Paper No. 149

ISMS 2016

Minimum strength required for resisting cyclic softening/failure of cemented paste backfill at early age

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ABSTRACT
This paper attempts to assess empirically the liquefaction susceptibility of cemented paste backfill (CPB) at early age (≤ 7 days). Early age CPB can be categorized as a "clay-like" material because their plasticity index, PI ≥ 5 (PI = Liquid Limit, LL – Plastic Limit, PL). For clay-like material such as CPB, the liquefaction susceptibility can be characterized by the "cyclic softening" or "cyclic failure" which is assessed using an empirical method developed for clays and clay-like materials. This analysis allowed the determination of the minimum undrained shear strength required to resist cyclic softening (failure) of cemented paste backfills which is directly related to the unconfined compressive strength (UCS).
KEYWORDS: Mine tailings; cemented paste backfill, Binder; undrained shear strength; cyclic softening

1. INTRODUCTION

The mining industry generates significant socio-economic benefits, but also generates huge amounts of solid wastes such as mill tailings and waste rock. These solid wastes can generate environmental pollution due to their inadequate containment. Due to more stringent environmental regulations, cemented paste backfill (CPB) allow to return a large part of mill tailings (up to 50%) for underground open stopes filling, hence improving ground support and ore recovery (Potvin et al., 2005; Belem and Benzaazoua, 2008). CPB is a mixture of mill tailings with a binding agent and mixing water. The purpose of binder addition is to generate typical unconfined compressive strength (UCS) ranging from 500 kPa up to 4.5 MPa, depending on the type of backfill (slurry backfill, cemented rockfill, or cemented paste backfill).

CPB cost generally accounts for between 10% and 20% of the total operating cost of a mine from which hydraulic binder represents up to 75-80% of that cost (Grice, 1998). That is why mining companies seek to reduce the binder cost by reducing the amount of binder in the CPB mixtures. One of the promising options to reduce backfilling operation costs is a partial replacement of typical cement (i.e., general use Portland cement) by industrial by-products and or other supplementary cementitious materials (Belem and Benzaazoua, 2008). Unfortunately, a reduction in the amount of binder could lead to a substantial decrease in the mechanical properties of CPB, particularly at early age (from 0 to 7 days of curing). Such a reduction in the mechanical properties of CPB may trigger cyclic softening/failure at early age due to several sources (consecutive sequences of blasting, rock burst, seismic events, ground vibration, etc.). In common usage, liquefaction refers to the loss of strength in saturated, cohesionless material due to the build-up of pore water pressures during dynamic loading (Sladen et al., 1985).

The cyclic actions of an earthquake or blast detonation have the effect of increasing the potential for paste backfill softening, causing compression, which reduces the volume of voids by increasing pore water pressure as well. This implies a loss or a significant reduction of undrained shear strength due to pore pressure in the backfill, which means shear strain under constant volume. This is essentially due to rapid shaking, too short because the dissipation of pore pressure accumulated in the fluid may have started (Seed and Idriss, 1971).

The main objective of this paper is to assess empirically the cyclic softening (liquefaction susceptibility) of cemented paste backfill at early age (curing time ≤ 7 days) by providing preliminary results. The liquefaction susceptibility will be assessed through the "cyclic softening or failure" analysis (for clay-like materials such as CPB) based on empirical method. The specific aim is to verify experimentally whether the cost of binder can be reduced by lowering the CPB binder content while keeping sufficient undrained shear strength to resist cyclic softening. Hence, the empirical method should allow determining the minimum undrained shear

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strength related to the unconfined compressive strength (UCS) for resisting cyclic softening (or failure) of CPB.

