Preliminary Investigation of Liquefaction Potential of Cemented Paste Backfill Under Dynamic Loading Using a Two-Dimensional, Time-Domain Ground Response Analysis

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ABSTRACT

Cemented paste backfill (CPB) is the emerging trend in integrated mine tailings management. CPB consists of mine tailing, binder agents, and water that provides ground support for underground openings known as stopes. CPB has tight characteristics and a very high delivery rate compared to other backfilling materials such as rockfill. Once placed in a stope, CPB is generally considered as a loose, saturated granular soil at its early stage of binder hydration. In this state, CPB might be prone to liquefaction if subjected to static or dynamic loadings. CPB liquefaction would lead to the application of excessive horizontal loads that can lead to barricade failure.

CPB liquefaction is much more complicated that the conventional liquefaction analysis of granular materials such as sand. Hydration and confined environment of a stope are a couple of these complicating factors in addition to the type of input ground motions (i.e., earthquakes vs mining-induced seismic loads). To be able to establish a design procedure, CPB liquefaction analysis must be done first using the conventional geotechnical earthquake engineering concepts. The current state of practice assumes that once the unconfined compressive strength of CPB reaches 100 kPa, the material is non-liquefiable (Clough et al. 1989).

In this paper, first a time domain ground response analysis is performed using FLAC3D. The time dependent material properties due to binder hydration are incorporated into the analysis. FLAC3D results are then compared with the one dimensional equivalent linear analysis results using SHAKE2000. The induced shear stresses are then compared with the laboratory obtained cyclic shear resistances to identify the regions of backfilled CPB that are prone to liquefaction.

INTRODUCTION

Over the past decade, cemented paste backfill (CPB), which is a mixture of mine tailings, water, and binder agents, has gained popularity as backfill material to fill previously mined underground openings (stopes). Fresh CPB is considered a non-self-supporting material when placed inside the stope. CPB will be transformed into a self-supporting material by time due to the cement hydration and consolidation processes.

Fresh CPB has to be held inside the stope using a concrete wall built in the undercut of a stope, called a barricade. The barricade is subjected to horizontal forces that emerge from within the body of the CPB. These forces are functions of effective stresses within CPB. The effective stresses would change due to static loads (e.g., CPB self-weight consolidation), dynamic loads (e.g., earthquakes or rockbursts), and cement hydration. Barricade must be designed to withstand these horizontal stresses. Under undrained loading conditions (static or dynamic), pore pressures within CPB would increase and effective stresses decrease. If the effective stresses reduce to zero, the material is deemed to be liquefied (Kramer 1996). CPB liquefaction of newly placed CPB due to static or dynamic loadings would lead to changes in the stresses that are acting on the barricade. Therefore, the liquefaction susceptibility of CPB at the early stage of hydration is of concerns in the design of barricades and CPB systems.

The state of practice in paste technology is to add a small quantity of cementitious materials (i.e., binder agents) to backfill in order to improve short term and long term strength. The ‘rule of thumb’ used to consider backfill as liquefaction resistant is to achieve an unconfined compressive strength (UCS) of 100 kPa (le Roux, 2004). This guideline has been adopted from the special case study on clean rounded cemented sand (Clough et al., 1989). However, this guideline might be conservative for the design of CPB systems and needs to be investigated.

The recent in-situ measurements showed that even with the formation of cement bonds, an extensive amount of pore water pressure is developed (which may result in liquefaction) within CPB and in the areas behind the barricade for an extended period after the deposition (Shahsavari and Grabinsky 2015 and 2016; Grabinsky et al. 2014; Thompson et al. 2012; Fahey et al. 2010). Moreover, the cement bonds are not well connected compared to the bonds that form in any other cemented materials such as concrete (Ramlochan et al. 2004). Therefore, the liquefaction analysis of fresh CPB can be performed within the boundaries of conventional geotechnical earthquake engineering principles.

To assess the liquefaction potential of CPB in a backfill system due to dynamic loadings, Shahsavari et al. (2014) used a screening method. In this method, an equivalent linear one-dimensional ground response analysis was performed to obtain the cyclic stress ratio (CSR) and laboratory test results were used to obtain the cyclic resistance ratio (CRR) of the material. Factor of safety against liquefaction (FSL) was then calculated using the ratio of CRR/CSR. It was shown that the CPB system was liquefaction resistant (FSL>1) under the dynamic loadings and backfill system geometry assumptions. This method was adopted based on the current state of practice for the level ground liquefaction assessment explained in Kramer (1996). However, the screen method (Shahsavari et al., 2014) ignored the effect of confinement (i.e. the surrounding rocks) on the dynamic response of CPB. The CPB in a backfill system is confined with rocks and hence one-dimensional ground response analysis may not be valid to obtain the CSR value.

