PRELIMINARY STATIC AND SEISMIC STABILITY OF STEEP SLOPES IN RECLAIMED MINE LANDS CONSTRUCTED WITH LOW COMPACTION IN APPALACHIA, USA.

Isaac JELDES¹, Eric DRUMM²

ABSTRACT

Since the Surface Mining and Control Reclamation Act of 1977 (SMCRA), U.S. coal mining companies have been required to restore the land to the approximate contours that existed prior to mining. To ensure mass stability and limit erosion, the reclaimed materials have traditionally been placed with significant compaction energy. The Forest Reclamation Approach (FRA) is a relatively new approach that has been successfully used to facilitate the fast establishment of healthy forests. To facilitate tree growth, this method specifies the use of low compaction energy in the top 1-1.5 m of the surface material which is in conflict with general considerations for mass stability of the slopes. To investigate the stability of the FRA on steep slopes, a proposed modification of the simple Infinite Slope equation for seismic (pseudo static) excitation was developed and results contrasted against the Simplified Bishop method of slices, for a range of seismic coefficients available in the literature. Based on the agreement of the results, a new method for estimating critical FS in reclaimed slopes based on a semi-empirical probabilistic model function of the 5% dumped spectral acceleration is presented and discussed. This method allows a more realistic representation of the slope performance and hence a more accurate estimation of the FS, since spectral response analysis is included. In addition, this model allows input of parameters that somehow have less selection subjectivity for the designer.

Keywords: slope stability, pseudo-static coefficients, spectral acceleration, infinite slope.

INTRODUCTION

Surface mining is one the central motors of the economic development in the Appalachian region. It is estimated that about 70 percent of U.S. coal is extracted via different surface mining methods. The direct value of surface mining has been estimated at more than U.S. $5 billion (NMA, 2003). Nowadays, coal supplies more than 50% of the electricity consumed by Americans (NMA), yet there can significant environmental impact associated with the activity.

After the Surface Mining and Control Reclamation Act of 1977 (SMCRA), coal companies in the U.S.A. were forced by law to restore the land to its pre-mined condition. Reclamation activities traditionally incorporated compaction procedures to augment the strength of the reclaimed material and ensure stability of the restored slopes. More recently in an attempt to encourage tree growth, reclamation methods employing low compaction effort in the uppermost 1-2 meters have been developed. The Forest Reclamation Approach (Sweigard et al., 2007a) was developed to overcome the detrimental effects that compaction can have on tree survival. While important for strength and erosion resistance, compaction

¹ Graduate Research Assistant, Dept. Civil and Environmental Engineering, University of Tennessee, Knoxville, TN 37996, e-mail: ijeldes@utk.edu
² Professor, Dept. of Biosystems Engineering and Soil Science, University of Tennessee, Knoxville, TN 37996.
diminishes soil porosity which reduces root penetration and soil permeability (Angel et al., 2007; Schor and Gray, 2007; Sweigard et al., 2007a; Sweigard et al., 2007b; Torbert et al., 1994).

To investigate the potential effects on stability and erosion/sediment yield resulting from the implementation of the Forest Reclamation Approach (FRA), three instrumented field sites were constructed to evaluate slope stability and sediment yield in steep slopes constructed with low compaction effort as directed by the FRA. The three sites are referred to here by the name of the initial coal operator (Premium, National and Mountainside), and this paper investigates the seismic slope stability analysis for the sites. Stability is evaluated with traditional limit-equilibrium procedures with pseudo-static forces. A variety of seismic coefficients from the literature were employed and discussed here. The lowest factor of safety was determined using two different approaches: a) a proposed version of the Infinite Slope equation that includes vertical and horizontal seismic forces and b) Simplified Bishop Method of Slices. Also, a proposed method for estimating critical FS in reclaimed slopes based on spectral response is presented in terms of solution charts.

LOCATION OF FIELD SITES

The three sites constructed using the low compaction grading technique are located in the northeastern Tennessee, with Premium located in Anderson County, National in Campbell County and Mountainside located in Claiborne County (Figure 1). All plots in each site were constructed with the help of the local mining company, and while not discussed here, instrumented in order to conduct erosion and sediment analysis (Hoomehr et al., 2010; Jeldes et al., 2010).