2. SIMPLIFIED PROCEDURE FOR CYCLIC SOFTENING ASSESSMENT

2.1 Susceptibility to dynamic loading

For cohesive materials such as CPB, the cyclic action of an earthquake or blast detonation is deeply influenced by the number of cycles N of the earthquake, the relative density D_r (density index) and the grain size of the matrial. The response of soil to seismic loading varies with soil type and state (void ratio, effective confining stress, stress history, etc.). Seed et al. (2003) and Boulanger and Idriss (2005) distinguished between “sand-like” and “clay-like” behaviour. According to Seed et al. (2003), sand-like soils are susceptible to cyclic liquefaction when their behavior is characterized by Plasticity Index (PI = LL – PL) < 12 and Liquid Limit (LL) < 37 and natural water content (w_n) > 0.8(LL). Clay-like soils are generally not susceptible to cyclic liquefaction when their behaviour is characterized by PI > 12 but they can experience “cyclic softening”. These criteria are generally conservative. Boulanger and Idriss (2005) suggested that sand-like behaviour is limited to PI < 7, while clay-like behavior can be expected for fine-grained soils that have PI ≥ 7, although a slightly lower transition point for soils with a CL-ML classification (perhaps PI ≥ 5 or 6) would be equally consistent with the available data. Based on this soil classification, CPBs can be categorized as clay-like materials. Also, it was observed in the literature that the plasticity index, PI of uncemented mine tailings from hard rock varies between 1 and 10.

2.2 Empirical assessment of cyclic softening

Most of the existing work on cyclic liquefaction has been primarily for earthquakes. Seed et al. (2003) developed a comprehensive methodology to estimate the potential for cyclic liquefaction due to earthquake loading, originally developed by Seed and Idriss (1971). The evaluation procedure used worldwide is termed the “simplified procedure” (U.S. National Center for Earthquake Engineering Research, NCEER, 1998) as described by Youd (2001), which uses generally conservative assumptions. The simplified approach to evaluate the triggering of seismic liquefaction involves comparing the Cyclic Stress Ratio (CSR_M) caused by the design earthquake of magnitude M_w with the Cyclic Resistance Ratio (CRR_M) of the soil pertaining to an earthquake of magnitude M_w = 7.5. A factor of safety against liquefaction FS_{Liq} is defined as the ratio of $CRR_{M=7.5}$ to CSR_M:

$$FS_{\mathrm{Liq}} = \frac{CRR_{\mathrm{M=7.5}}}{CSR_{\mathrm{M=7.5}}} = \frac{CRR_{\sigma=1,\alpha=0} K_\sigma K_\alpha}{CSR_{\mathrm{M}}} MSF \tag{1}$$

where $CRR_{M=7.5}$ = cyclic resistance ratio pertaining to a magnitude 7.5 earthquake = $CRR_{\sigma=1,\alpha=0}$ = CRR for level ground conditions and an effective overburden stress (σ'_{v0}) of one atmosphere (≈ 100 kPa); K_σ = correction factor for the effects of σ'_{v0} on CRR; K_α = correction factor for the effects of static initial shear stress on CRR; α = static horizontal shear stress ratio ($\alpha = \tau_s/\sigma'_{v0}$); τ_s = static horizontal shear stress (kPa); *MSF* = Magnitude Scaling Factor (also called magnitude-correlated duration weighting DWF_M) for adjusting the induced *CSR* during earthquake of magnitude M_w to an equivalent *CSR* for an earthquake magnitude, M_w = 7.5. If α = 0, i.e. τ_s = 0 (no sloping), then K_α = 1.

The recommended MSF by the NCEER Workshop in 1998 (Youd, 2001) is given as follows:

$$MSF = \frac{174}{M^{2.56}} \qquad \text{Eq.(2)}$$

where M_w = moment magnitude of the earthquake. More recently, Idriss and Boulanger (2006) proposed an updated version given as follows:

$$MSF = 6.9\exp\left(\frac{-M}{4}\right) - 0.058 \le 1.8 \tag{3}$$

A simplified method to estimate CSR_M was also developed by Seed and Idriss (1971) based on the maximum (or peak) ground horizontal acceleration (PGA or a_{max}) at the site. The cyclic stress ratio proposed by Seed and Idriss (1971) is given as follows:

$$CSR_{\mathrm{M}} = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{\mathrm{v0}}}{\sigma'_{\mathrm{v0}}}\right) r_d \tag{4}$$

where a_{max} (= PGA) is in g (1g = 9.81 m/s^2); σ_{v0} = total vertical stress (kPa) and σ'_{v0} = effective vertical stress (kPa) at depth z (m). The parameter r_d in Eq. 4 is a stress reduction coefficient that accounts for the flexibility of the soil column. Youd (2001) proposed the following relations suggested by the NCEER (1998):

$$\begin{cases} r_{\mathrm{d}} = 1.174 - 0.0267z & 9.15\mathrm{m} < z \le 23\mathrm{m} \\ r_{\mathrm{d}} = 0.744 - 0.008z & 23.0\mathrm{m} < z \le 30\mathrm{m} \end{cases} \tag{5}$$

