waves tended to get converted into surface waves traveling upwards on the slope face and to p-waves, which radiate back into the slope. This is consistent with the behavior predicted by Boore et al. (1981) using finite difference analysis. Figures 1 and 2 show examples of different critical stress states due to a low frequency loading.
frequency (1 Hz) and high frequency (5 Hz) motion. The terms “high” and “low” frequency used in this paper indicate the motion frequency relative to the natural frequency of the slope.

Figures 1 and 2 display the stresses within the slope in several ways briefly described here: The approximate stress field is shown in four quadrants of the plots. The two upper quadrants show the first principal stress magnitude on the left and the second principal stress magnitude on the right. The principal stresses are measured as an average for a rectangular grouping (cell) of particles. Each cell contains approximately 100 particles. Compression is indicated by positive stresses (red) and tension is indicated by negative stresses (blue). Both upper quadrants have horizontal lines drawn at H/10 and H/2 (where H is slope height) indicating where cell stress states have been extracted for further plotting on the lower two quadrants. The lower left quadrant shows the stressed along the two measurement lines in principal stress space. The Hoek-Brown failure envelope for the BPM is also plotted to indicate where stresses would be expected to produce failure when damage is allowed in the model. The dashed line in this plot represents the volumetric stress axis, where $\sigma_1 = \sigma_2$. The plot on the lower right shows the stresses along the same measurement lines with stress on the y-axis and distance on the x-axis.

The stress state shown in Figure 1 is produced as a result of 1 Hz loading. First and second principal stresses have opposite signs and have highest amplitudes at the toe of the slope. The principal stress space plot in the lower right quadrant indicates that the model is just at the point of failure at the toe of the slope. At the H/10 level, both first and second principal stresses amplitudes are highest at the slope face. At the H/2 level, the first principal stress remains relatively constant with distance from the slope face and the second principal stress magnitude is at a minimum at the slope face and increases with increasing distance from the slope face. These patterns indicate a compression failure at the toe of the slope and possible tensile failures higher in the slope set back from the slope face.

The stress state shown in Figure 2 is produced as a result of 5 Hz loading. At the point in time shown in Figure 2, the highest stress concentration in the slope is concentrated at the slope face about halfway up the slope. A critical stress state is also present at H/10, but at a distance set back from the slope face. At other points in time, the wave travelling throughout the slope produces stress concentrations along the slope face and crest. These patterns indicate

Figure 2. Stress state showing a moment of tensile stress concentration at the slope face near H/2 due to high frequency (5Hz) loading.
Multiple failure mechanisms and modes were observed in the model. The terms “mechanism” and “mode” have distinct meanings in this paper. A failure mechanism is a failure of intact rock mass due to a stress state at a particular location. A failure mode is a significant movement of rock mass following a failure mechanism.

Four primary failure mechanisms and two secondary failure mechanisms were observed in the model. A “primary” mechanism is one that can initiate without another failure mechanism preceding it. A “secondary” mechanism is one that was only observed following another mechanism. These labels do not indicate the importance of the failure mechanism to the overall behavior of the slope. The mechanisms and modes are illustrated in Figures 3 and 4 and described in the following list:

Primary failure mechanisms:
- Crushing of the rock mass at the toe.
- Wedge failures created by tensile rupture behind the slope face.
- Scattered tensile and shear failure of material at the crest of the slope.
- Deep tensile cracking at the base of the slope set back from the toe.

Secondary failure mechanisms:
- Transverse cracking – where sets of cracks initiate and propagate outward from and perpendicular to another crack or set of cracks.
- Deep circular shear failure. This mechanism was often observed after extensive damage due to multiple other mechanisms.

Failure modes:
- Cliff-collapse: Cliff-collapse involves a relatively shallow failure of intact or partially intact rock along the cliff face, creating a disrupted mass. This falls into the category of a rock
fall as presented by Keefer (1984). The term “disrupted” is used to mean a rock mass that has been broken up into numerous small blocks and rock fragments, after Keefer (2013).

- Slumping of failure wedges: Rock slumps involve the movement of large, coherent failure masses along planar to curved sliding surfaces.
- Toppling of discrete blocks near the crest: Toppling of discrete blocks is preceded by cracking in the model that creates discrete blocks. When these blocks form and topple from the slope, they fall and/or bounce down the slope face. Therefore, toppling can also be categorized as rock fall.