Figure 1. Field sites location at Northeastern Tennessee referred as Mountainside, National and Premium sites.
SEISMIC ACTIVITY IN THE APPALACHIA

Local seismic activity in the region has been well documented. It is characterized by a cloud of events in eastern Tennessee (Figure 2). Although the level of seismic activity has been found to be lower than the New Madrid area (west Tennessee), the magnitude of the events have been similar. Figure 2 shows events of magnitude 1 to 5 recorded in the area since 1998 (http://tanasi.gg.utk.edu).

![Figure 2. Local Seismic Activity Since 1998 (University of Tennessee Geophysical Research)](image)

PSEUDO-STATIC COEFFICIENT FOR SEISMIC ANALYSIS

Pseudo-static analysis remains one of the most widely used methods for addressing seismic hazards in civil engineering practice, due to its simplicity. This method is based on d'Alembert’s principle of mechanics: “A system may be set in a state of dynamic equilibrium by adding to the external forces a fictitious force commonly known as inertial force” (Paz, 1997). That force is equivalent to the maximum acceleration of the structure as a fraction of the gravity of acceleration (seismic coefficient $K$) times the mass of the structure.

One of the big problems of this method relies in the fact that in reality seismic forces are cyclic, going back and forward in seconds or probably tenth of the second, producing most likely a series of displacement impulses instead of failure when FS is lower than unity (Bray and Travasarou, 2009; Towhata, 2008). Thus, this method is recognized for being a conservative approach. Nevertheless, a more interesting issue arises from the fact that the maximum acceleration experienced during an earthquake is not equivalent to the seismic coefficient regardless of d'Alembert’s principle (Towhata, 2008). Many authors have proposed expressions for horizontal seismic values as a function of the maximum or peak ground acceleration. Terzaghi (1950) proposed 0.1g for severe earthquakes, 0.25g for violent and destructive earthquakes, and 0.5g for Catastrophic earthquakes. Noda et al. (1975) developed an empirical back-calculated equation based on regression analysis of a series of acceleration records. Makdisi and Seed (1978) proposed values of 0.1g and 0.15g for earthquakes of approximate magnitude 6.5 and 8.25 respectively, accompanied by a strength reduction factor due to cyclic strength degradation. Seed (1979) lists a number of seismic coefficients accepted for design of earth dams in several countries, all ranging from 0.1g to 0.15g. Hynes-Griffin and Franklin (1984) suggested that $Kh = 0.5PGA$ will not generate excessively large deformations in the slope if the factor of safety is higher than 1. Kavazanjian et al. (1997) proposed acceleration reduction equal to 0.17 if response analysis is conducted, otherwise 0.5. Bray and Rathje (1998) proposed a value of 0.75PGA along with a conservative soil strength value. Bray and Travasarou (2009) proposed a more sophisticated procedure; they assumed $Kh$ is not a fraction of the PGA but a function of the 5% damped elastic spectral acceleration ($Sa$) of the site, the desired displacement ($Da$), initial fundamental period ($Is$) and earthquake magnitude ($M$). This last approach is
discussed later in this article. Table 1 presents a summary of the methods for estimating pseudo-static coefficients.

Table 1. Summary of recommended pseudo-static values and expressions, excluding coefficients for earth dams reported by Seed (1979). (Adapted from Duncan, 2005)

<table>
<thead>
<tr>
<th>Method or Reference</th>
<th>Seismic Coefficient</th>
<th>Accl.</th>
<th>Accl. Reduction Factor or Expression</th>
<th>Strength Reduction Factor</th>
<th>Min. FS</th>
<th>Max. Allowable Displ. (m)</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terzaghi (1950)</td>
<td></td>
<td>0.10g</td>
<td>-</td>
<td>-</td>
<td>&gt; 1.00</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25g</td>
<td>-</td>
<td>-</td>
<td>&gt; 1.00</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.50g</td>
<td>-</td>
<td>-</td>
<td>&gt; 1.00</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Noda et al. (1975)</td>
<td></td>
<td>PGA_H</td>
<td>-</td>
<td>-</td>
<td>&gt; 1.00</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Makadisi and Seed (1978)</td>
<td></td>
<td>0.20g</td>
<td>0.50</td>
<td>0.80</td>
<td>1.15</td>
<td>≈ 1.0</td>
<td>For M=6.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.75g</td>
<td>0.50</td>
<td>0.80</td>
<td>1.15</td>
<td>≈ 1.0</td>
<td>For M=8.25</td>
</tr>
<tr>
<td>Hynes-Griffin and Franklin (1984)</td>
<td></td>
<td>PGA_H</td>
<td>0.50</td>
<td>0.80</td>
<td>&gt; 1.00</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Kavazanjian et al. (1997) (*)</td>
<td></td>
<td>PGA_H</td>
<td>0.17</td>
<td>0.80</td>
<td>&gt; 1.00</td>
<td>1.0</td>
<td>With response analysis</td>
</tr>
<tr>
<td></td>
<td></td>
<td>PGA_H</td>
<td>0.50</td>
<td>0.80</td>
<td>&gt; 1.00</td>
<td>1.0</td>
<td>Without response analysis</td>
</tr>
<tr>
<td>Bray et al. (1998)</td>
<td></td>
<td>PGA_H</td>
<td>0.75</td>
<td>-</td>
<td>&gt; 1.00</td>
<td>0.15 – 0.30</td>
<td>Requires conservative strength values</td>
</tr>
</tbody>
</table>