For depth z > 34 m, Idriss (1999) also proposed the following relationship:

$$r_d = 0.12 \cdot \exp(0.22 \cdot M) \tag{6}$$

Because of the cohesive nature of clay-like materials, they tend to develop smaller pore pressures under undrained cyclic loading than sand-like materials. Therefore, clay-like materials do not reach zero effective stress with resulting large deformations under cyclic loading (not susceptible to cyclic liquefaction). However, when the cyclic stress ratio (CSR) is large relative to the undrained shear strength ratio (S_u/σ'_{vc}; where S_u = undrained shear strength, σ'_{vc} = effective vertical confining stress) of clay-like materials, cyclic strain or softening can develop. However, post-earthquake volumetric strains tend to be small. Boulanger and Idriss (2005) used the term "cyclic failure" (instead of liquefaction) to describe this build-up of strain under cyclic loading in clay-like soils. Boulanger and Idriss (2004) showed that the CRR for cyclic failure in clay-like materials is controlled by the undrained shear strength ratio. These authors recommended the following relation for $CRR_{M=7.5}$ (for a moment magnitude 7.5 earthquake) of clay-like soils:

$$CRR_{M=7.5\alpha} = 0.80\left(\frac{S_u}{\sigma'_{vc}}\right)K \tag{7a}$$

and

$$K_\alpha = 1.344 - \frac{0.344}{\left(1 - \dfrac{\tau_s}{S_u}\right)^{0.638}} \tag{7b}$$

where K_α was previously defined in Eq. (1); (τ_s/S_u) = ratio of static initial shear stress and undrained shear strength. Boulanger and Idriss (2004) also suggested that there is no need for overburden correction factor K_σ which is taken implicitly into account in the undrained shear strength ratio.

3. PROCEDURE FOR CYCLIC FAILURE ASSESSMENT OF CPB

Knowing that S_u = UCS/2 for saturated CPB sample and that the most common parameter in backfill engineering is UCS, equation 7 along with equations 1 – 6 can be combined in the following forms, for a given total vertical stress σ_{v0} (= γ_{wet}z) at a given depth:

$$FS_{\text{Failure}} = \left(\frac{0.8 \cdot UCS}{2}\right)\left(\frac{g}{a_{\max}}\right)\frac{K_\alpha \cdot MSF}{(0.65 \cdot \sigma_{v0} \cdot r_d)}$$

$$= 0.6154\left(\frac{g}{a_{\max}}\right)\left(\frac{UCS \cdot K_\alpha}{\sigma_{v0} \cdot r_d}\right)MSF \tag{8}$$

where UCS = Unconfined Compressive Strength (kPa) of CPB; σ_{v0} = initial total vertical stress (kPa) at a given depth *z* (m); K_α = correction factor for static shear stress (always = 1 for vertical stopes but should be around 0.8-0.9 for sub-vertical stopes); a_{max}, r_d and *MSF* are previously defined.

Vibration amplitude in the CPB is weaker than that in the rock. The magnitude of vibration (or peak particle velocity PPV) recorded in CPB was in the range 25 – 65 mm/s (0.025 – 0.065 m/s). Signal frequencies are about 20 – 120 Hz (Liu, 2004). The PPV can be converted to peak ground horizontal acceleration (PGA or a_{max}) for sine waves using the following relation (e.g., Dowding, 1985):

$$a_{\max}\ (g) = \frac{2\pi \cdot PPV(\text{m}\cdot\text{s}^{-1}) \cdot f(Hz)}{9.81\ (\text{m}\cdot\text{s}^{-2})} \tag{9}$$

where PPV = peak particle velocity (m.s^{-1}); *f* = frequency or number of oscillations per second (Hz); 9.81 m/s^2 correspond to 1*g*.

The calculated a_{max} corresponding to blasts ranged from 0.3*g* to 5.0*g* compared to a minimum a_{max} value of 0.1*g* necessary to trigger cyclic liquefaction of soils. These values may seem too high, but it is for a very short period of time (0.07 – 0.13 sec). For comparison, the range of frequency band for strong motion (usually causing structural damage during strong ground shaking of about 5 – 30 seconds) is ~0.05 Hz to ~10 Hz (Berkley Seismological Laboratory).