In this paper, first, a time domain ground response analysis was performed under a rockburst loading event using FLAC3D to consider the effect of surrounding rocks. FLAC3D results are then compared with the one-dimensional equivalent linear analysis results presented in Shahsavari et al. (2014). To make such a comparison, the same time dependent CPB properties previously used in Shahsavari et al. (2014) were incorporated into the FLAC3D analysis. Second, the CRR values were obtained from the laboratory measurements are then compared with the CSR values obtained using the time-domain ground response analysis. The details of the method are explained in the following sections.

CEMENTED PASTE BACKFILL PROPERTIES

As explained in the previous section, the same time dependent CPB properties previously used in Shahsavari et al. (2014) were used in this study. The mine tailings were obtained from Kidd Mine and all geotechnical laboratory tests including index properties measurements and cyclic triaxial testing were performed by Abdelaal (2011) at the University of Toronto.

CPB Geotechnical Index Properties

The CPB was a mixture of mine tailings, sand, binder, and water. The tailings have a liquid limit and plastic limit of 23% and 18%, respectively and hence a plasticity index of 5% (Abdelaal 2011). At this plasticity index, the mine tailings are classified as non-plastic (Boulanger and Idriss 2004).

The binder used in the CPB was a mixture of 90% blast furnace slag and 10% Portland cement. The CPB had a binder content of 4.5% of the dry weight of the solids. The fresh CPB had a water content of 25% when placed in the stope and hence was fully saturated. The initial unit weight of the material was 19.7 kN/m^3 (Abdelaal 2011). The solids portion consisted of 55% mine tailings and 45% sand. Grain size distribution of mine tailings, sand, and the mixture of mine tailings and sand (MTS45) are shown in Figure 1. The 7 month old undisturbed samples of the same CPB that was obtained from the mine site showed minimal changes in the void ratio and water content[1]. Therefore, the effect of self-weight consolidation and hydration on the unit weight was ignored in the current analysis and a constant unit weight was assumed.

CPB Dynamic Properties

The CPB was considered as a time dependent material due to the hydration of binder agents. It has been shown that the calcium-silicate-hydrate (C-S-H) bonds are usually formed within CPB (Ramlochan et al. 2004). The C-S-H bonds presence changes material's stiffness and tortuosity continuously leading to time dependent shear and bulk moduli values. Abdelaal (2011) performed constant strain cyclic triaxial tests on samples of a Kidd CPB and obtained the time dependent small strain (i.e. maximum) shear modulus. The variation of the maximum shear modulus with time is shown in Figure 2. To obtain the bulk modulus, a constant Poisson's ratio of 0.25 (Abdelaal 2011) was used in the analysis.

Maturity function is widely used in concrete technology to show the time dependency of concrete properties (Illston et al., 1979) and is adopted for this study. The maturity function for the shear modulus is shown in Eq.1 (Shahsavari and Grabinsky 2016).

$$G = G_i + a * \exp(-d / \sqrt{t - t_i}) \qquad [1]$$

where G_i and t_i are the initial shear modulus and time prior to the cement hydration initiation and take values of 2.7 MPa and 3 hours for the Kidd CPB, respectively. a and d are fitting parameters to the laboratory testing results (Abdelaal, 2011) and have values of 177.31 MPa and 4.67 hour ^{½}, respectively. The measured and fitted shear modulus values for the Kidd CPB are shown in Figure 2.

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[1] Internal communication within the geotechnical engineering lab at University of Toronto