(*) Reduction factor reported for soil conditions.

Regardless of the seismic coefficient developed by Noda et al. (1975), all the expressions were developed from procedures originally created to estimate displacements and in one way or the other all are based in the block principle first proposed by Newmark (1965). Performance of the slope is conditioned by allowable displacements and a minimum factor of safety; Duncan and Wright (2005) provides a good summary of displacement and factor of safeties values for an acceptable performance for many of the procedures described above.

Regarding the vertical seismic coefficient $K_v$, it has been usual practice either to assume a vertical component equal one half of the horizontal acceleration or neglect its contribution. Examples of recorded seismically activity (e.g. Towhata, 2008 ) shows that the maximum vertical acceleration is lower than the horizontal and it can be assumed with little error 0.5PGA, while neglecting its effect may lead to unstable slopes with factor of safety over 1.

Other issues related with this approach have been also identified. The assumption of locating the inertial force in the center of gravity of the slice is an issue identified by Seed (1979), where more critical Factors of Safety were obtained when the inertial force were applied in bottom of the slice. Also, Terzaghi (1950) commented about the inaccuracy of the model. Kramer (1996) based on Terzaghi’s work states that the selection of the seismic coefficient is not an easy job, and even with a FS > 1 the slope may fail. However, the experience has shown that this method combined with adequate engineering judgment may produce successful engineering designs of structural and geotechnical structures.
CHARACTERISTICS OF THE STUDY RECLAIMED MINE SITES

Geometry
A topographical survey data of the study sites was conducted using a Total Station instrument to determine the plot dimensions, and the inclination of the study sites estimated using a Suunto Mechanical Inclinometer model PM-5/360PC. Table 2, summarizes only slope length and approximate inclination of the slope. Further geometrical details are found in Jeldes et al. (2010).

Unit Weight
For each of the three sites unit weight data was collected using a Troxler 3411-B Nuclear Density Gauge (NDG) device between June, 2009 and August, 2009. A randomized systematic sampling technique (3m by 3m grid) to reduce data tendency or bias in the in measurements was employed (Sweigard et al., 2007b). Table 2 includes mean unit weight values for each site. More thorough information regarding data collection and data processing are found in Jeldes et al. (2010).

Preliminary Estimation of Shear Strength Parameters
Due to the extreme range of particles size (from clay particles to boulders), characterization and estimation of the shear strength parameters in mine spoil material difficult. Some attempts have been made to understand the role of large particles in the shear strength i.e. (Hosseinpour, 2008; Su, 1989), but there is no consensus on how large particles influence the shear strength, strain-stress behavior and failure mechanisms.

A preliminary estimation of the static internal friction angle under drained conditions for the three sites was offered by White et al. (2009). The estimation was made based on the observed angle of repose of the material, defined as “the steepest stable slope for loose packed granular material and represents the angle of internal friction at its loosest state” (Holtz and Kovacs, 1981). This assumption seems appropriate for the surface materials in FRA when the low compaction grading techniques are used (Jeldes et al., 2010). The granular nature of these materials in a loose state allows the assumption of negligible cohesion value (Holtz and Kovacs, 1981; Lambe and Whitman, 1969; Salgado, 2008)

For seismic analysis, typically undrained shear strength values are used since seismic excitation may be considered a short-term load condition, and excess pore water pressure may develop. However, it is assumed that the coarse, low density material resulting from the FRA compaction methods would drain appropriately, and the probability of significant excess of pore pressure development during an earthquake event is very low (Duncan and Wright, 2005). Consequently, identical drained and undrained strength parameters are used for both static and dynamic slope stability analysis for these reclaimed slopes. Table 2 includes estimated friction angles for each site.