A blast vibration can be compared to a strong motion (1 – 10 Hz) through the calculation of kinetic energy E_k released [E_k = $Mass_{CPB}$*PPV^2/(2*a_{max})]. The calculated E_k is in the range 10^{-3}*$Mass_{CPB}$ – 4*10^{-4}*$Mass_{CPB}$ for the observed magnitude of blast vibration PPV in the range of 25 – 65 mm/s (20 – 120 Hz). This corresponds to a range of equivalent moment magnitude M_w of 4.4 – 6.1 earthquakes with a maximum duration of 0.13 sec. The corresponding a_{max} will be in the range 0.009*g* to 0.08*g*. When considering that a real earthquake of magnitude 6 can last about 8 seconds compared to 0.13 seconds for the blast, a simple rule of three gives an equivalent actual energy released by the blast that should be (1/8)*(1.3/10) lower (approximately 62 times smaller than an actual earthquake).

From equation (8), the UCS values can be directly used, for the first time, to assess the cyclic failure (or softening) of CPB materials. Furthermore, equation (8) can be rearranged to provide the minimum compressive strength (UCS_{min}) required for resisting against cyclic failure or softening of CPBs due to strong motion. The minimum compressive strength ratio (UCS_{min}/σ_{v0}) is given as follows:

$$\frac{UCS_{min}}{\sigma_{v0}} = 1.625\left(\frac{a_{max}}{g}\right)\left(\frac{r_d}{K_\alpha \cdot MSF}\right)FS_{min} \qquad (10)$$

where FS_{min} = minimum factor or safety against cyclic failure of CPB (≥ 1.0); $\sigma_{v0} = \gamma_{wet} * z$ (total unit weight of CPB (kN/m^3) x depth of concern in a backfilled stope (m)).

If FS_{min} = 1.0, the Cyclic Resistance Ratio ($CRR_{M=7.5}$) is equal to the Cyclic Stress Ratio generated by a magnitude M_w earthquake (CSR_M/MSF). It is believed that this condition is not conservative enough to resist to cyclic failure triggering. Based on slope stability analysis principles, it may be recommended that the minimum factor of safety against cyclic failure should be FS_{min} = 1.1. Putting this value along with K_α = 1.0 into equation (10) yields:

$$\frac{UCS_{min}}{\sigma_{v0}} = 1.7875\left(\frac{a_{max}}{g}\frac{r_d}{MSF}\right) \qquad (11)$$

Equation (11) shows that the minimum compressive strength ratio (UCS_{min}/σ_{v0}) depends on the stress reduction coefficient r_d (see equation 5) that accounts for the flexibility of the CPB Mass.

It should be noted that UCS_{min} or UCS_{min}/σ_{v0} can be represented as a function of backfilled standalone stope depth *z* (or the total or overburden stress σ_{v0}), the maximum or peak ground horizontal acceleration (a_{max} or PGA) or Magnitude M_w earthquake.

4. SAMPLE APPLICATION

4.1 Cemented paste backfill mixtures preparation

The backfill specimens were prepared using tailings sampled from Brunswick's mine (BM). The CPB batch of mixtures were prepared by adding progressively the appropriate mass of tailings, binder and mixing water to a Hobart mixer and mixed for about 10 minutes. The binder type used was the high sulphate resistant Portland cement HS. Four different CPB mix recipes were considered with five binder contents (= mass of binder/mass of CPB): 1, 1.5, 1.75 and 2% (corresponding to binder ratio $B_{w\%}$ = mass of binder/mass of dry tailings = 1.22, 1.84, 2.15 and 2.47 wt%, respectively). For all the mixtures the average solid mass concentration $C_{w\%}$ was about 83% which correspond to standard slump height ranging between 140 and 165 mm (5½ and 6½ inches). The CPB mixtures were poured into 76 mm diameter and 152 mm height capped plastic molds (3 in x 6 in) and left to cure in a humidity chamber at ambient temperature (25°C) and > 90% relative humidity. The strength development is assessed for undrained noted UD (non-perforated molds) and drained noted FD (perforated molds). The curing times were 3, 7, 28 and 56 days.

4.2 Unconfined compression tests

The cyclic failure potential will be assessed on saturated CPB specimens at early age (≤ 7 days) curing times, through the unconfined compression tests in order to determine the UCS values which corresponds two time the undrained shear strength (UCS = 2*S_u), assuming Tresca criterion. The compression tests were performed using a MTS 10/GL universal hydraulic press of 50 kN loading capacity (compression rate of 1 mm/min).