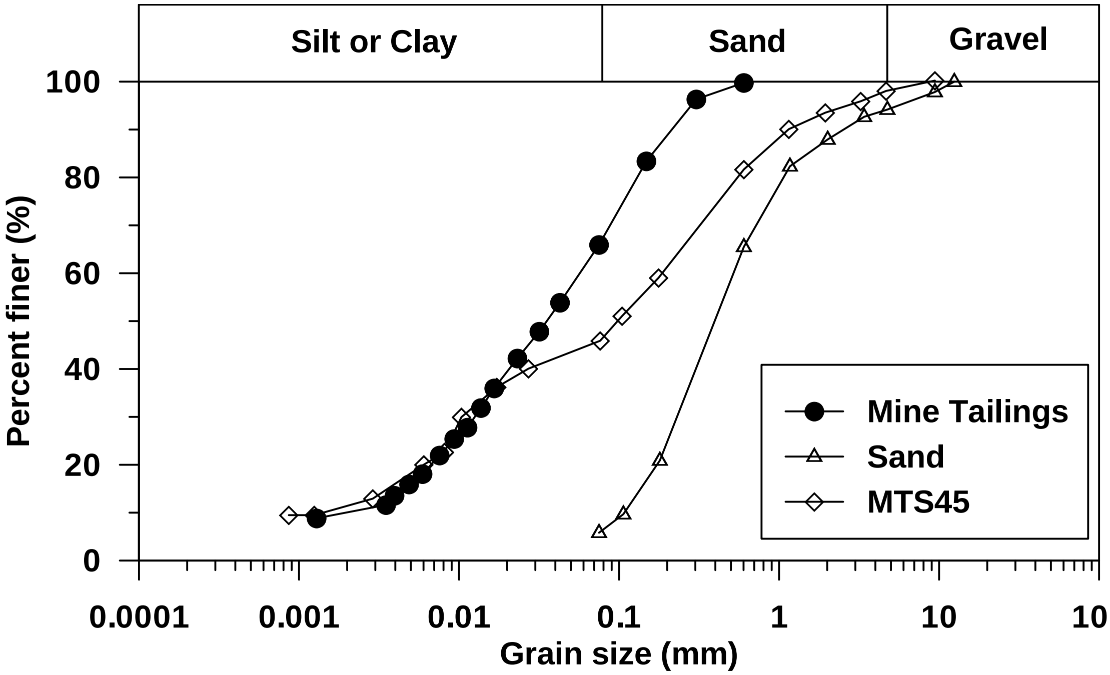

Figure 1. Grain size distribution of Kidd mine tailings, sand, and mixture of 55% mine tailings and 45% sand (MTS45) (after Abdelaal, 2011)

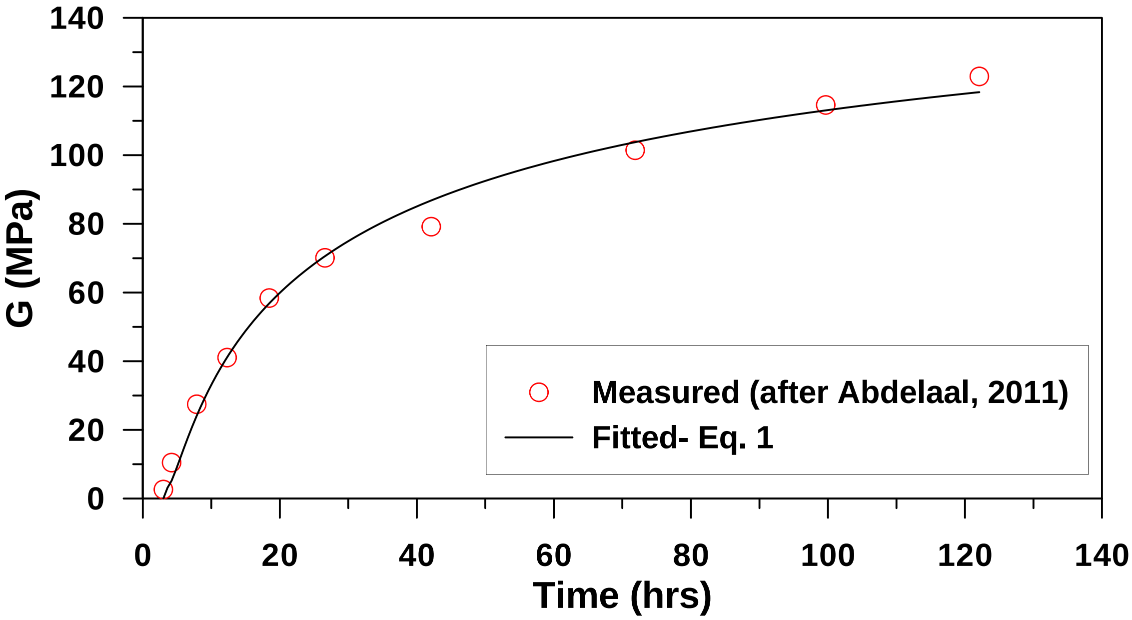

Figure 2. Small strain shear modulus variation with time

Cyclic Resistance Ratio

Based on Seed (1983), the CRR of soils can be obtained using the laboratory measurements. However, it has been noted that the triaxial or the simple shear laboratory testing are not well suited in repressing in-situ conditions prior to liquefaction (Youd et al. 2001). Therefore, correction factors must be applied to the laboratory measured CRR values.