<table>
<thead>
<tr>
<th>Sites</th>
<th>Slope Length (m)</th>
<th>Average Slope angle (Degrees)</th>
<th>Dry Unit Weight, γ (kN/m³)</th>
<th>Wet Unit Weight, γ (kN/m³)</th>
<th>Internal Friction Angle, φ (Degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Premium</td>
<td>31</td>
<td>28</td>
<td>16.2</td>
<td>18.5</td>
<td>38</td>
</tr>
<tr>
<td>National</td>
<td>48</td>
<td>20</td>
<td>18.5</td>
<td>20.3</td>
<td>37</td>
</tr>
<tr>
<td>Mountainside</td>
<td>46</td>
<td>28</td>
<td>18.9</td>
<td>20.4</td>
<td>38</td>
</tr>
</tbody>
</table>
A PROPOSED MODIFICATION OF THE INFINITE SLOPE EQUATION FOR HORIZONTAL AND VERTICAL PSEUDO-STATIC FORCES

The infinite slope method is a limit equilibrium analysis in which the failure surface is assumed to be roughly parallel to the ground surface, at a depth that is small with respect to the length of the slope (Skempton and Delory, 1957). This method is a simple and quick procedure for estimating the FS of a slope for cohesionless materials, when the fundamental failure mode is a shallow failure surface. Because the low compaction surface in slopes constructed according to the FRA creates a surface layer of loose material, this failure mode seems especially appropriate under these conditions. Slope stability analysis conducted by Jeldes et al. (2010), showed that the infinite slope equation for static conditions shows good agreement with limit equilibrium methods for the soil conditions present at those sites (e.g. search box feature in XSTABL with assumed soil properties for the base core material, and with a critical slip surface search focused on the upper 1.5 m) Additional analyses using method of slices (e.g. Simplified Bishop) simulating homogeneous slopes with properties of the weak, loose layer have also demonstrated that the fundamental failure mode is shallow and concentrated approximately in the first 1.5 m. Thus this failure mechanism was investigated for seismic stability as well.

Figure 3 shows a slip surface of depth $z$ below the ground surface (adapted from Salgado, 2008). The weight of the slice is $W$, where $W$ is the total initial weight of the soil, $b$ is the width of the slice and $z$ is the depth of the slice. The width $b$ may be expressed as $b = l \sin \beta$ (where $l$ is the length of the corresponding slip segment) and the slice weight may be expressed as $W = bhz$. $K_hW$ and $K_vW$ are the corresponding horizontal and vertical pseudo-static seismic forces applied in the center of mass of the segment and $P$ and $T$ are the corresponding resultant forces at the bottom of the slice.

![Figure 3. Infinite Slope Method. Figure Adapted From Salgado, (2008)](image)

The infinite slope assumption concludes that $Q_R$ and $Q_L$ on either side of the vertical element are opposite and equal, and can be neglected. Analyzing all the remaining forces in a simple rotated free diagram (rotated in a $-\beta$ angle with respect the horizontal), the corresponding resultant forces are

$$
\begin{align*}
(1) & \\
(2) & \end{align*}
$$

Where

$$
\begin{align*}
\end{align*}
$$

In term of the stresses the FS may be expressed as

$$
\begin{align*}
\end{align*}
$$
Inserting equations (1) and (2) into (4), we obtain the following FS in terms pseudo-static forces

\[ \text{FS} \]  

For cases of negligible cohesion, the equation simplifies to:

\[ \text{FS} \]  

If the contribution of the vertical ground acceleration is neglected, then equation (5) simplifies to an equation equivalent to the one proposed by Duncan and Wright (2005)

\[ \text{FS} \]

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SLOPE STABILITY ANALYSIS

Slope stability method and pseudo-static coefficient

In this analysis the proposed seismic Infinite Slope equation was employed and compared with the Simplified Bishop method of slices (XSTABL, 2008) in terms of resulting factor of safeties. All pseudo-static coefficients based on PGA previously discussed were tested using both procedures. The approach suggested Bray and Travasarou (2009) is also employed and discussed later in this article.