5. RESULTS

5.1 UCS and S_u data

Figure 1 shows the relationship between binder content and UCS values. It can be observed that water drainage induces an increase in UCS due to self-weight consolidation. This figure can be used mainly to determine how the cement content can be reduced while maintaining the same strength performance of the CPB.

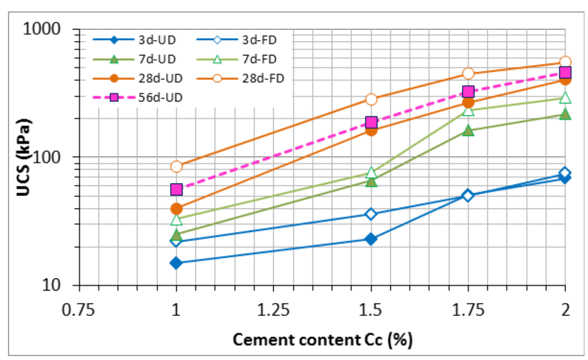

Figure 1: Relationship between binder content and UCS for drained and undrained CPB specimens

Table 1 contains the UCS data from Figure 1 and the corresponding calculated undrained shear strength S_u. The undrained strength S_u can be obtained directly from the UU triaxial test, direct simple shear (DSS) test, direct shear test (fast shear rate) or uniaxial compression test (saturated specimens). The simplest way is to calculate S_u from the UCS data (S_u

= UCS/2) assuming or ensuring that the CPB specimens are saturated (Belem and Benzaazoua, 2008 showed that when the CPB specimens are unsaturated, $S_u \neq$ UCS/2).

Table 1: UCS and corresponding undrained shear strength S_u values of the Brunswick's Mine CPB

Binder (%)	Curing (day)	UCS (kPa)	S_u = UCS/2 (kPa)	Relative consistency
1	3	15	7.5	Very soft
1	7	25	12.5	Soft
1.5	3	23	11.5	Very soft
1.5	7	66	33	Medium
1.75	3	51	25.5	Medium
1.75	7	162	81	Stiff
2	3	69	34.5	Medium
2	7	218	109	Very stiff

5.2 Cyclic failure potential of CPB

For this evaluation, 30 m stope height is considered. As the CPB is assumed fully saturated, the water table level is supposed to be on top of any stope (depth z = 0 m). It is assumed that the minimum pore water pressure before any shaking is the hydrostatic pressure ($u_0 = \gamma_w$*z = 9.81z kPa). Table 2 presents the total ($\sigma_{v0} = \gamma_{wet}$*z) and effective ($\sigma'_{v0} = [\gamma_{wet} - \gamma_w]$z) stresses calculated at backfilled stope depths of 15, 20 and 30 m (with tailings specific gravity of 4.03 and $C_{w\%}$ = 83%, the calculated γ_{wet} = 26 kN/m^3).

Table 2: Total and effective vertical stresses

Depth *z* (m)	σ_{v0} (kPa)	u_0 (kPa)	σ'_{v0} (kPa)
15	390	147	243
20	520	196	324
30	780	294	486

Table 3 presents the results of calculated factor of safety against cyclic failure $FS_{Failure}$ with a_{max} = 0.074g (earthquake moment magnitude M_w = 6), MSF = 1.482, r_d = 0.458, and K_α = 1 (no sloping of the backfill mass).

Table 3: Calculated factor of safety against cyclic failure or deformation for BM's CPB

binder (%)	Curing (day)	S_u = UCS/2 (kPa)	CSR (M=6)	CRR (M=7.5)	$FS_{Failure}$ (M=6)
1	3	7.5	0.024	0.0124	**0.52**
1	7	12.5	0.024	0.0206	**0.86**
1.5	3	11.5	0.024	0.0189	**0.79**
1.5	7	33	0.024	0.0544	**2.28**
1.75	3	25.5	0.024	0.0420	**1.76**
1.75	7	81	0.024	0.1334	**5.59**
2	3	34.5	0.024	0.0568	**2.38**
2	7	109	0.024	0.1795	**7.52**

If $FS_{Failure}$ > 1.0, the CPB will not fail cyclically; if $FS_{Failure} \leq 1$, the CPB is likely to undergo cyclic failure. From Table 3 it is clear that the CPB prepared with only 1% of binder ($B_{w\%}$ = 1.22%) will probably undergo cyclic failure or large softening ($FS_{Failure}$ < 1.0). The same observation is made for CPB with 1.5% of binder ($B_{w\%}$ = 1.84%) cured after 3 days only. Nevertheless, after 7 days of curing this CPB develops sufficient undrained shear strength to resist cyclic failure. Therefore, Table 3 suggests that the recommendable minimum binder proportion for the CPB to resist cyclic failure is 1.75% ($B_{w\%}$ = 2.15%). The binder proportion can however be lowered to 1.5% ($B_{w\%}$ = 1.84%) if the minimum allowed curing time is 7 days, but this is not safe.