Laboratory CRR

Generally, the CRR is defined as the cyclic shear stress over the vertical stress ratio that causes liquefaction under a fixed number of cycles. Based on National Research Council (NRC, 1985) recommendations, a soil is considered liquefied if the shear strain exceeds 3.75% (or its axial strain equivalent). To obtain the CRR of a specific soil, cyclic triaxial or simple shear tests can be performed at different levels of peak shear stresses using uniform cycles of loading. Then the CSR that causes liquefaction at a specific number of uniform cycles of loading is interpolated and used as the CRR in the analysis. Figure 3 shows the variation of number of cycles to liquefaction with the magnitude of CSR under the cyclic triaxial loading mode for the Kidd CPB (Abdelaal 2011).

The design seismic event for this analysis is a rockburst event with a magnitude of M3.1 (Shahsavari et al. 2014). The rockburst magnitude corresponds to 5 cycles of uniform loading (Gheibi and Bagheripour 2011; Seed et al. 1975; and Idriss 1997). Therefore, the CRR was estimated to be 0.21 based on the CSR curve shown in Figure 3 and 5 cycles of uniform loading.

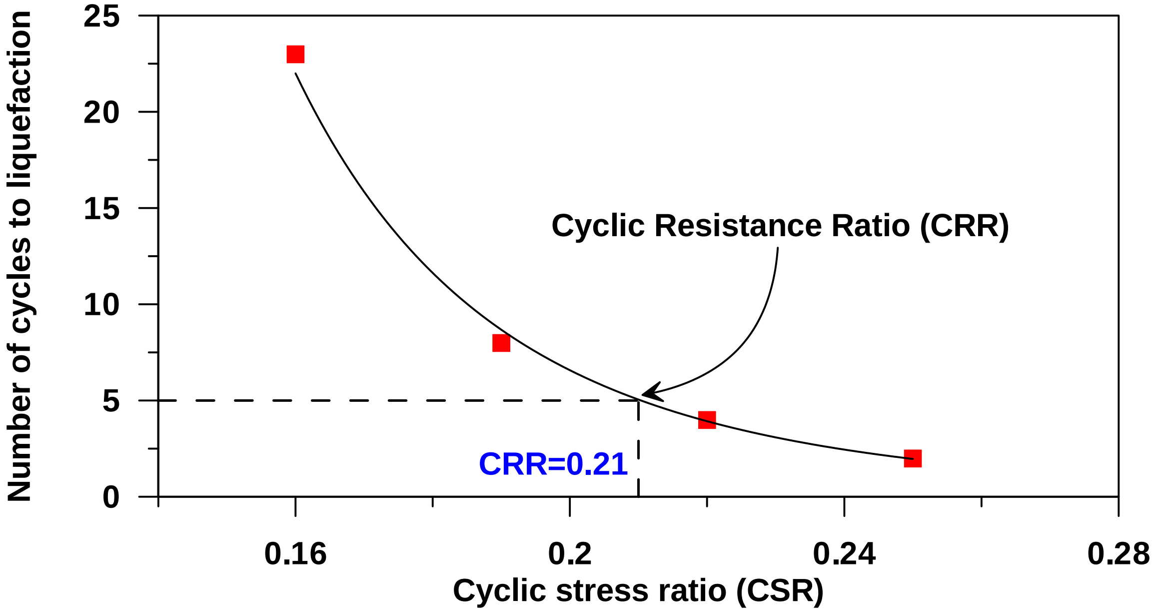

Figure 3. Variation of number of cycles to liquefaction with the cyclic stress ratio (after Abdelaal 2011)

In-situ CRR

The CRR values are usually obtained from cyclic triaxial tests on saturated soil samples that are consolidated isotropically under 100 kPa effective confining stress. However, the in-situ loading condition are not always isotropic and the confining (or vertical) stresses vary with depth. To be able to obtain the in-situ CRRs properly, correction factors must be applied to the CRR values obtained from laboratory measurements. Idriss and Boulanger (2008) showed that the CRR values obtained from triaxial cyclic tests must be corrected for the in-situ state of stresses using Eq. 2.

$$CRR_{field} = 0.90 * \frac{1 + 2 * k_{0,field}}{3} * CRR_{triaxial} \qquad [2]$$

where $k_{0,field}$ is the ratio of lateral to vertical in-situ effective stresses. Due to hydration $k_{0,field}$ is not constant and changes with time. Thompson et al. (2012) field measurements at a different mine site showed that the variation of the lateral earth pressure coefficient within the backfilled CPB does not follow a specific path. For the simplicity of the analysis in the current study a mean value of 0.33 was used for $k_{0,field}$. Shahsavari et al. (2014) showed that Eq.2 is not very sensitive to the range of reported in-situ lateral earth pressure coefficients and the assumption of a constant $k_{0,field}$ is reasonable.