Peak Ground Acceleration (PGA)

Estimation of the PGA was made based on local hazard maps developed by the U.S. Geological Survey (USGS) web site (http://earthquake.usgs.gov/earthquakes). For a 2% of probability of exceedance in 50 years, the PGA was estimated to be 0.2g and for 10% of probability of exceedance in 50 years, the estimated PGA was 0.07g. It is important to mention that the PGA varies slightly among the three sites, but the maximum PGA found among the three sites (0.2g and 0.07g) are used in the analyses.

Considered geometry and soil properties

A generic slope representative of the most probable severe condition at the 3 project sites was investigated. The slope was assumed to have a height of 14 m and an inclination of 28 degrees, with the properties given in Table 2 for the Premium site.

Makdisi and Seed (1978) reported behavior of clay soils under cyclic loads and indicated that “in most cases this value (cyclic shear strength) would appear to be 80% of the static undrained strength” suggesting later that 0.8 would be an appropriate value for stability analysis. The same reduction factor was proposed by Hynes-Griffin and Franklin (1984). Kavazanjian et al.(1997) proposed the same reduction factor only when fully saturated or sensitive clays are being analyzed. During cyclic loading, clays usually present a high non-linear shear modulus behavior that decreases with the number of cycles. Idriss et al.(1978) compared the hysteresis loop behavior of a clay soil at 1 and 10 cycles, showing that clays tend to become softer when the number of cycles increases. The reasons may be found in the excess of pore water pressure developed during the cyclic excitation that in turn decreases the effective stress,
and the destruction of the electrical and chemical bonding between clay particles (Towhata, 2008). On the other hand, granular material usually behaves in the opposite way; it tends to become stiffer when the number of cycles increases. After the discussion presented above, the application of the suggested strength reduction factor may be not applicable for the type of material found at these sites, and therefore no reduction was applied to the material properties.

**Resulting Factor of safeties**

Static slope stability analysis conducted for these sites have showed that the minimum FS is essentially the same when the slope is modeled assuming a surface layer with a uniform thickness of 1.5 m over a highly strong core-base material or when it is modeled as a homogeneous slope with the soil properties of the looser surface layer; the fundamental failure mechanism runs near the surface due to the frictional behavior of the soil present in the region. Then, a homogeneous slope model was analyzed using the software XSTABL and the Simplified Bishop method of slices. Table 3 summarizes the obtained FS using the proposed Infinite Slope equation and Simplified Bishop method, for a variety of pseudo-static coefficients.

**Table 3. Summary of horizontal and vertical pseudo-static coefficients and obtained Factor of safeties for a 2 and 10% of probability of exceedance in 50 yr**

<table>
<thead>
<tr>
<th>Method or Reference</th>
<th>Seismic Coefficient</th>
<th>Computed Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>K</td>
<td></td>
</tr>
<tr>
<td>Theoretical d’Alembert Principle</td>
<td>Kh</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>Kv</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Makadisi and Seed (1978) (M≈6.5)</td>
<td>Kh</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>Kv</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hynes-Gri... and Franklin (1984)</td>
<td>Kh</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td>Kv</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Noda et al. (1975)</td>
<td>Kh</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>Kv</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bray et al. (1998)</td>
<td>Kh</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>Kv</td>
<td>0.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kavazanjian et al. (1997) (Without response analysis)</td>
<td>Kh</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>Kv</td>
<td>0.10</td>
</tr>
</tbody>
</table>

For all pseudo-static coefficients, all combination of force orientations were tested, with the most critical being when the vertical seismic force and the horizontal seismic force act simultaneously away from the slope. Notice the good agreement between the proposed infinite slope equation and the Simplified Bishop method in terms of critical factor of safety for seismic loads.

When predicted parameters for a 10% of probability of exceedance in 50 yr are used, almost all factors of safety are well above 1. It is expected then that the shear strength along the slip surface be 25-31% greater than the required to maintain equilibrium. For a 2% of probability of exceedance, the computed factors of
safety are naturally much lower. When the applied inertial force is applied directly (without acceleration reduction factors summarized in Table 1) the obtained FS=0.91 is below the unity and indicates failure. Hynes-Griffin and Franklin (1984) and Makdisi and Seed (1978) approaches give a FS=1.14-1.15. Hynes-Griffin requires a minimum FS=1, while Makdisi and Seed require minimum FS=1.15. Underthese criteria, the slope should be stable. Under static conditions, the factor of safety for those sites is FS=1.47, meaning that under seismic conditions the ratio of available shear strenth to shear stress is reduced about 30%.