As indicated by the stresses measured in the slope presented in the previous section, the lower frequency motions tended to induce crushing at the toe and wedge failures from tensile rupture. The resulting failure modes typical of lower frequency motions were cliff-collapse and slumping of failure wedges. Also consistent with the stresses in the previous section, the higher frequency motions tended to have a more disrupted failure pattern as high-amplitude dynamic waves scattered across the slope. Scattered damage at the slope crest and face were observed as a result of high frequency motions. The resulting failure modes typical of higher frequency motions were toppling and cliff-collapse.

Figure 4. Failure modes observed in the models. Yellow and green dashed lines indicate tensile and shear damage, respectively. Blue arrows indicate the movement of the rock mass.

Figure 5. Typical damage patterns in models due to lower (a) and higher (b) frequency harmonic input.

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Plots of damage during harmonic loading in Figure 5 illustrate the different mechanisms at lower frequencies (Figure 5a) and higher frequencies (Figure 5b). In Figure 5a, crushing and wedge failures are evident, and the slope is experiencing cliff-collapse. The collapsing face contains relatively intact bodies separated from the rest of the slope. And the damage patterns behind the cliff-collapse zone show that a slumping mode may develop. In Figure 5b, damage is much more scattered throughout the slope, particularly at the crest, where the damage initiated. The scattered damage has produced a more disrupted mass with disruption being highest at the crest and decreasing with depth into the slope. Small bodies can be observed beginning to rotate and topple outwards from the crest.

4.3 Destructive earthquake loading

When subjected to a suite of recorded ground motions, the observed failure mechanisms and modes were fairly uniform regardless of the dominant frequency of the input. In every simulation, if damage to the slope occurred the initial failure mechanism was wedge failure from tensile rupture. After the wedge failure, crushing at the toe occurred. These failure mechanisms combined to produce a shallow cliff-collapse failure mode. With increasing ground motion amplitude, cliff-collapse was followed by slumping of failure wedges. Figure 6 shows typical post-shaking configurations for slopes experiencing cliff-collapse but not slumping (6a) and for slopes experiencing slumping after the initial cliff-collapse (6b). The pseudostatic yield acceleration was determined by incrementally increasing a horizontal gravity component until failure.
The pseudostatic failure involved wedge failure followed by crushing at the toe and cliff-collap-
sing, corresponding closely to the behavior of the slope during recorded ground motions.

Figure 6a is the post-shaking configuration resulting from the Loma Prieta (G1) motion
scaled to $a_{max} = 0.8g$. The slope has experienced cliff-collapse, but no slumping mode develop-
ed. The tension crack in the slope behind the failure surface shows the start of what could become a wedge failure with continued shaking. Figure 6b is the post-shaking configuration
resulting from the Northridge motion scaled to $a_{max} = 0.94g$. The slope has experienced cliff-
collapse and one slumping failure. The slumping mass remains largely intact, while the
volume associated with the initial cliff-collapse is highly disrupted. Damage in the slope
behind the failure mass indicates additional slumping failures would occur with continued shaking or higher amplitude shaking.

When the ratio of the slope’s pseudostatic yield acceleration, $k_y$, to the peak motion amplitude, $k_{max}$ (or $a_{max}$), is compared to induced failure volumes, the influence of failure mode on
the failure volume can be seen. Figure 7 shows the failure volumes induced by recorded ground motions at different $k_y/k_{max}$ ratios. For all three ground motions, a sharp increase in
failure volume occurs as $k_y/k_{max}$ drops below 1. This corresponds to the relatively constant
volume associated with the cliff-collapse failure mode. Failure volumes associated with the slumping mode increase with increased amplitudes.

5 CONCLUSIONS

The failure modes observed in the simulations are consistent with observations from Keefer’s
1984 survey of landslides caused by earthquakes in several respects. Cliff-collapse and top-
pling dominate the response of the slopes in dynamic loading. Slumping occurred less fre-
quently and was associated with higher levels of shaking and took more time to fully develop.

The BPM analysis presented in this paper helps explain some of the mechanisms that lead
to this behavior. Both low and high frequency energy lead to the cliff-collapse failure mode,
although through different failure mechanisms. Higher frequency motions tend to create more
interrupted failure masses because of the more complex wave behavior associated with scattering. In recorded ground motions, with complex frequency content, cliff-collapse was observed
to be the dominant failure mode and preceded slumping failures, when those occurred.
Increased ground motion amplitude tends to create progressively larger failure volumes after
the loading moves from the range that induces cliff-collapse to the range that induces more
deep-seated failures.

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