Another important factor that influences the CRR of granular materials is the void ratio. Several researches showed that the CRR increases as void ratio decreases (e.g. Shahsavari Goughari 2012 and Vaid et al. 2001). Samples obtained from the cured backfilled CPB show minor changes in the void ratio during and after deposition (Grabinsky et al. 2008). However, the triaxial samples used by Abdelaal (2011) has a void ratio of 0.42 which is almost 40% of the maximum attainable in-situ void ratio for Kidd CPB (Grabinsky et al. 2014). Therefore, the CRR obtained from Figure 3 must be corrected for the in-situ void ratio. It has been shown that the cyclic resistance of loose sand is almost 30% of that of dense sand (Shahsavari Goughari 2012; Vaid et al. 2001; and Vaid and Sivathayalan 2000). Therefore, the CRR value is reduced by 30% to account for the effect of void ratio.

In addition, the in-situ vertical (or confining) stresses are not always 100 kPa at all depths as they are assumed in obtaining the laboratory CRR values. Boulanger (2008) suggested Eq. 3 to correct the experimentally obtained CRR values for the effective overburden pressure (σ'_{vc}).

$$K_\sigma = 1 - C_\sigma \ln(\frac{\sigma'_{vc}}{P_a}) \leq 1.1 \qquad [3]$$

where $C_\sigma = \frac{1}{18.9-17.3D_{rc}} \leq 0.30$, D_{rc} is the relative density, σ'_{vc} is the effective vertical stress, and P_a is the atmospheric pressure. Due to a lack of data on the maximum and minimum void ratios of CPB, a relative density of 30% is assumed in Eq.2. Figure 4 shows the variation of the triaxial CRR (Abdelaal 2011) and the corrected in-situ CRR values with depth for the Kidd Mine stope geometry used in this study. The effective stresses in Eq. 3 correspond to the end of deposition values. These effective stresses were obtained using the same modeling approach introduced by Shahsavari and Grabinsky (2016) to model the CPB consolidation. The effect of drainage and hydration are considered in the vertical effective stresses calculation. Further details of the consolidation analysis is explained in Shahsavari and Grabinsky (2016) and is beyond the scope of this paper.

GROUND RESPONSE ANALYSIS

Ground response analysis can be performed through empirical equations (e.g. Gheibi et al. 2014), simple constitutive modeling, or advanced constitutive modeling. In the previous analysis, Shahsavari et al. (2014) performed a simple one-dimensional ground response analysis called the equivalent linear method using SHAKE2000 (Ordonez 2008) to obtain the response of the Kidd CPB to a rockburst event and an M5.9 magnitude earthquake (Richter scale). The equivalent linear method assumes that the ground is unconfined and the material behaves elastically. The equivalent linear method analyzes the ground response in the frequency domain and hence can use the frequency independent soil damping parameters. To consider the effect of ground confinement more advanced methods must be used. In this paper, a time domain method of analysis was performed using the coupled hydro-mechanical approach introduced by Shahsavari and Grabinsky (2016) and developed in FLAC3D. In the time domain analysis, the equation of motion for all grid points is solved simultaneously (Park and Hashash 2004). In the following sections the input motions and the numerical analysis details are explained.