Notice that the equation proposed by Noda et al. (1975), for a 2% of probability of exceedance in 50 yr., gives factors of safety similar to the case when no reduction has been made to the inertial force, but for a 10% of probability of exceedance in 50 yr. Noda’s procedure computes the lowest factor of safety among the authors (FS=1.07), which is even lower than the factor of safety for the maximum ground acceleration without reduction. Noda et al. (1975) developed the equation based on limit-equilibrium analysis to structures (quay walls) that were already damaged by earthquakes, where the results and parameter selection probably have a high level of subjectivism. As Towhata (2008) states “…there are many uncertainties in this study, probably including the determination of appropriate soil strength in the limit equilibrium analysis”.

Regarding seismic coefficient to be used for these sites, Hynes-Griffin and Franklin (1984), Bray and Rathje (1998) and Kavazanjian et al. (1997) seem to be the most applicable approaches for these sites. Even though Makdisi and Seed (1978) produces give the same $K$ value than Hynes-Griffin, it is based in a most general approach. Since current hazard maps have broad coverage and easy access, a more site-based design is possible.

**SLOPE STABILITY CHARTS BASED ON SPECTRAL ACCELERATIONS**

Based on a new procedure proposed by Bray et al. (2009) and the proposed seismic infinite slope equation, a simple, graphical procedure is developed for a quick estimation of the factor of safety.

**Bray’s procedure for calculating the pseudo-static seismic coefficient**

Bray and Travasarou (2007), using 688 earthquake records and a non-linear coupled stick-slip deformable sliding block model, developed a semi-empirical probabilistic equation for estimating permanent displacements due to seismic forces. Rathje and Bray (2001) in a previous work suggested that a stick-slip deformable sliding block model offer a more realistic representation than the rigid block model first proposed by Newmark (1965). Using this model, a series of displacements where calculated for different yielding seismic factors ($K_y$) and initial fundamental periods ($I_s$). Then, a relationship between induced displacement and a single value of motion intensity was found to be optimally satisfied nor by PGA, but by the 5% damped elastic spectral acceleration ($S_a$) at the degraded fundamental period. They found that the spectral acceleration at the degraded period is essentially $1.5I_s$, due to material non-linearity. Later, Bray and Travasarou (2009) offered a procedure to calculate the pseudo-static coefficient based on the Bray and Travasarou approach; now, instead of calculating displacements, the yielding seismic coefficient can be estimated as a function of allowable seismic displacement ($D_a$), the 5% damped elastic spectral acceleration ($S_a$) of the site, the initial fundamental period ($I_s$), a random normal distributed variable ($\varepsilon$) and earthquake magnitude ($M$). The Bray and T seismic coefficient is:

\[
(8)
\]

\[
(9)
\]

\[
(10)
\]
The assumptions behind the original model are a uniform unit weight equal to 17.6 kN/m$^3$, shear modulus dependent on the strain level and soil damping ratio for plasticity index = 30%. However, as the authors states, adjustments to those parameters do not produce significant effect on the displacements (Bray and Travasarou, 2007) and therefore do not significantly impact the calculated seismic coefficient.

**Determination of the Spectral acceleration**

Local spectral acceleration values as a function of the earthquake period for every site were obtained from the USGS web site (http://earthquake.usgs.gov/earthquakes) for 2 and 10% of probability of exceedance in 50 years. Here, one or more ground acceleration records representative of the “maximum credible ground motion” are converted to response spectrum, usually for a 5% damping. After smoothing, a curve of $Sa(T)$ is obtained and used for engineering purposes (National Research Council (U.S.). Committee on Safety Criteria for Dams., 1985). Figure 4 shows the spectral acceleration plot for the Premium site.

![Figure 4. Spectral Acceleration Plot for Premium site (USGS)](image)

**Proposed equation for determining Factor of Safety**

Bray’s seismic coefficient was unified with the proposed infinite slope equation to yield a single expression for estimating the FS of reclaimed slopes under seismic conditions:

$$\text{FS} = \frac{a}{b}$$

Where $a$ and $b$ are defined by equations (9) and (10). The vertical coefficient $Kv$ was assumed to be one half of the horizontal coefficient $Kh$.