Figure 2 shows the relationship between UCS_{min}/σ_{v0} required to resist cyclic failure as a function of backfilled stope depth z and for FS_{min} of 1.0 (critical case) and 1.1. These curves can be considered as "cyclic softening or failure curves" and are compared to the actual UCS data of the prepared cemented paste backfill. The shape of these curves is dictated by the stress reduction factor r_d.

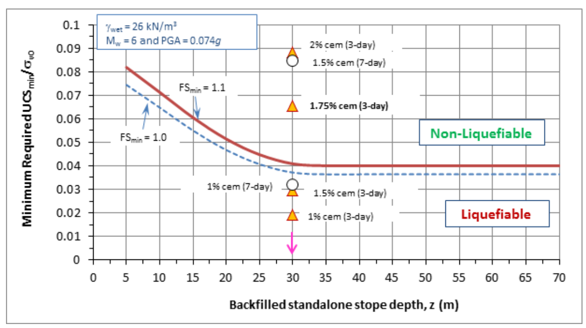

Figure 2: Relationship between UCS_{min}/σ_{v0} required for resisting cyclic failure as a function of stope depth z

6. DISCUSSION

The minimum compressive strength UCS_{min} required for resisting against cyclic failure of CPB specimens in this study at the bottom of 30 m height stope (σ_{v0} = 780 kPa) is 32 kPa only, for a moment magnitude M_w = 6 earthquake (a_{max} = 0.074g). The Japanese Port and Harbour Research Institute (1996) have observed similar low values for cemented sands. Their test results showed that when UCS was in the range 49 – 98 kPa, the cemented sand could not undergo liquefaction. Based on shaking table tests (a_{max} = 0.25g), they also found that by adding 1% (by dry mass of sand) of cement and after 7 days of curing the cemented sand (with an average UCS of 29 kPa) didn't liquefied. It was concluded that the percent of cement addition required for treatment to produce a material that will not undergo liquefaction differs according to the soil type. It should be noted that there are very few studies on the evaluation of liquefaction potential of cemented paste backfill (Blight, 1990; Belem et al., 2013). Blight (1990) suggests that, even very severe lateral accelerations (up to 10g!), will induce only moderate shear stresses (about 100 kPa) in the backfill contained in a narrow, tabular stope. Since then, it is customary to assume that a CPB having a UCS of about 100 kPa will never undergo liquefaction.

7. CONCLUSION

This paper presents the evaluation of the possibility to lower CPB binder content used for stope filling without causing liquefaction (or “cyclic failure”). The cyclic failure or softening (equivalent to liquefaction for clay-like materials such as CPB) was assessed based on empirical method proposed by leading world experts on liquefaction, namely Boulanger and Idriss (2005). This method is based on comparing the Cyclic Resistance Ratio of a soil subjected to shaking by an earthquake of M_w = 7.5 ($CRR_{M=7.5}$) to the Cyclic Stress Ratio imposed to the soil by a shaking due to an earthquake of M_w (CSR_M). A Magnitude Scaling Factor (MSF) is used for the calculation of the Factor of Safety against the cyclic failure $FS_{Failure}$ which must be higher than 1.0 to resist cyclic failure or softening. Sample application of this method shows that CPBs prepared with 1% of cement (at 3 and 7 days of curing) and 1.5% of cement at 3 days of curing will undergo cyclic failure or softening for an earthquake of magnitude M_w = 6 (a_{max} = 0.074g).

Based on the Boulanger and Idriss (2005) empirical model, a procedure using the UCS values is proposed for the assessment of cyclic failure of CPB in general. For the first time it is possible to connect the cyclic failure (softening) potential with the minimum required strength value (UCS_{min}) and the height of the backfilled stope. However, further study is needed for validating and refining the proposed method by performing cyclic direct simple shear (DSS) testing or direct shear test at fast shear rate on different CPB mixture recipes at early ages.

8. ACKNOWLEDGEMENT

This research was financially supported by NSERC discovery grant. The authors would like to acknowledge their helpful support.

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