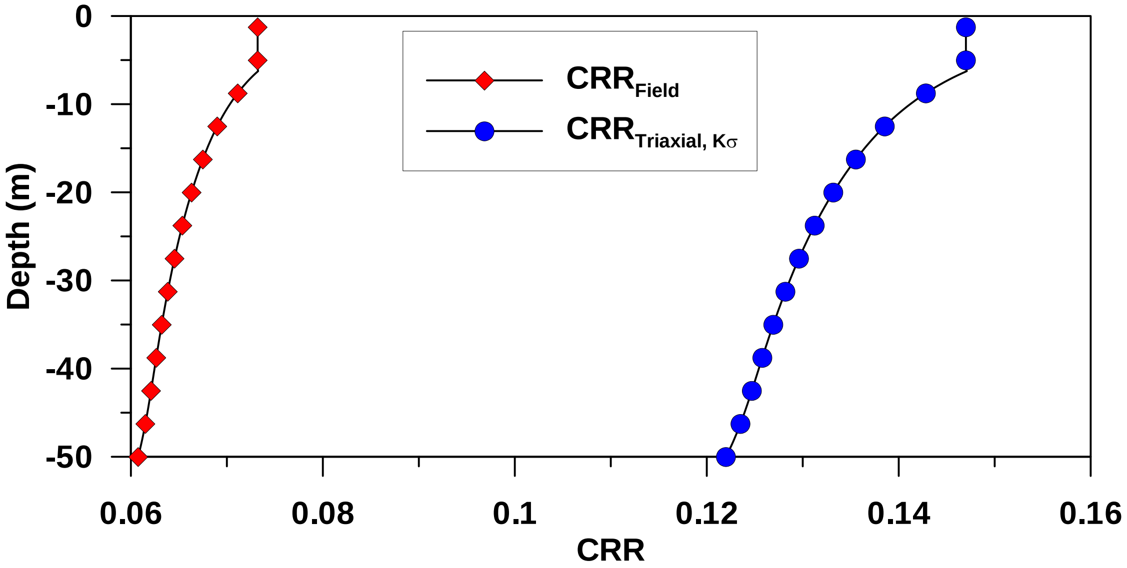

Figure 4. Variation of corrected Kidd Mine CPB CRR with depth based on Abdelaal (2011) laboratory testing

Input Motion

A rockburst event that was recorded at the Kidd mine site in 2009 was used in the current analysis. The input motion had a peak acceleration of 0.4 g (Saebimoghaddam 2010). The time history of the input motion is shown in Figure 5.

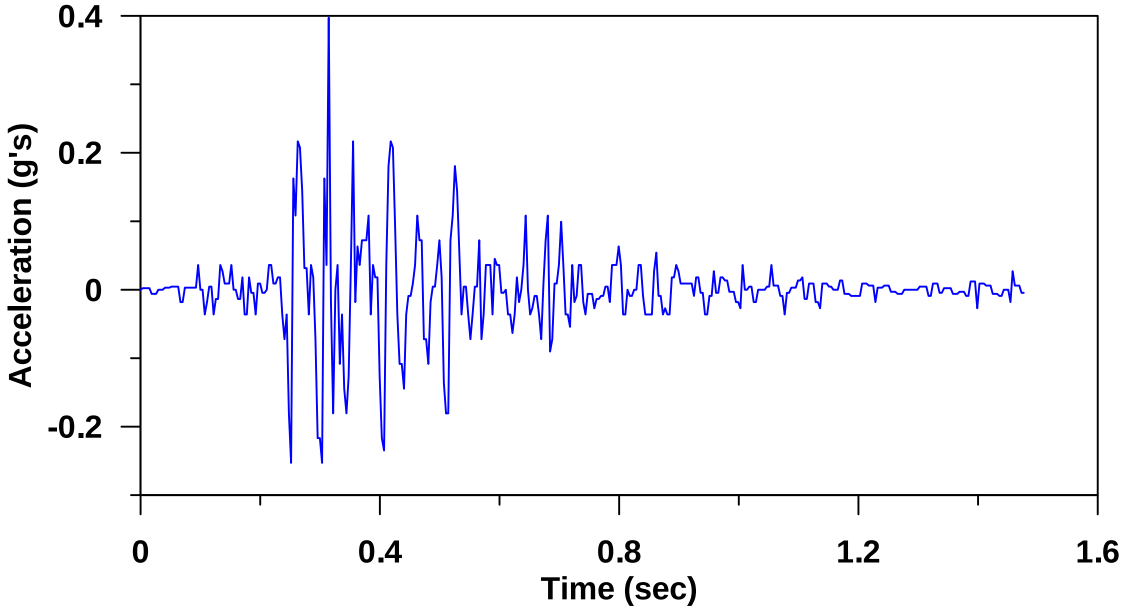

Figure 5. Acceleration time histories of the Rockburst event (after Saebimoghaddam 2010)

Numerical Analysis

FLAC3D is used to simulate the dynamic event. The stope in the current analysis is 20 m x 20 m in the horizontal section and 50 m deep. This stope is selected to make the comparison with the previous ground response analysis that used the equivalent linear method and SHAKE2000 (Shahsavari et al. 2014). A two-dimensional geometry was created in FLAC3D. Figure 6 shows the geometry of the model.