**Solution charts**

Graphical chart solutions of equation (11) for different earthquake magnitudes, maximum allowable displacements and soil properties have been developed. Even though the proposed solution is valid for any slope constructed with FRA and any reclaimed slope built with granular material whose fundamental failure mode shallow is (i.e. cohesionless material), the example solution presented in Figure 5 is applied to the particular conditions at Premium site. Similar plots were developed for the other two sites. The magnitude of the earthquake, for simplicity was assumed to be equal to 5.0, since this is about the highest earthquake magnitude usually recorded in the zone. Also, here the allowable displacement is assumed to be 30 cm, while other plots have been created for different allowable displacements. The random variable
ε was selected to be 0.66 representing a 16% of allowable displacement exceedance (Bray and Travasarou, 2009).

**Figure 5. FS chart for Premium site as a function of Spectral Acceleration and initial fundamental period**

**Illustrative example**

For this example, a slope with the geometric conditions and soil properties described for the Premium site is used for the simplicity of the illustration. The designer needs to decide what magnitude $M$ and maximum allowable displacement ($D_a$) to use, according to the available local seismic information and importance of the project. For Premium site, $M=5.0$ and $D_a=30$ cm may be acceptable, since those slopes are constructed in rural areas. Other magnitudes and displacements may be selected as well. The initial fundamental period of the slope may be calculated using

$$T_s = \frac{H}{V_s}$$

where $H$ is the slope height and $V_s$ average shear wave velocity of the slope. Since we have not measured the shear wave velocity at the site, the assumption of $T_s=0.5$ s and the use of $S_a(T_s=0.2)$ are probably reasonable and within a safe bound. Using Figure 4, $S_a \approx 0.45$. Then, for $T_s=0.5$ s and $S_a=0.45$ g the computed FS is approximately 1.17.

**DISCUSSION AND CONCLUSIONS**

A modification of the infinite slope equation was presented, for simple and quick determination of factor of safeties for reclaimed slopes constructed on a FRA basis in the Appalachian region, or any reclaimed slope built with frictional material where the fundamental failure mode is shallow. The infinite slope equation showed good agreement with the simplified Bishop method of the slices under seismic loads. It was also discussed that the selection of the pseudo-static coefficient requires an understanding of the assumptions behind the coefficient. For these sites, reasonable results were obtained when the horizontal seismic coefficient proposed by Hynes-Griffin and Franklin (1984), Bray and Rathje (1998) and Kavazanjian et al. (1997) where employed. Nevertheless, using a fraction of the PGA to represent the behavior of a slope under seismic excitation may not be accurate enough since permanent displacements or slope failure may be the result of incremental slope failures, which is related with the time duration of maximum credible earthquake and number of cycles of strong motion (National Research Council (U.S.). Committee on Safety Criteria for Dams., 1985). Also, ground motions with similar PGA may significantly differ in terms of frequency and duration (Bray, 2007). Under these conditions a procedure based on spectral response may provide a better tool for evaluating slope stabilities. The combination of Bray and Travasarou (2009) approach with the proposed modification of infinite slope equation yielded a simple
procedure for estimating factor of safeties as a function of parameters that somehow have less selection subjectivity for the designer. Charts were developed for this method employing site-specific $Sa$ values and local soil properties and slope geometry. The selection of $Da$, $Sa$, $M$, and $Is$ rest on the engineering criteria in the slope design of these reclaimed slopes. In general, those parameters need to be a function of the importance of the project, available seismic data, and accuracy of the site characterization. It is suggested the use of spectral values predicted for a 2% of probability of exceedance in 50 yr or higher, except when the designer has supported reasons for using lower local values, stating clearly the design assumptions.

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REFERENCES


Hynes-Griffin, M.E., and A.G. Franklin. (1984). Rationalizing the seismic coefficient method. Army Engineer Waterways Experiment Station Vicksburg Ms Geotechnical Lab Vicksburg, MS


Sweigard, R.J., V. Badaker, and K. Hunt. (2007b). Development of a field procedure to evaluate the reforestation potential of reclaimed, surface-mined land. Department of Mining Engineering, University of Kentucky, Lexington, KY