The FLAC3D model was divided into zones of 0.125m thick in both x and z directions. In the horizontal direction the model dimension was chosen to be 40 m to accommodate two pillars of rock surrounding the CPB. The rock had a Poisson's ratio of 0.2 and Young's modulus of 50 MPa with a dry density of 28 kN/m^3. It was assumed that the rock is impervious and the water flow only takes place in the vertical (z) direction within the CPB. First the rock body was excavated and then backfilled using the coupled hydro-mechanical approach introduced by Shahsavari and Grabinsky (2016). CPB was consolidated and hydrated as it was being deposited. This modeling approach allowed the use of time dependent material properties. Both the surrounding rock and CPB were assumed to behave elastically.

The dynamic load was then applied at the end of deposition. For the sake of simplicity and comparison with the one-dimensional ground response analysis, the input motion was applied to the bottom of the model. In one-dimensional ground response analysis Shahsavari et al. (2014), the input motion was applied at the top of the rock bellow CPB. However, in the current analysis, the motion was directly applied to the bottom of both CPB and the surrounding rock. This loading scenario is more severe than the previous analysis done by Shahsavari et al. (2014).

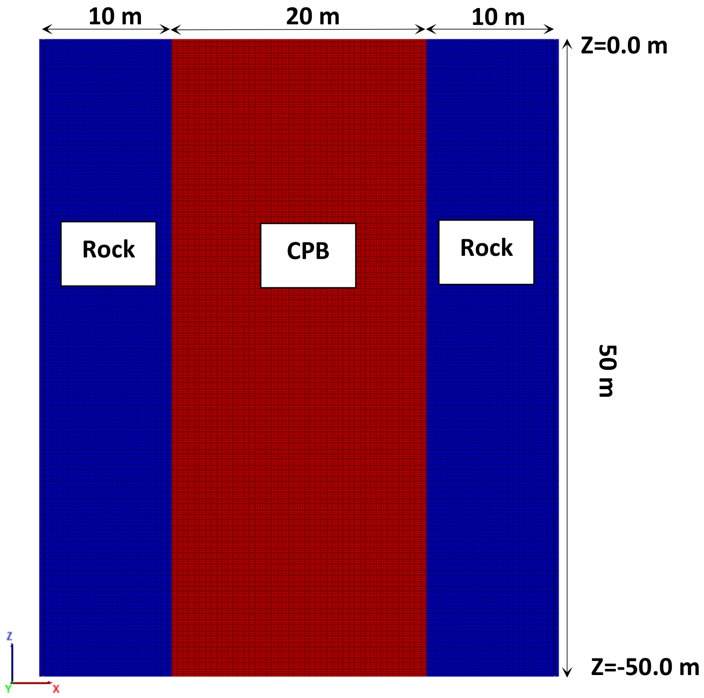

Figure 6. FLAC3D model geometry

Cyclic Stress Ratio (CSR)

Figure 7 presents the variation of peak ground acceleration (PGA) and the CSRs with depth based on the current and the previous analyses (i.e the equivalent linear method). It is apparent that the top 35 m of the CPB in the stope has higher CSR values in the two-dimensional analysis (using FLAC3D) compared to the one-dimensional analysis (using SHAKE2000). However, both SHAKE2000 and FLAC3D analyses yield the same peak shear stresses at depths below 35m. This similarity in the results can be attributed to the fact that the cured CPB has a shear modulus that is very close to the surrounding rock's shear modulus below 35 m. This similarity in properties reduces the 2D analysis to 1D unconfined type of analysis.

LIQUEFACTION POTENTIAL

The liquefaction potential of the CPB profile was assessed by a comparison between the field CRR (Figure 4 and also shown in Figure 7a) and the CSR estimated in this analysis. Figure 7(a) shows that CPB in the top 5 m of the stope would liquefy as its CRR is less than the induced CSR. However, the liquefaction of the CPB in the top 8 m is not of design concern. The CBP behind the barricade is safe against liquefaction in this loading scenario.

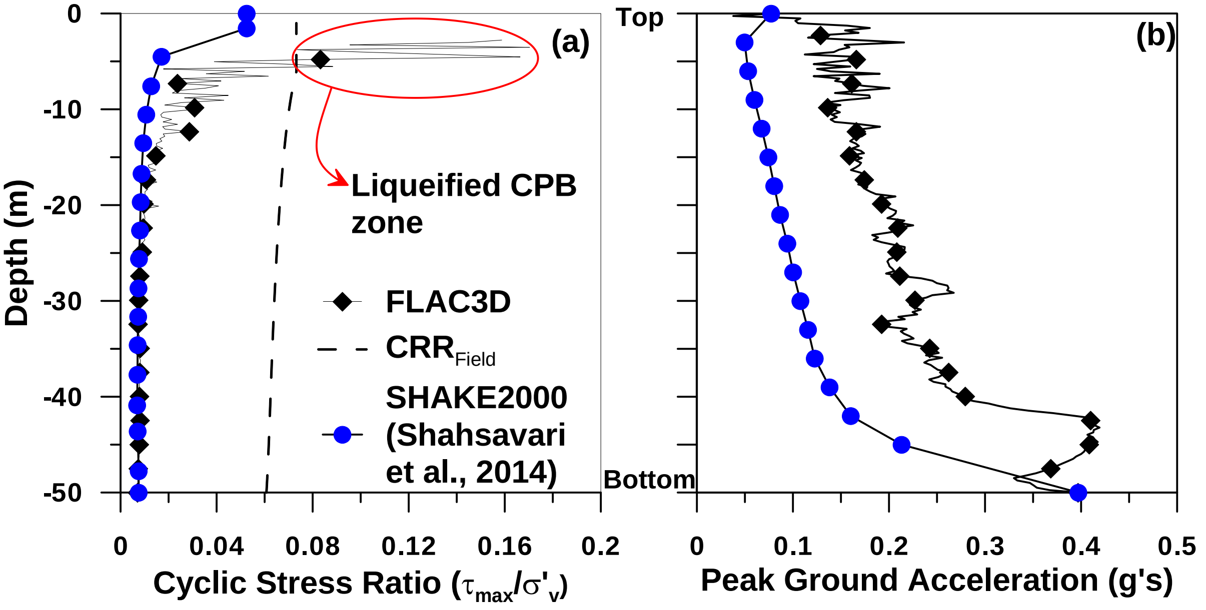

Figure 7. Variation of (a) CSR and (b) peak ground acceleration with depth

The finding that the CPB in the top 5 m of the stope liquefies implies that fresh CPB can be prone to liquefaction if the effect of confinement is considered in the dynamic analysis. SHAKE200 results (Shahsavari et al. 2014) showed that even the top layers of the CPB will not liquefy under the same rockburst event. It must be noted that the fresh CPB’s CRR was used to evaluate the liquefaction potential. The cured CPB has a higher CRR and hence the FSL would be even higher for the CPB at depths bellow 35 m.

Analysis Limitations

Several assumptions were made to make the current liquefaction analysis possible. First of all, the correction factors that were used to obtain the field CRR were based on laboratory tests on sand. These correction factors might be higher or lower that the used values for mine tailings and specifically CPB. However, 55% of the CPB used in this study is sand and the use of these correction factors seems reasonable for an early stage analysis. For a more accurate analysis, further investigation of these factors is needed for different CPB mixtures. Suazo et al. (2016) studied the effect of void ratio and overburden pressure on CRR of unclassified tailings and showed that the sand K_σ values are lower than the ones obtained for mine tailings. However, Suazo et al. (2016) tailings did not have binder and sand and therefore, their applicability to the current analysis is questionable.

Another simplification that was made in the current analysis was that the CRR values that were compared with CSR values were based on triaxial tests on fresh CPB while the CSR values were obtained using the time-dependent shear modulus. Hardened CPB would have a higher CRR (Saebimoghaddam 2010) and hence less prone to liquefaction. The use of elastic material is also questionable. In the next phase of analysis a more advanced constitutive will be integrated into the analysis.

CONCLUDING REMARKS

Liquefaction resistance of cemented paste backfill against a rockburst event was studied in this paper. A time domain method of analysis was performed for an underground stope backfilled with CPB using the coupled hydro-mechanical approach introduced by Shahsavari and Grabinsky (2016) and developed in FLAC3D. It was shown that the top 8 m of the CPB will liquefy if the rockburst event used in this study happens at the end of backfilling. However, the CPB at the bottom of the stope and close to where the barricade might be located was safe against liquefaction.

The comparison between the one-dimensional equivalent linear method of ground analysis (with no confinement) and the two-dimensional analysis (confined with surrounding rocks) showed that higher CSR values were developed in the fresh CPB surrounded by rock. Therefore, it is important to consider the rock boundary condition in a ground response analysis. Further analysis requires the application of different dynamic loads such as earthquakes or exploration blasts. In addition the stop height might have an effect on the results and its effect must be studied.

ACKNOWLEDGMENT

The first author is a recipient of an Itasca Education Partnership which provides training for, and use of Itasca codes including FLAC3D. The support of Jim Hazzard in Itasca's Toronto Office is gratefully acknowledged. Funding for this work has been provided by Barrick Gold Corporation as part of a larger investigation into the behavior of high performance paste backfill.

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